TABLE OF CONTENTS

Section			Page

SECTION 1

INTRODUCTION

1-01 GENERAL

This manual is developed to present current Department policy pertaining to roadway design. It will provide a means of developing uniformity and safety in the design and plan preparation of a highway system consistent with the needs of the motoring public.

It is recognized that situations will occur where good engineering judgement dictates deviations from the normal design policy. Any such deviations from normal design policy shall be approved by the Chief Engineer, Design.

It is not the intent of this manual to reproduce all the information that is adequately covered by textbooks and other publications which are readily available to the designer and the technicians.

This manual, when used in conjuction with engineering knowledge of highway design and good judgement, should enable the designer to perform his job more efficiently.

Design of roadways on the Federal Aid Secondary System should conform to the standards as indicated in the "Secondary Road Plan, 1979" as approved by the FHWA.

1-02 POLICY ON USE OF AASHTO STANDARDS

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

1-03 REFERENCE PUBLICATIONS

1. AASHTO (AASHO) Publications - American Association of State Highway and Transportation Officials

   - A Policy on Geometric Design of Rural Highways

     (1965)

   - A Policy on Design of Urban Highways and Arterial Streets

     (1973)

   - Highway Definitions (1968)

- A Policy on Design Standards for Stopping Sight Distance
- Guide for Selecting, Locating, and Designing Traffic Barriers (1977)

2. Highway Research Board
   - Highway Capacity Manual (1965) SR 87
3. Manual on Uniform Traffic Control Devices
4. FHWA Federal-Aid Highway Program Manual (FHPM)

SECTION 2

GENERAL DESIGN CRITERIA

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.

2-02 HIGHWAY CLASSIFICATION

Highway classification refers to a process by which roadways are classified into a set of sub-systems, described below, based on the way each roadway is used. Central to this process is an understanding that travel rarely involves movement along a single roadway. Rather each trip or sub-trip initiates at a land use, proceeds through a sequence of streets, roads and highways, and terminates at a second land use.

The highway classification process is required by federal law. Each state must assign roadways into different classes in accordance with standards and procedures established by the Federal Highway Administration. Separate standards and procedures have been established for rural and urban areas. For a futher description of the classification process, see Highway Functional Classification (FHWA, July 1974, Transmittal #155).

2-02.1 Principal Arterial Highways

Principal arterial highways form an inter-connected network of continuous routes serving corridor movements having the highest traffic volumes and the longest trip lengths. In rural areas, travel patterns should be indicative of substantial statewide or interstate travel. In urban areas, principal arterials should carry a high proportion of total urban area travel on a minimum of mileage.

The principal arterial highway system is stratified into the following two sub-systems:

1. Interstate system -- all presently designated routes of the Interstate System.

2. Other principal arterials -- all non-Interstate principal arterials.

"Other principal arterial" highways may be freeways, expressways or land service highways (see types of highways). However, because of the function of principal arterial highways, the concept of service to abutting land should be subordinate to the provision of travel service to major traffic movements. Where permitted, direct access to abutting property should be carefully regulated by license. No absolute right exists for access to a principal highway, and the rights of the travelling public to a safe and efficient roadway must be guaranteed. However, abutting property owners do have aright of reasonable access to the system of highways, unless such right has been acquired by the State.

Except for toll roads, most "other principal arterials" are included in the Federal Consolidated Primary (FAP) highway system.

2-02.2 Minor Arterial Highways

highway

Minor arterial highways interconnect with and augment the principal higway system. In urban areas, minor arterial highways are ususally included in the Federal Aid Urban System (FAUS), and serve trips of moderate length at a somewhat lower level of travel mobility. Access to abutting property should be minimized to facilitate traffic flow and safety. In rural areas, minor arterial highways will usually be included in the Federal Consolidated Primary (FAP) system, and serve trip lengths and travel densities greater than those served by collector roads. Rural minor arterials should provide relatively high overall travel speeds, with minimum interference to through movements. Because of the high speeds, access to abutting property should be either controlled or carefully regulated.

2-02.3 Collector Roads

Collector roads primarily serve trips of intracounty rather than statewide importance. Travel speeds and volumes are less than on arterial roadways, but are still high relative to local roads. These roads provide for both land access and traffic circulation. In urban areas, these roads connect neighborhoods or other districts with the arterial system, and will usually be part of the Federal Aid Urban System (FAUS). In rural areas, these roads may be subclassified into two groups:

Major collectors -- Serve important intracounty traffic corridors and provide service to major county traffic generators. These roads will usually be included in the Federal Aid Secondary (FAS) system.

Minor collectors -- Serve smaller places and towns and connect locally important traffic generators. These roads usually will not be on a federal aid system.

2-02.4 Local Roads

The local street and road system constitutes all roads not included in the higher classifications. These streets and roads provide direct access to abutting land and permit access to the roads of higher classification. They offer the lowest level of mobility. Service to through traffic movement usually is deliberately discouraged, especially in urban areas. The local road system contains the large majority of all roadway mileage in a state, but only a small percentage of total traffic. For example, in New Jersey local roads include 67% of total road mileage, but only 20% of total vehicular miles travelled.

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 Classification

   . Topography and Physical Features

   . Traffic

2. Secondary controls

   . Design Speed

   . Design Vehicle

   . Capacity

2-03.2 Primary Controls

1. Highway Classification

   Separate design standards are appropriate for different classes of roads, since the classes serve different types of trips and operate under different conditions of both speed and traffic volume. Tables 2-2 thru 2-4 indicate the values which are intended to be applied to the various classes of roadways utilized in the State. In special cases of restrictive or unusual conditions, it may not be practical to meet these guide values. For more detailed descriptins of the various guide values indicated, reference is made to other Sections of this Manual.

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 a 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.

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 loadpower characteristics are some of the aspects taken into account.

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

ADT Average Daily Traffic. The total volume during a given time period greater than one day but less than one year divided by the number of days actually counted.

AADT Average Annual Daily Traffic. The total yearly volume in both directions of travel divided by 365 days.

DHV The design-hourly volume. Normally estimated as the 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 the 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 highway. For through roads, 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 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.

Generally on new highways or highways on new alignment, the design speed should be considered as 10 mph greater than the posted speed or anticipated posted speed on the highway under design. On existing highways, the design speed should be 5-10 mph greater than the posted speed.

2. <u>Design Vehicle</u>

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 established 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 Vehicles

Design Vehicle		Dimensions in Feet*			Overall Length	Overall Width
		Wheel-	Overhang			
Type	Symbol	base	Front	Rear		
Passenger Car	P	11	3	5	19	7
Single Unit Truck	SU	20	4	6	30	8.5
Single Unit Bus	BUS	25	7	8	40	8.5
Semitrailer Combination, Intermediate	WB-40	13+27= 40	4	6	50	8.5
Semitrailer Large	WB-50	20+30=	3	2	55	8.5
Semitrailer-Full Trailer Combination	WB-60	9.7+20 +9.4+ 20.9= 60	2	3	65	8.5

Source: A Policy on Design of Urban Highways and Arterial Streets, 1973
*Design vehicle dimensions are intended for use in the design of roadways and do not define the legal vehicle dimensions in the State.

3. Capacity

a. General

The term "capacity" is used to express the maximum number of vehicles which have a reasonable expectation of passing over a section of a lane or a roadway during a given time period under prevailing roadway and traffic conditions. However, in a broad sense, capacity encompasses the relationship between highway characteristics and conditions, traffic composition and flow patterns, and the relative degree of congestion at various traffic volumes throughout the range from light volumes to those equaling the capacity of the facility as defined above.

Highway capacity information serves three general purposes:

(1) For transportation planning studies to assess the adequacy or sufficiency of existing highway networks to current traffic demand, and to estimate when, in time, projected traffic demand, may exceed the capacity of the existing highway network or may cause undersirable congestion on the highway system.

(2) For identifying and analyzing bottleneck locations (both existing and potential), and for the evaluation of traffic operational improvements projects on the highway network.

(3) For highway design purposes.

b. Level of Service

The level of service concept places various traffic flow conditions into 6 levels of service. These levels of service, designated A through F, from best to worst, cover the entire range of traffic operations that may occur.

The factors that may be considered in evaluating level of service include the following.

(1) Speed and travel time
(2) Traffic interruptions or restrictions
(3) Freedom to maneuver
(4) Safety
(5) Driving comfort and convenience
(6) Economy

However, in a practical approach to identifying the level of service, travel time and the ratio of demand volume to capacity are commonly used.

In general, the various levels of service would have the following characteristics.

(1) Level of Service A - free flow, with low volumes and high speeds. Traffic density is low, with speeds controlled by driver desires, speed limits, and physical roadway conditions. There is little or no restriction in maneuverability due to presence of other vehicles, and drivers can maintain their desired speeds with little or no delay.

(2) Level of Service B is in the zone of stable flow, with operating speeds beginning to be restricted somewhat by traffic conditions. Drivers still have reasonable freedom to select their speed and lane of operation. Reductions in speed are not unreasonable, with a low probability of traffic flow being restricted. The lower limit (lowest speed, highest volume) of this level of ~~of~~ service has been associated with service volumes used in the design of rural highways.

(3) Level of Service C is still in the zone of stable flow, but speeds and maneuverability are more closely controlled by the higher volumes. Most of the drivers are restricted in their freedom to select their own speed, change lanes, or pass. A relatively satisfactory operating speed is still obtained, with service volumes perhaps suitable for urban design practice.

(4) Level of Service D approaches unstable flow, with tolerable operating speeds being maintained though considerably affected by changes in operating conditions. Fluctuations in volume and temporary restrictions to flow may cause substantial drops in operating speeds. Drivers have little freedom to maneuver, and comfort and convenience are low, but conditions can be tolerated for short periods of time.

(5) Level of Service E cannot be described by speed alone, but represents operations at even lower operating speeds than in Level D, with volumes at or near the capacity of the highway. At capacity, speeds are typically, but not always, in the neighborhood ~~of~~ of 30 mph. flow is unstable, and there may be stoppages of momentary duration.

(6) Level of Service F describes forced flow operation at low speeds, where volumes are below capacity. These conditions usually result from queues of vehicles backing up from a restriction downstream. The section under study will be serving as a storage area during parts or all of the peak hour. Speeds reduced substantially and stoppages may occur for short or long periods of time because of the downstream congestion. In the extreme, both speed and volume can drop to zero.

Reference is made to the 1965 Highway Capacity Manual for a thorough discussion on the level of service concept.

c. Service Volume

For highway design purposes, the service volume is related to the "Level of Service" selected for the proposed facility. (No service volumes are defined for Level of Service F.) Service volume is defined as the maximum rate of flow which may be accommodated under prevailing traffic and roadway conditions while still maintaining a quality of service appropriate to the indicated Level of Service. The service volume varies with a number of factors, including:

(1) Level of service selected;
(2) Width of lanes;
(3) Number of lanes;
(4) Presence or absence of shoulders;
(5) Grades;
(6) Horizontal alignment;
(7) Operating speed;
(8) Lateral clearance;
(9) Side friction generated by parking, driveways, intersections, and interchanges;
(10) Volumes of trucks, buses, and recreational vehicles;
(11) Spacing and timing of traffic signals.

The objective in highway design is to create a highway of appropriate type with dimensional values and alignment characteristics such that the resulting service volume will be at least as great as the design volume, but not much greater as to represent extravagance or waste. More detailed data on service volume are available in the "Highway Capacity Manual" published by the Highway Research Board in 1965 and AASHTO "A Policy on Design of Urban Highways and Arterial Streets, 1973".

Table 2-2

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 Traffic Projection	20yrs. New Alignments 10yrs. Resurfacing	20 yrs. New Alignment 10 yrs. Resurfacing	20 yrs. Alignment 10 yrs. Resurfacing
Number of Lanes	2 to 8	2 to 4	2
Lane Width, Feet	12 Desirable 11 Minimum	12 Desirable 10 Minimum	10-12
Median Width (If Applicable)	8' Minimum w/barrier 32' Minimum without barrier	16' minimum	-----
Right Shoulder Width, Feet Desirable	12	10'	8'
Left Shoulder Width, Feet Desirable (If Applicable)	5' (4 lanes) 10' (6-8 lanes)	5'	-----
Max. Superelevation, Percent	6 desirable 4 minimum	4	-----
Curve Radius	Fig. 4-C	Fig. 4-C	Fig. 4-C
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, % maximum	6 Level 8 Rolling/Mt.	7 Level 10 Rolling/Mt.	8 Level 18 Mt.

Table 2-3

General Design Controls for Rural Roads and Highways

Highway Type	Arterial	Collector	Local
Design Speed, MPH	Level 50-65 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 Traffic Projections	20 yrs. New Alignment 10 yrs. Resurfacing	20 yrs. New Alignment 10 yrs. Resurfacing	20-yrs. 10 yrs. Resurfacing
Number of Lanes	2 to 6	2 to 4	2
Lane Width, Feet	12 Desirable 11 Minimum	12 Desirable 10 Minimum	12 Desirable 10' minimum
Median Width (If Applicable)	46' Desirable	36' Desirable	-----
Right Shoulder Width, Feet Desirable	12	10	8
Left Shoulder Wideth Feet, Desirable (If Applicable)	5' (4 lanes) 10' (6 lanes)	5'	-----
Max. Superelevation, Percent	6	6	-----
Curve Radius	Fig. 4-B	Fig. 4-C	Fig. 4-C
Stopping Sight Distance	Table 4-1	Table 4-1	Table 4-1
Passing Sight	Table 4-1	Table 4-1	Table 4-1
Profile Grade,% Maximum	Level 3 Rolling 4 Mount. 6	Level 6 Rolling 9 Mount 12	15 Max.

Table 2-4

General Design Controls Interstate and for Freeways

Location	Urban	Rural
Design Speed, MPH	50-70	70
Level of Service	C	B
Design Vehicle	SU;WB-50	SU; WB-50
Design Traffic Projections	20 yrs. 10 yrs. Resurfacing	20 yrs. 10 yrs. Resurfacing
Number of Lanes	4 to 8	4 to 6
Lane Width, Feet	12	12
Median Width (If Applicable	46' Desirable	84' Desirable
Shoulder Width, Feet Desirable	12 Right 5 Left (4 lanes) 10 Left (6-8 lanes)	12 Right 5 Left (4 lanes) 10 Left (6-8 lanes)
Max. Superelevation, %	6	6
Curve Radius	Fig. 4-E	Fig. 4-E
Stopping Sight Distance	Table 4-1	Table 4-1
Profile Grade, % Maximum	3 level 6 for Rolling or Mount Terrain	3 level 8 for Hilly Terrain

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.

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

A general term denoting a public way for purposes of vehicular travel, including the entire area within the right-of-way lines. Recommended usage in urban areas, highway or street; in rural areas, highway or road.

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, including shoulders, for vehicular use.

4 Traveled Way

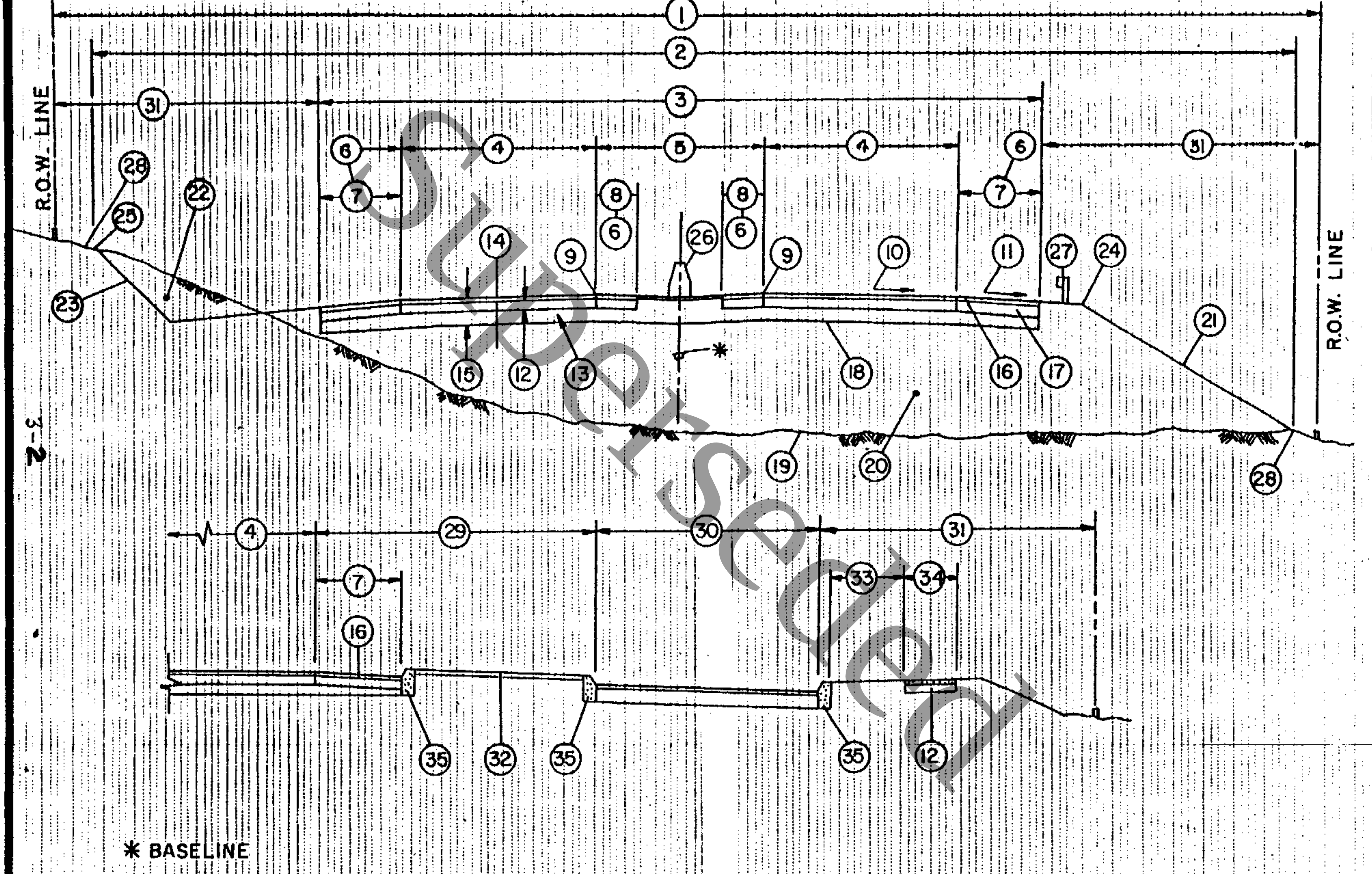
The portion of the roadway provided for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

5 Median

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

6 Shoulder

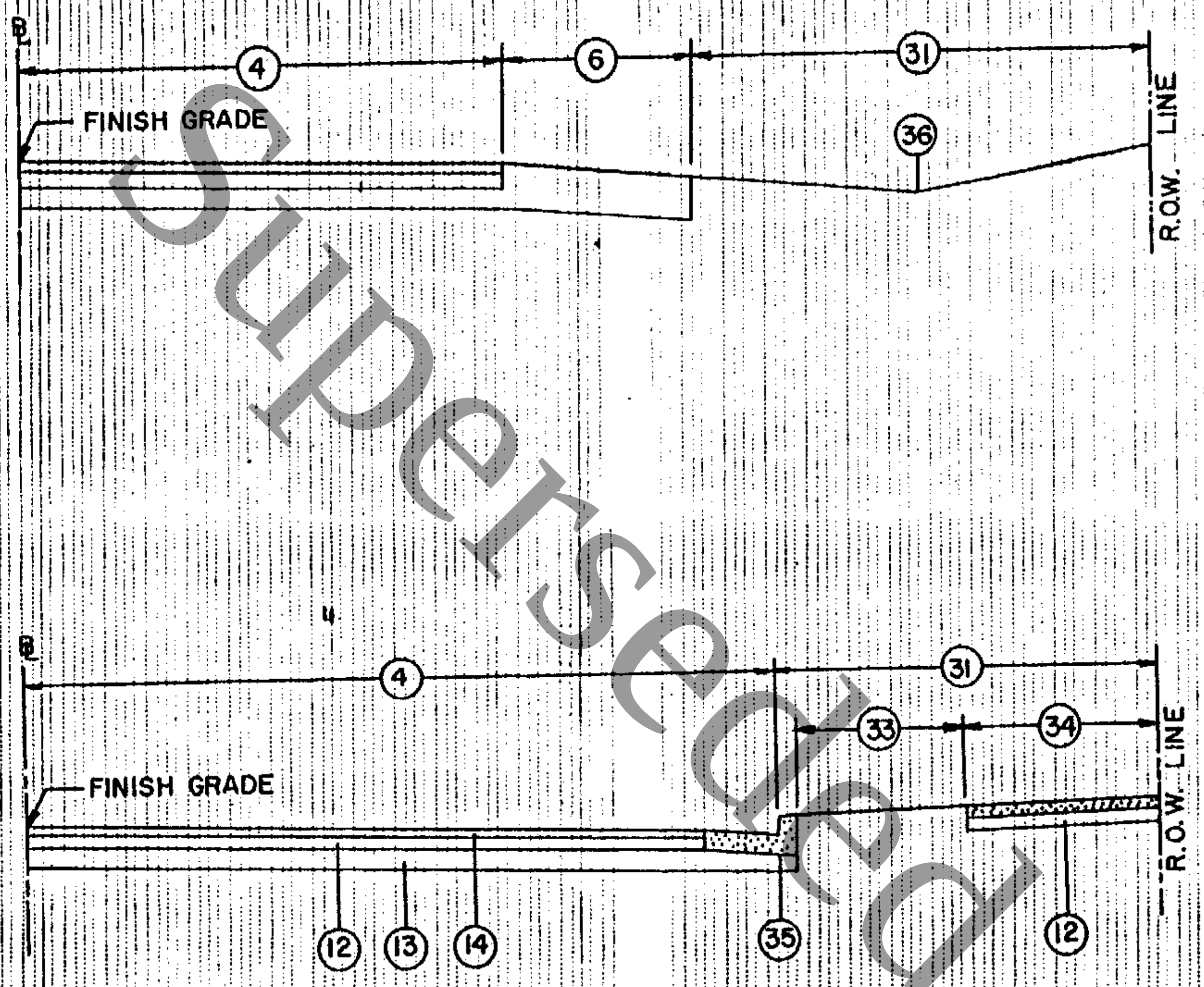
The portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles for emergency use, and for lateral support of the base and surface courses.



ROAD CROSS SECTION TERMINOLOGY

FIGURE 3-A

10/24/83



ROAD CROSS SECTION TERMINOLOGY

FIGURE 3-B

10/24/83

7 Surfaced Right Shoulder

That portion of the outside paved shoulder to provide all-weather load support.

8 Surfaced Left Shoulder

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

9 Profile 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 subgrade to support a surface course.

13 Subbase

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 accomodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintergrating effects of climate.

15 Pavement Structure

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 (P.V.I.)

The intersection of shoulder slope planes with fill slope planes.

25 Rounding

At the intersection of existing ground and cut slope.

26 Median Barrier

A longitudinal barrier used to prevent an errant vehicle from crossing the portion of a divided highway separating the travelled ways for traffic in opposite directions.

27 Guiderail

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

28 Top of Slope

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 road.

30 Frontage Road

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 adjoining the outer edge of the roadway (normally applies to freeways). The term "border" or "sidewalk area" is usually referred to street type facilities.

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

3-03 GENERAL TERMS

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

Auxilary Lane - The portion of the roadway adjoining the travelled way intended for speed change, storage, weaving, climbing lane, and for other purposes supplementary to through traffic movement.

Acceleration Lane - An auxilary lane including tapered areas, primarily for the acceleration of vehicles entering the through traffic lanes.

Deceleration Lane - An auxilary lane including tapered areas, primarily for the deceleration of vehicles leaving the through traffic lanes.

Capacity - The maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway in one direction or in both directions for a two-lane or a three-lane highway during a given time period under prevailing roadway and traffic conditions.

Climbing Lane - An auxiliary lane introduced at the beginning of a sustained positive grade in the direction of traffic flow, to be used by slow moving vehicles such as trucks and buses.

Collector-Distributor Road, (C-D) - An auxiliary roadway separated laterally from, but generally parallel to, an expressway which serves to collect and distribute traffic from several access connections between selected points of ingress to and egress from the through traffic lanes. Control of access is exercised outside a C-D Road.

Control of Access - The condition where the rights of owners, occupants or other persons of land abutting a highway to access, light, air or view in connection with the highway are fully or partially controlled by a public agency.

> Full Control - The condition under which the authority to control access is excercised to give preference to through traffic to a degree, but in addition to interchange connections with selected public roads there may be some intersections at grade.
>
> Partial Control - The condition under which the authority to control access is excercised to give preference to through traffic to a degree that, in addition to access connections with selected public roads, there may be some crossings at grade and some private driveway connections.
>
> ~~Full Control~~ - The condition under which the authority ~~to~~ control access ~~is exercised~~ to give ~~perference~~ to through traffic to a degree, but in addition to interchange connections with selected public roads there may be ~~some intersections~~ at grade.

Corridor - A strip of land between two termini within which traffic, topography, environment and other characteristics are evaluated for transportation purposes.

Cul-de-Sac - A local street or road open at only one end with special provisions for turning around.

Dead-End Road - A local street or road open only at one end without special provisions for turning around.

Density - The number of vehicles per mile on the traveled way at a given instant.

Design Year - The year, generally twenty years or more as determined, after completion of constuction, whose estimated traffic volumes are used as a basis for design.

<u>Direct Connection</u> - A one-way turning roadway which does not deviate greatly from the intended direction of travel.

<u>Directional Design Hourly Volume, (DDHV)</u> - An hourly volume determine for use in design, representing traffic expected to use one direction of travel on a highway. (Unless otherwise stated it is the directional hourly volume during the 30th highest hour).

<u>Diverging</u> - The dividing of a single stream of traffic into separate streams.

<u>Divided Highway</u> - A highway street or road with opposing directions of travel separated by a median.

<u>Expressway</u> - A divided multi-lane arterial highway for through traffic with full or partial control of access and generally with grade separations at major intersections. On rare occasions expressways may also include two lane roadways.

<u>Freeway</u> - An expressway with full control of access and grade separations at all intersectins.

<u>Frontage Road or Frontage Street</u> - 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.

<u>Gore</u> - The area immediately beyond the divergence of two roadways, bounded by the edges of those roadways.

<u>Grade Separation</u> - A crossing of two highways or a highway and a railroad at different levels.

<u>Highway Overpass</u> - A grade separation where the subject highway passes over an intersecting highway or railroad.

<u>Highway Underpass</u> - A grade separation where the subject highway passes under an intersecting highway or railroad.

<u>Inside Lane</u> - On a multi-lane highway the extreme left hand traffic lane, in the direction of traffic flow, of those lanes available for traffic moving in one direction. Also referred to as left lane.

<u>Interchange</u> - A system of interconnecting roadways providing for the movement of traffic between intersection legs.

Land Service Highway - An arterial or collector highway on which access to abutting property is permitted. On arterial highways and major collector roads, such access is usually regulated in order to protect the public safety and maintain the efficiency of the highway.

Left Turn Slot - A speed-change lane within the median to accommodate left turning vehicles.

Loads - Traffic data required for the establishment of geometric controls for highway design.

Major Street or Major Road - 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.

Separated Roadways - ~~A~~ Highways with opposing directions of travel having independent alignment and gradient.

Sight Distance - The length of roadway visible to the driver of a vehicle at a given point on the roadway when the view is unobstructed.

Slip Ramp - An angular connection between an expressway and a parallel frontage road.

Stopping Sight Distance - The distance required by a driver of a vehicle, traveling at a given speed, to bring his vehicle to a stop before reaching an object on the roadway after the object has become visible. (The distances used in design are calculated on the basis of the driver's ability to see a 6-inch high object in the road ahead when his eye level is 3 feet - 9 inches above the roadway surface).

Thirtieth Highest Hourly Volume (30HV) - The hourly volume in both directions of travel that is exceeded by 29 hourly volumes during a designated year.

Through Lane - The lane or lanes signed for through traffic continuing through an interchange area.

Through Street, Road or Highway - Any roadway, or portion thereof, on which vehicular traffic is given preferential right-of-way, and at the entrances to which vehicles from intersecting highways are required by law to either stop or yield.

Traffic Lane - The portion of the roadway for the movement of a single line of vehicles.

Weaving - The crossing of traffic streams moving in the same general direction, accomplished by merging and diverging.

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 alingment, 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 principal 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

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.

Table 4-1

Sight Distances for Design

Design Speed MPH	Sight Distance in Feet Stopping Desirable	Minimum	Passing* Minimum
20	120	120	-----
25	150	150	-----
30	200	200	1,100
35	250	225	1,315
40	300	275	1,500
50	450	350	1,800
60	650	475	2,100
70	850	600	2,500

*Not applicable to multi-lane highways.

4-02.2 Passing Sight Distance

Passing sight distance is the minimum sight distance that must be available to enable the drive 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 of any place is the longest distance at which a driver whose eyes are 3.75 feet above the pavement surface can see the top of an object 4.25 feet high on the road.

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.

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 3.75 feet above the pavement surface, to an object 0.5 foot high on the road.

The stopping sight distances shown in table 4-1 should be increased when sustained downgrades are steeper than 3 percent. Increases in the stopping sight distances on downgrades are indicated in the AASHTO 1965 A Policy on Geometric Design of Rural Highways pg. 139.

4-02.5 4 Stopping Sight Distance on Vertical Curves

See Section 4-04.4 for discussion on vertical curves.

4-02.6 5 Stopping Sight Distance on Horizontal Curves

Where an object off the pavement such as a bridge pier, bridge rail, 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-A. It is assumed that the driver's eyes are 3.75 feet above the center of the inside lane (inside with respect to curve) and the object is 0.50 foot high. The line of sight is assumed to intercept the view obstruction at the midpoint of the sight line and 2 feet 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 the 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-A gives the clear distance M from centerline of inside lane to the obstruction.

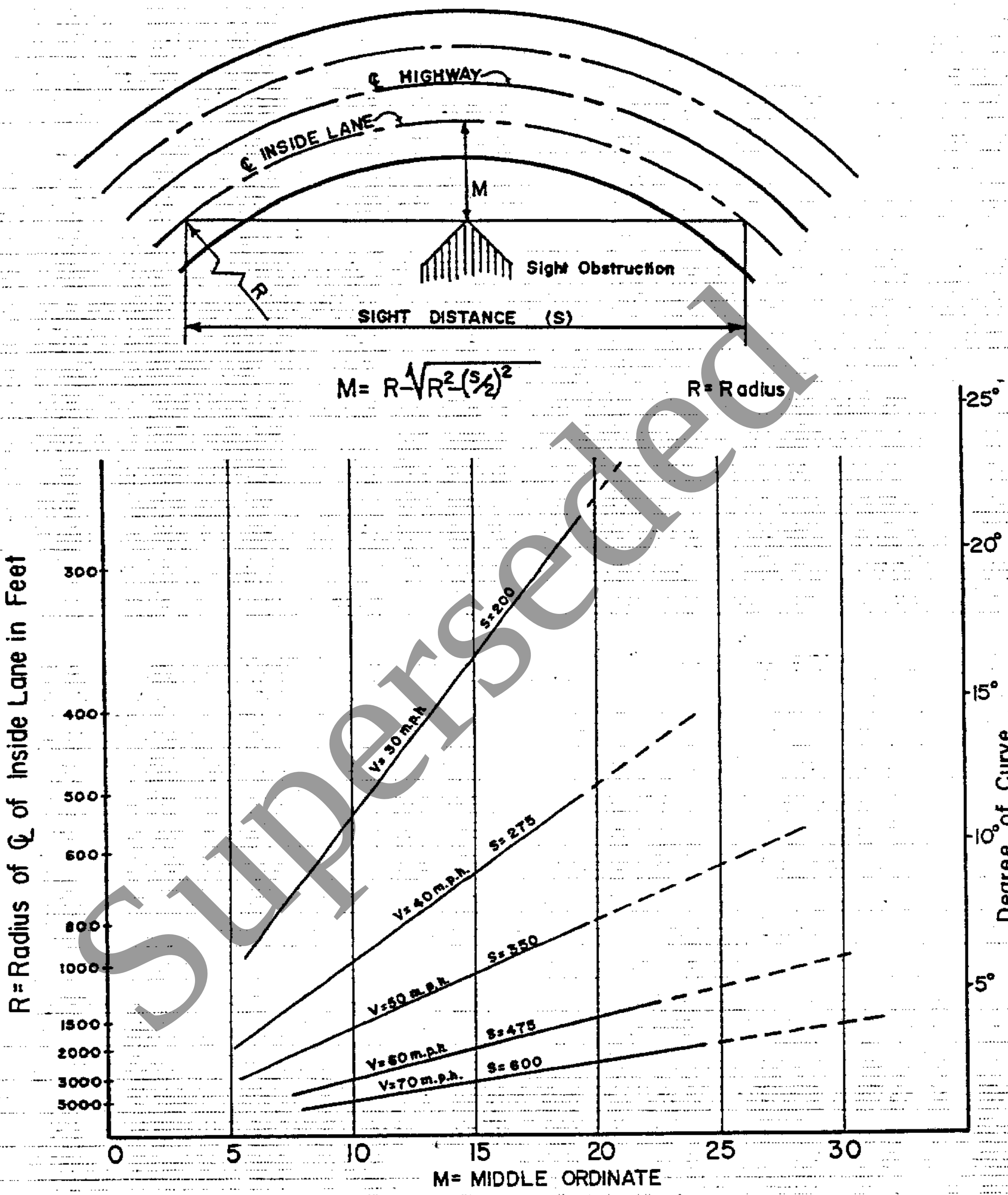
When the design speed and the clear distance to a fixed obstruction are known, this figure also gives the required minimum radius which satisfies these conditions.

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, type of facility, design speed, topography and construction cost. In design, safety is always considered, either directly or indirectly. Topography controls both curve rarius 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, adquate for the design classification of the roadway or highway.



DESIRABLE STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES

FIG. 4-A

Source: A Policy on Design of Urban Highway and Arterial Streets: AASHTO 1973

10/24/83

3

4-02.2 Superelevation

When a vehicle travels on a circular curve, it is forced radially outward by centrifugal force. This effect becomes more pronounced as the radius of the curve is shortened. This is counter-balanced by providing roadway superelevation and by the side friction between the vehicle tires and the surfacing. Safe travel at different speeds depends upon the radius of curvature, the side friction, and the rate of superelevation.

The maximum rates of superelevation used by the Department is 6% for all expressways and other major rural highways, and 4% on local roads and urban highways.

Figures 4-B & 4-C give the design values for each rate of superelevation to be used for various design speeds and radii.

1. Axis of Rotation

a. Undivided Highways

For undivided highways, the axis of rotation for superelevation is usually the centerline of the traveled way. 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 rotation.

c. Divided Highways

(1) Freeways

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

Where the initial median width is greater than 30 feet and the ultimate median width is 30 feet 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.

SUPERELEVATION - 6% MAXIMUM RATE

FIGURE 4-B

SUPERELEVATION (PERCENT) FOR DESIGN SPEEDS OF					
RADIUS (ft)	30 MPH	40 MPH	50 MPH	60 MPH	70 MPH
300	5.9				
400	5.6				
500	5.1				
600	4.7	5.9			
700	4.4	5.7			
800	4.1	5.4			
900	3.9	5.1	6.0		
1,000	3.7	4.9	5.9		
1,200	3.3	4.5	5.5		
1,400	2.9	4.1	5.2	6.0	
1,600	2.7	3.8	4.9	5.8	
1,800	2.4	3.6	4.6	5.5	
2,000	2.2	3.3	4.3	5.3	5.9
2,500	1.8	2.8	3.8	4.7	5.6
3,000	1.6	2.4	3.4	4.3	5.1
3,500	1.5	2.1	3.0	3.9	4.7
4,000	1.5	1.9	2.7	3.5	4.3
4,500	NC	1.7	2.5	3.2	3.9
5,000		1.6	2.2	3.0	3.6
6,000		1.5	1.9	2.6	3.1
7,000		1.5	1.7	2.3	2.8
8,000		NC	1.5	2.0	2.5
9,000			1.5	1.8	2.2
10,000			1.5	1.6	2.0
12,000			NC	1.5	1.7
14,000				1.5	1.5
16,000				NC	1.5
18,000					1.5
19,000					NC

NC = Normal Crown

NO SUPERELEVATION REQUIRED WHEN RADIUS (FEET) IS GREATER THAN

30 MPH	40 MPH	50 MPH	60 MPH	70 MPH
4,250	7,160	10,810	14,690	19,100

TRANSITION NOT ESSENTIAL WHEN RADIUS IS GREATER THAN

30 MPH	40 MPH	50 MPH	60 MPH	70 MPH
1,500	3,000	4,000	6,000	8,000

11/9/83

SUPERELEVATION - 4% MAXIMUM RATE

FIGURE 4-C

SUPERELEVATION (PERCENT) FOR DESIGN SPEEDS OF				
RADIUS (ft)	30 MPH	40 MPH	50 MPH	60 MPH
300	4.0			
400	3.8			
500	3.6			
600	3.4	4.0		
700	3.2	3.9		
800	3.0	3.8		
900	2.8	3.7		
1000	2.7	3.5	4.0	
1200	2.5	3.3	3.9	
1400	2.4	3.1	3.7	
1600	2.2	2.9	3.5	4.0
1800	2.1	2.7	3.3	3.9
2000	2.0	2.6	3.2	3.8
2400	1.7	2.4	3.0	3.6
2800	1.5	2.2	2.8	3.4
3200	1.5	2.0	2.6	3.2
3600	1.5	1.9	2.4	3.0
4000	1.5	1.8	2.3	2.8
4500	NC	1.6	2.1	2.7
5000		1.5	2.0	2.5
6000		1.5	1.7	2.3
7000		1.5	1.5	2.1
8000		NC	1.5	1.9
9000			1.5	1.5
10000			1.5	1.5
12000			NC	1.5
14000			NC	1.5

NC = Normal Crown

SUPERELEVATION NOT REQUIRED WHEN RADIUS IS GREATER THAN

30 MPH	40 MPH	50 MPH	60 MPH
4250	7160	10810	14690

11/10/83

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

To avoid a 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.

2. Superelevation Transition

The superelevation transition generally consists of the superelevation runoff (length of roadway needed to accomplish the change in cross slope from a normal crown section to a fully superelevated section or vice versa).

The superelevation transition should be designed to satisfy the requirements of safety and comfort and be pleasing in appearance. The length of superelevation transition should be based on a desirable distribution rate of 2% per second of time for the design speed. 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.

After a superelevation transition is designed, profiles of the edges of pavement and shoulder should be plotted and irregularities removed by introducing smooth curves by the means of a graph profile. 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.

3. Transition Curves & Superelevation

The use of transition curves on arterial highways is encouraged. Figures 4D thru 4H enclusive indicate the desirable treatment on highway curves including the method of distributing superelevation.

4-03.3 Curvature

1. General

The changes in direction along a highway are basically accounted for by simple curves or compound curves. Excessive curvature or poor combinations of curvature generate accidents, limit capacity, cause economic losses in time and operating costs, and detract from a pleasing appearance. To avoid these evidences of poor design practices, the general controls in the following paragraphs should be used.

2. Curve Radii for Horizontal Curves

Table 4-2 gives the minimum radius of a curves for specific design speeds. This table is based upon a 6% maximum superelvation; it ignores the sight distance factor.

Table 4-2

Standards for Curve Radius*

Design Speed Miles Per Hour	Minimum Radius of Curve--Feet
20 -----------	125
30 -----------	275
40 -----------	500
50 -----------	850
60 -----------	1,275
70 -----------	1,910

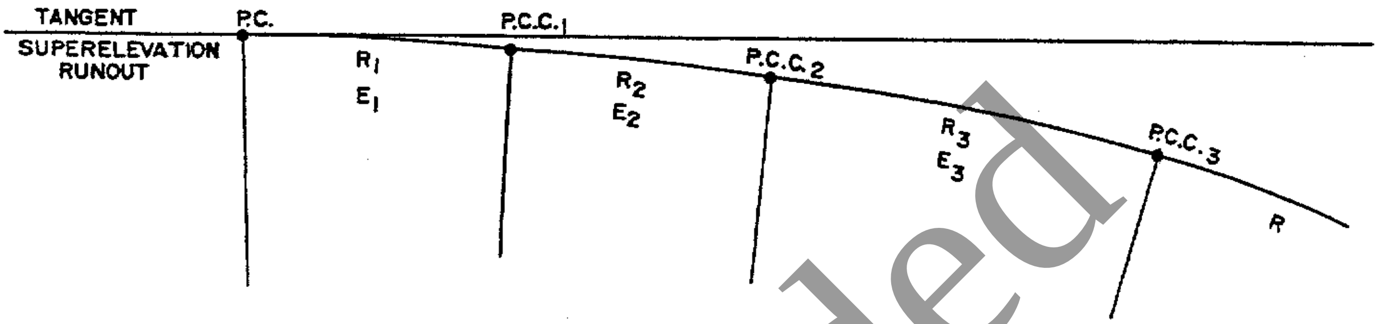
* Based on 6% Maximum Superelevation

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 is 3000 feet in rural areas and 1600 feet in urban areas.

TRANSITION CURVES

FIGURE 4-D



<u>For Design Speeds 30 thru 70 MPH</u>

1. Determine if radii transition is needed for radius R using chart "Transition Not Essential When Radius (R) is greater than".

2. If required, use standard Transition Curves.

3. At P.C.C. 3 hold maximum E for radius R.

4. Using superelevation chart, determine if superelevation is needed for R.

5. If superelevation is needed for R1, use 2/3 maximum superelevation for R1 at P.C.

6. Distribute superelevation evenly between P.C.C. 3 and P.C.

7. Distribute superelevation at the same rate as in Step 6 on tangent up to normal section.

<u>On Existing Roadways Or Where Radii Transitions Can Not Be Provided</u>

1. Determine maximum superelevation needed for radius (R).

2. Use 2/3 maximum superelevation at P.C. and P.T. of curve.

3. Distribute superelevation at a maximum rate of 2%/sec.

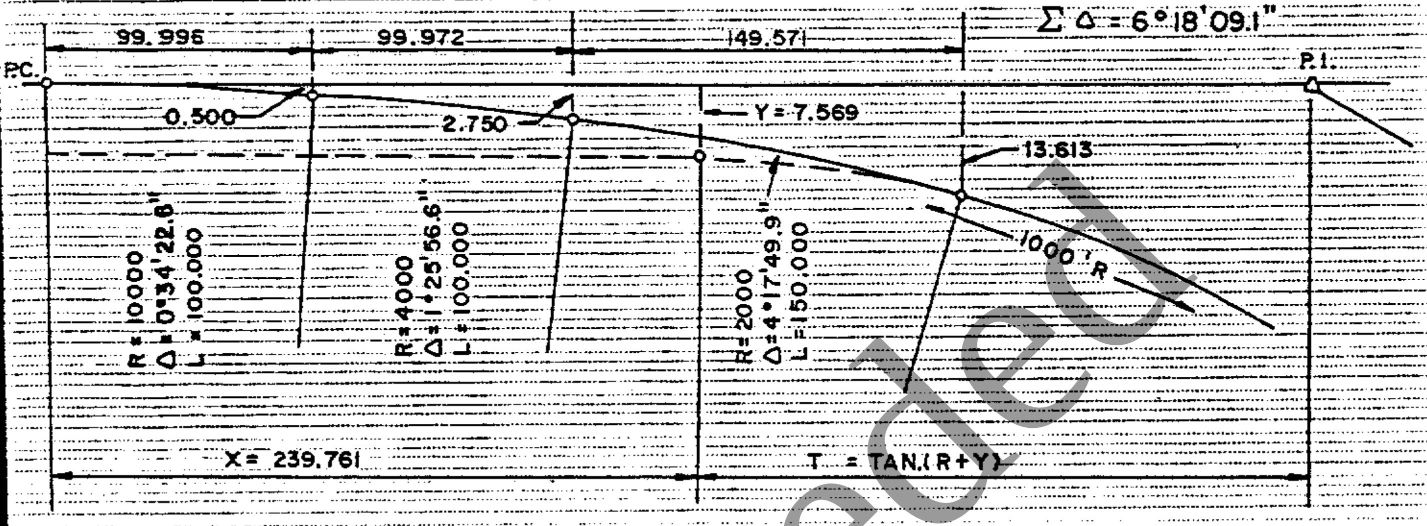
TRANSITION CURVES

FIGURE 4-E

1000' TO 1600' RADIUS

CONSTANTS:

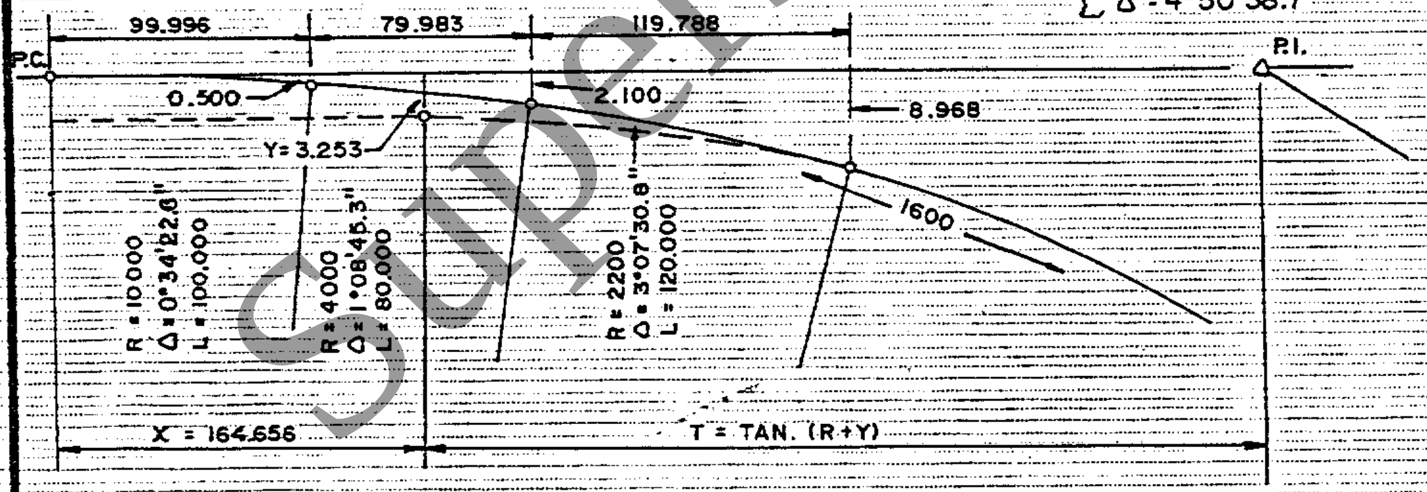
$\Sigma \Delta = 6°18'09.1''$



1600' TO 1800' RADIUS

CONSTANTS:

$\Sigma \Delta = 4°50'38.7''$



NOTE: TO LOCATE TRANSITION P.C.:

(1) FIND X AND Y FOR DESIRED RADIUS
(2) ADD RADIUS R TO Y DISTANCE
(3) FIND T FOR R & Y
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.

X & Y DECREASE BY THE AMOUNT OF THE CONSTANT PER FOOT INCREASED IN RADIUS R.

11/2/83

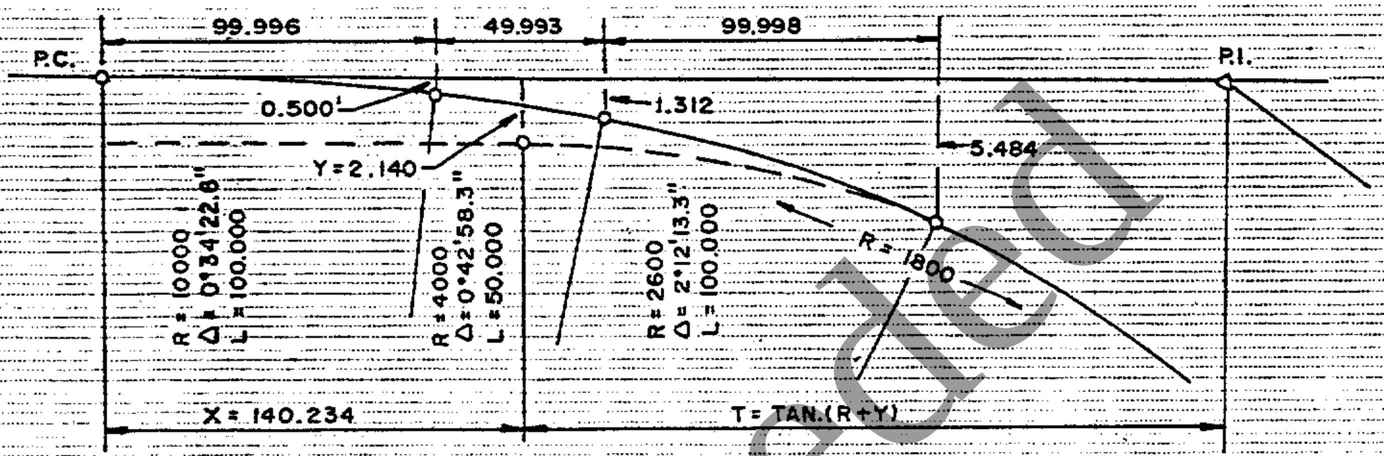
TRANSITION CURVES

FIGURE 4-F

1800' TO 2200' RADIUS

CONSTANTS X → .06092369
Y → .00185753

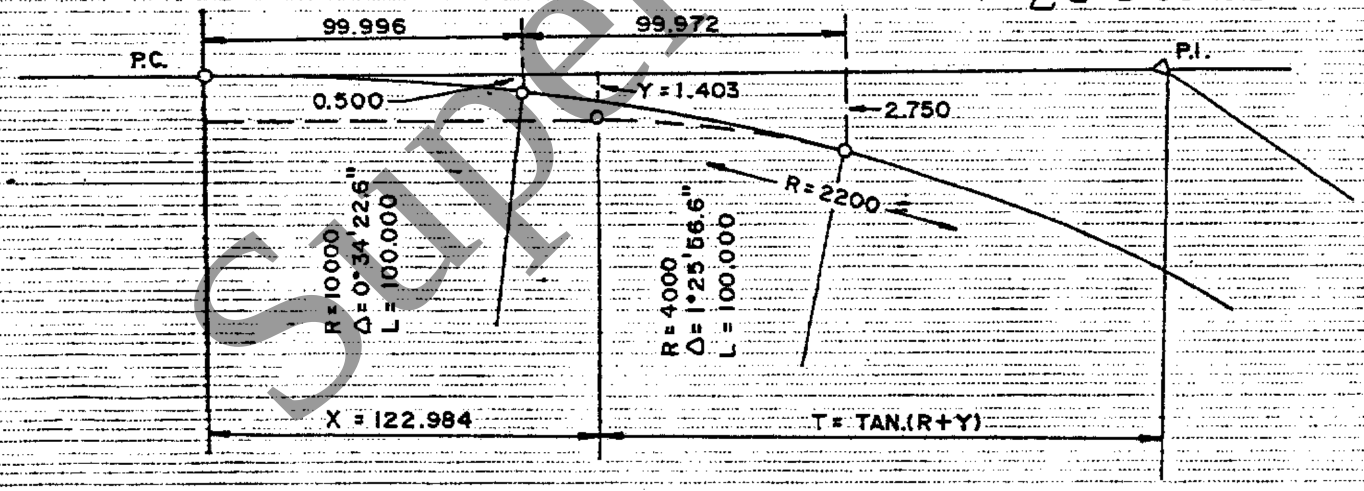
$\sum \Delta = 3°29'34"$



2200' TO 3000' RADIUS

CONSTANTS X → .03499253
Y → .00061242

$\sum \Delta = 2°00'19.2"$



NOTE: TO LOCATE TRANSITION P.C.:

(1) FIND X AND Y FOR DESIRED RADIUS
(2) ADD RADIUS R TO Y DISTANCE
(3) FIND T FOR R & Y
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.
X & Y DECREASE BY THE AMOUNT OF THE CONSTANT PER FOOT INCREASED IN RADIUS R.

11/2/83

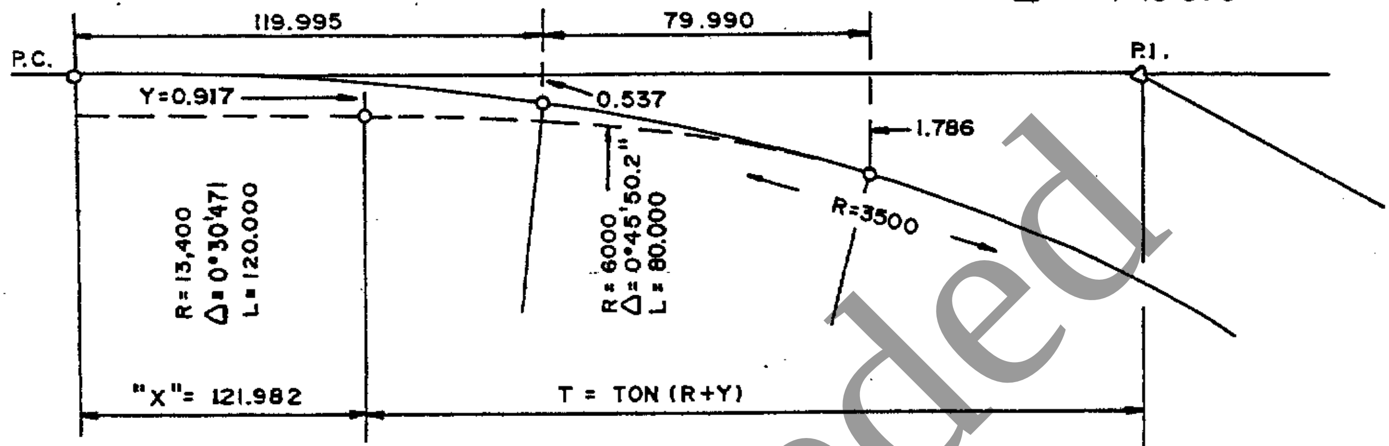
TRANSITION CURVES

FIGURE 4-G

3500' TO 4500' RADIUS

CONSTANTS X → .02228650
Y → .00024837

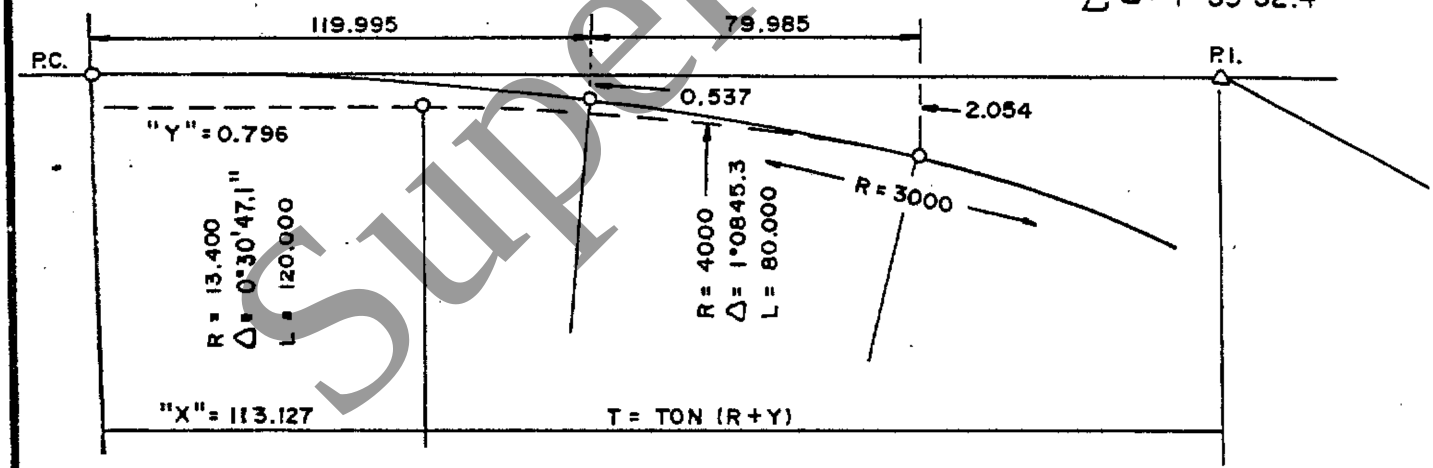
$\sum \Delta = 1°16'373"$



3000' TO 3500' RADIUS

CONSTANTS X → .02895097
Y → .00041917

$\sum \Delta = 1°39'32.4$



NOTE: TO LOCATE TRANSITION P.C.:
(1) FIND X AND Y FOR DESIRED RADIUS
(2) ADD RADIUS R TO Y DISTANCE
(3) FIND T FOR R & Y
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.
X & Y DECREASE BY THE AMOUNT OF THE CONSTANT PER FOOT INCREASED IN RADIUS R.

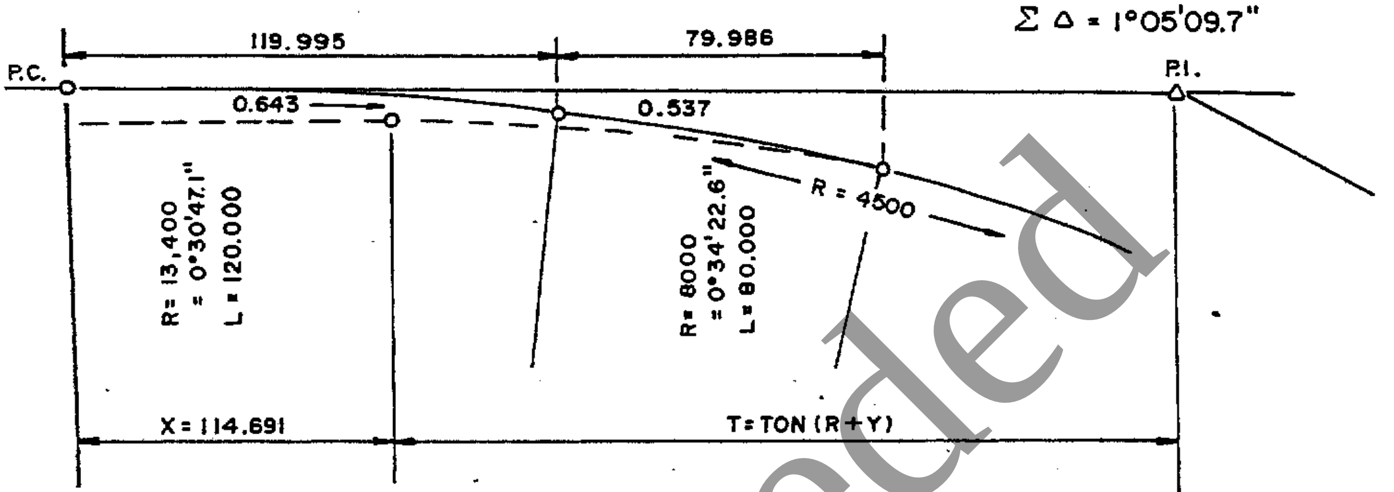
TRANSITION CURVES

FIGURE 4-H

4500' TO 6000' RADIUS

CONSTANTS X→ Y→

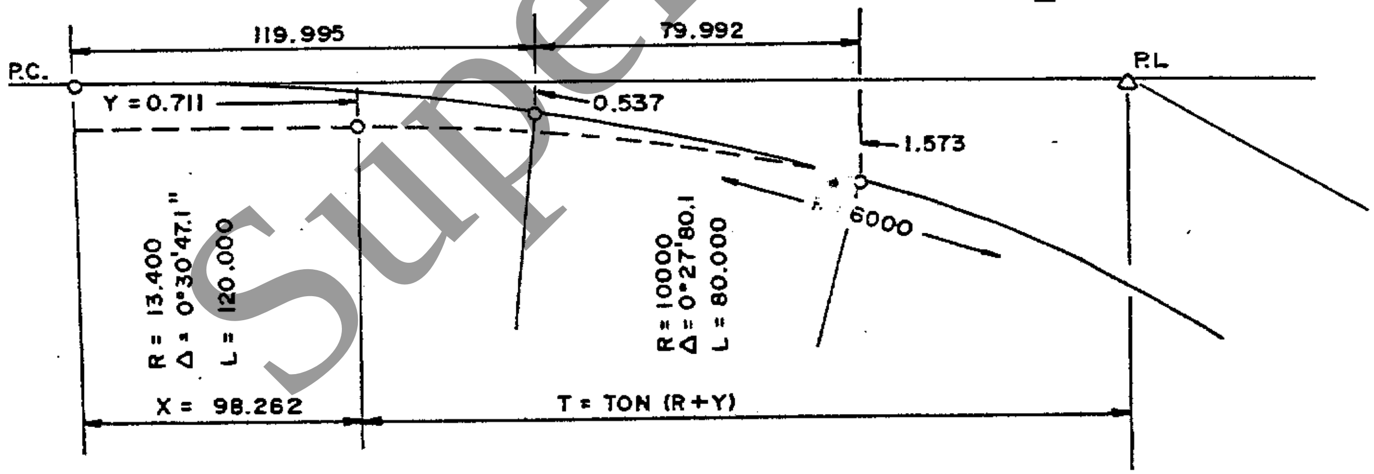
$\Sigma \Delta = 1°05'09.7"$



6000' TO 8000' RADIUS

CONSTANTS X→ Y→

$\Sigma \Delta = 0°58'17.2"$



NOTE: TO LOCATE TRANSITION P.C.:

(1) FIND X AND Y FOR DESIRED RADIUS
(2) ADD RADIUS R TO Y DISTANCE
(3) FIND T FOR R & Y
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.

X & Y DECREASE BY THE AMOUNT OF THE CONSTANT PER FOOT INCREASED IN RADIUS R.

If a glare screen of any kind or a median barrier is contemplated, either initially or utimately, 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.

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 a 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.

4. Stopping sight Distance

Horizontal alignment should afford at least the desirable stopping sight distance for the design speed 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 5 degrees should be 500 feet long, and the minimum length should be increased 100 feet for each 1- degree decrease in the central angle to avoid the appearance of a kink. For central angles smaller than 30 minutes, no curve is required. In no event shall sight distance or other safety considerations be scrificed to meet the above requirement.

6. Compound Curves

On compound curves for arterial highways, the curve treatment shown in figures 4D thru 4H should be used. For compound curves at intersections and ramps, the ratio of the flatter radius to the sharper radius should not exceed 2.0.

7. Reversing Curves

Reversing curves without an intervening tangent are not permitted. The minimum tangent distances are shown in Table 4-3 Severe physical restrictions may dictate the use of curves in opposite directions with a short intervening tangent. In cases where the design speeds are less than 50 mph the preferred length of tangent shall be sufficient to accommodate the superelevation transition.

Table 4-3

Design Speed(mph)	Min. Tangent(ft.)	Min. Des. Tangent(ft.)
50	500	600
60	600	800
70	800	1000

8. 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. Table 4-4 indicates the minimum tangent length between same direction curves. The minimum tangent distance should be exceeded when both curves are visible for some distance ahead.

Table 4-4

Design Speed(mph)	Min. Tangent(ft.)
50	1000
60	1500
70	2500

9. 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 profile 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 profile line must meet sight distance requirements for the design speed classification of the road.

In flat terrain, the elevation of the profile line is often controlled by drainage considerations. In rolling terrain, some undulation in the profile 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 profiles, economic comparisons should be made. For further details, see the AASHTO publication: A Policy on Geometric Design of Rural Highways, 1965.

4-04.2 Position with Respect to Cross Section

The profile line should generally coincide with the axis of rotation for superelevation; Its relation to the cross section should be as follows.

1. Undivided Highways

The profile line should coincide with the highway centerline.

2. Ramps and Freeway to Freeway Connections

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

3. Divided Highways

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

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

b. The two roadways are at different elevations.

4.04.3 Separate Grade Lines

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

They are not normally considered appropriate whre medians are less than 30 feet wide. Exceptions to this may be minor differences between opposing grade lines in special situations.

In addition, 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 obsured-due to excessive differential.

4-04.4 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 6 percent and the minimum 0.5 percent for arterial streets and highways. The desirable maximum for freeways is 3 percent.

4-04.5 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. Figure 4-I and 4-J give the minimum lengths of crest and sag vertical curves for various design speeds and algebraic differences in grade.

Flat vertical curves may develop poor drainage at level sections. Highway drainage must be given more careful consideration when the design speed exceeds 55 mph and 65 mph for crest vertical curves and sag vertical curves respectively. The minimum length of vertical curve should not be less than 3 times the design speed.

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.

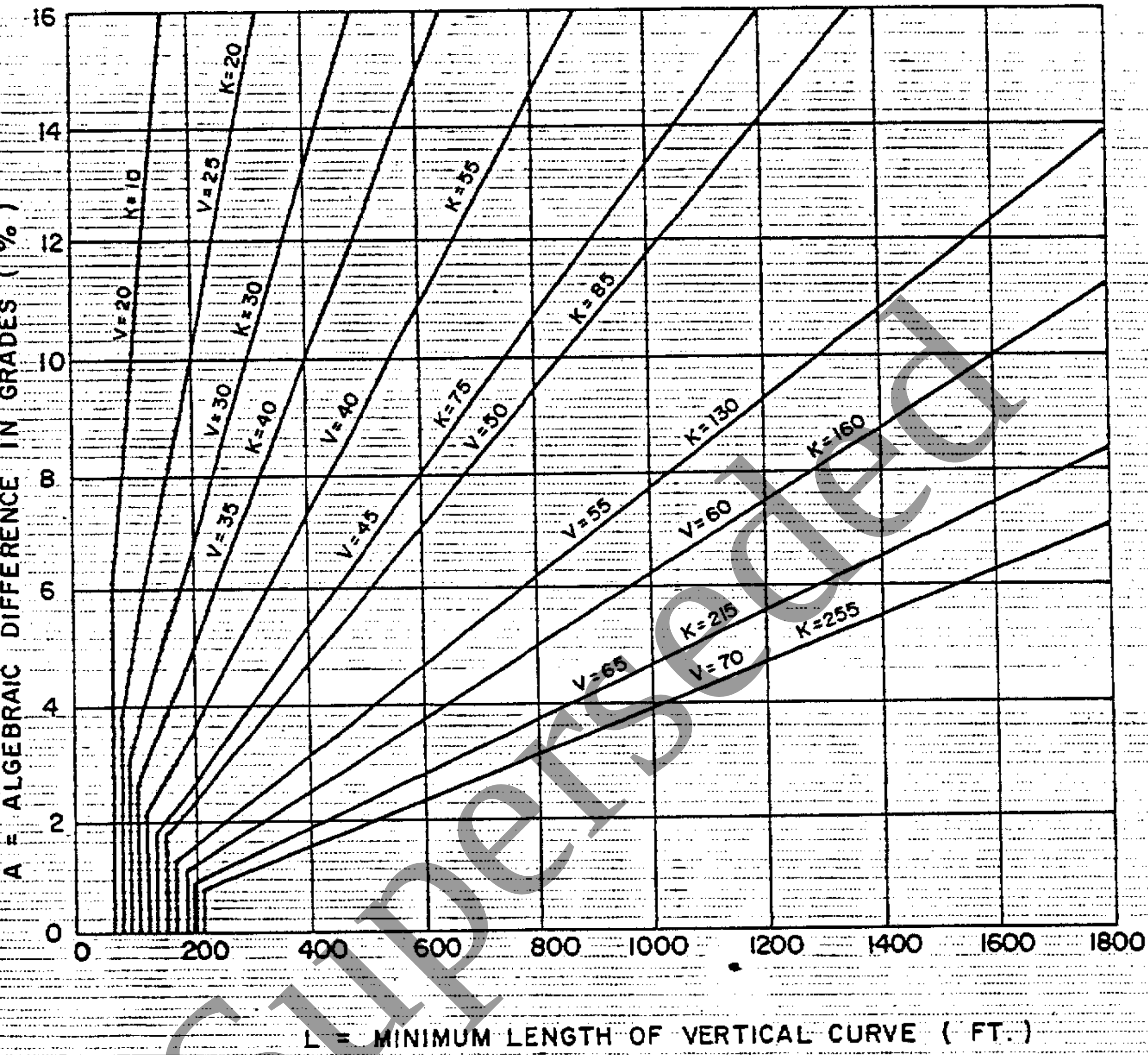
Broken back vertical curves consist of two vertical curves in the same direction, separated by a short grade tangent. A profile with such curvature normally should be avoided.

4-04.6 Heavy Grades

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

DESIGN CONTROLS FOR CREST VERTICAL CURVES

FIGURE 4-I



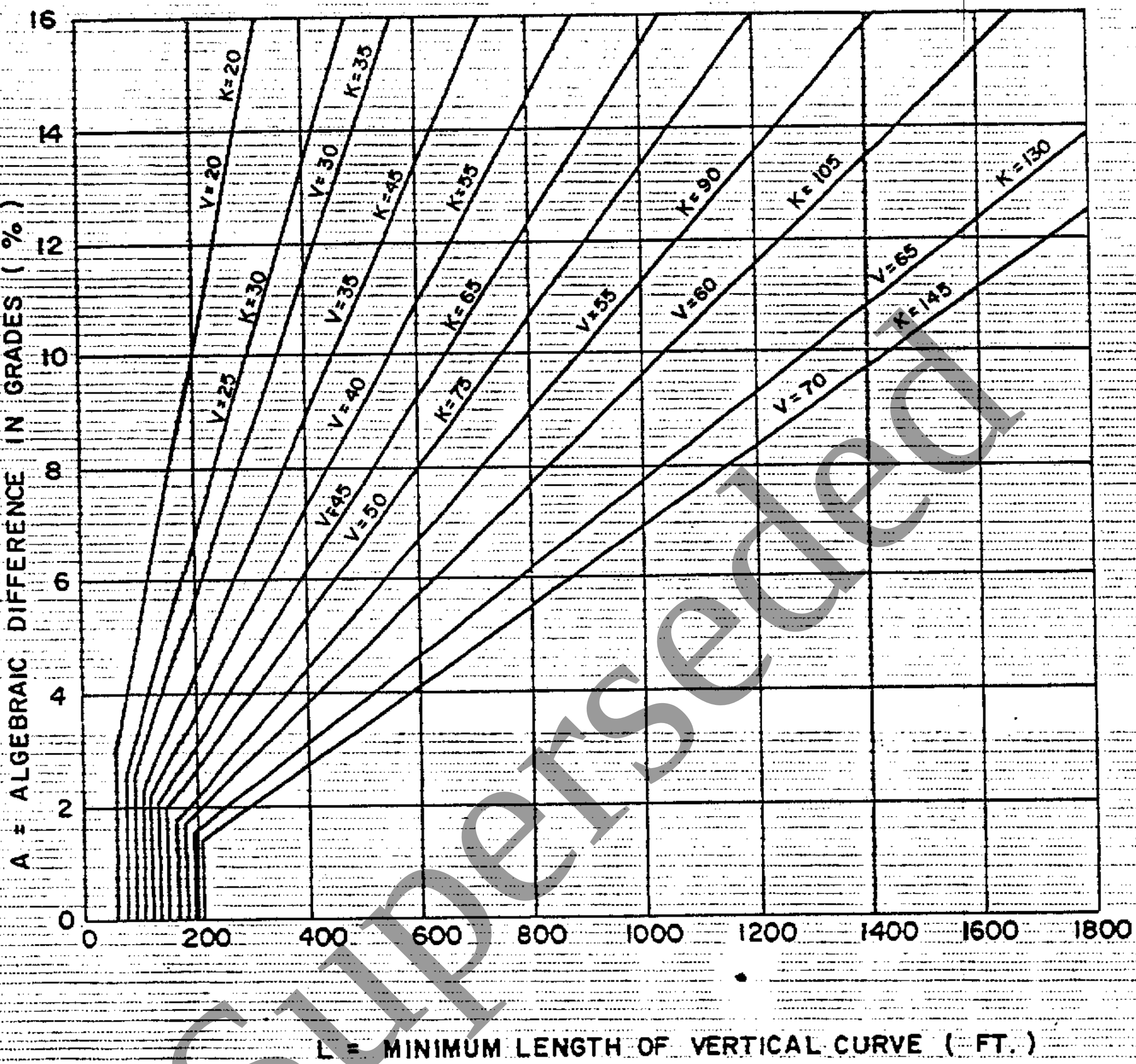
NOTE: DRAINAGE OF THE ROADWAY ON CREST VERTICAL CURVES MUST BE MORE CAREFULLY DESIGNED WHEN THE DESIGN SPEED EXCEEDS 55 MPH.

Superseded

11/3/83

DESIGN CONTROLS FOR SAG VERTICAL CURVES

FIGURE 4-J



NOTE: DRAINAGE OF THE ROADWAY ON SAG VERTICAL CURVES MUST BE MORE CAREFULLY DESIGNED WHEN THE DESIGN SPEED EXCEEDS 65 MPH.

11/3/83

From a truck operation standpoint, a profile with sections of maximum gradient broken by length 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.

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 overlay the horizontal curve.

When vertical and horizontal curves are thus superimposed, the superelevation may cause distortion in the outer pavement edges. Profiles of the pavement edge 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.

4-05 CLIMBING LANE

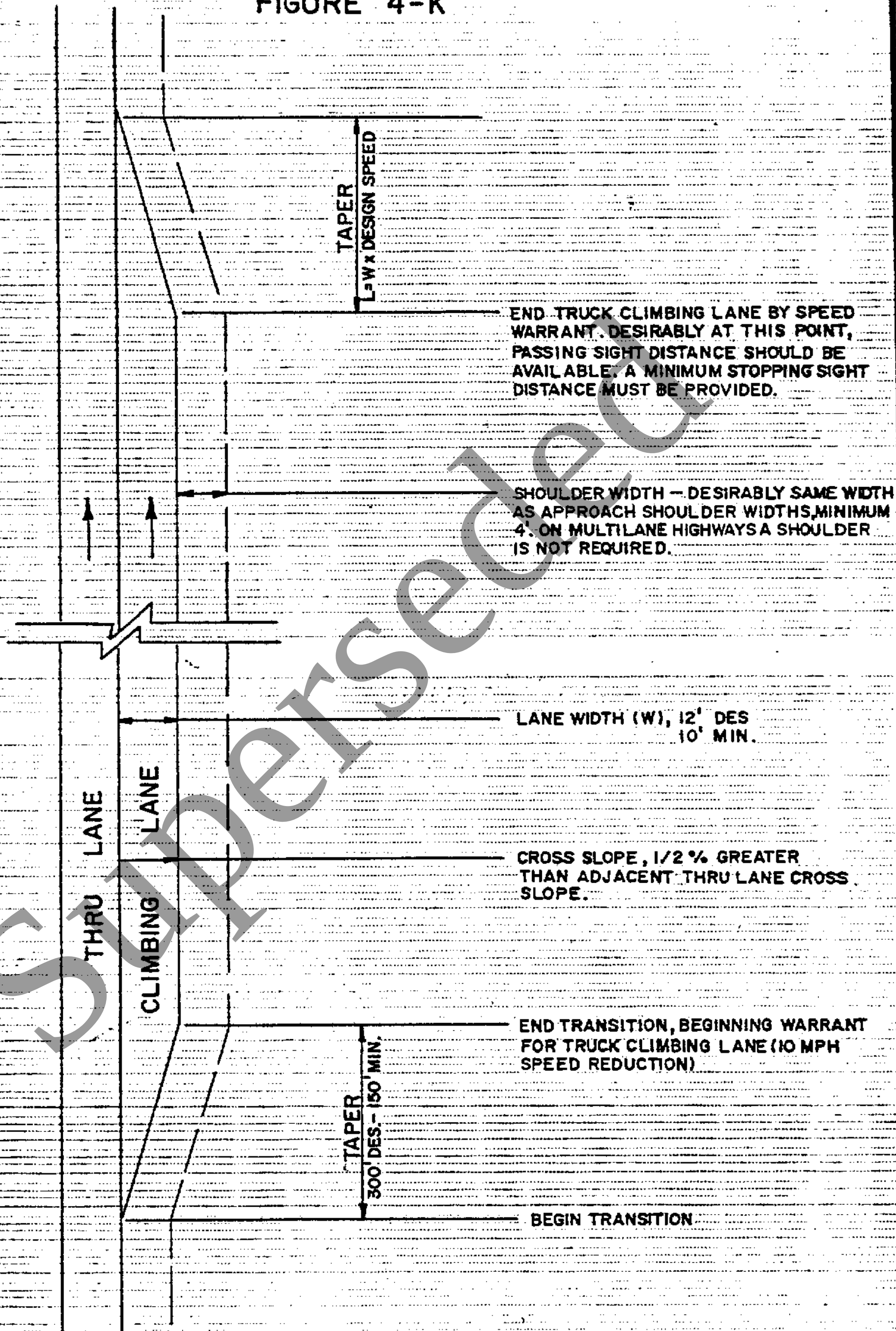
A climbing lane figure 4-K is an auxiliary lane introduced at the beginning of a sustained positive grade for the diversion of slow traffic. Generally, climbing lanes will be provided when the requirements of two warrants, speed reduction and design capacity are satisfied. The requirements of one or the other of these warrants could be waived if, for example, slower moving truck traffic was the major contributing factor causing a high accident rate and could be corrected by addition of a climbing lane.

1. Speed Reduction - The beginning warrant for a truck climbing lane shall be that point where truck operating speed is reduced by 10 MPH. Figure 4-L shall be used for locating this point. The beginning of the climbing lane should be preceded by a tapered section, desirably 300 feet, however a 150 minimum taper may be used.

2. Reduction in Capacity - The capacity warrant for a climbing lane is met when the traffic volume is 120% and 130% of the design capacity for two lane ans multi-lane highways respectively. The capacity, level of service and truck equivalent factors shall be those in the Highway Capacity Manual.

CLIMBING LANE

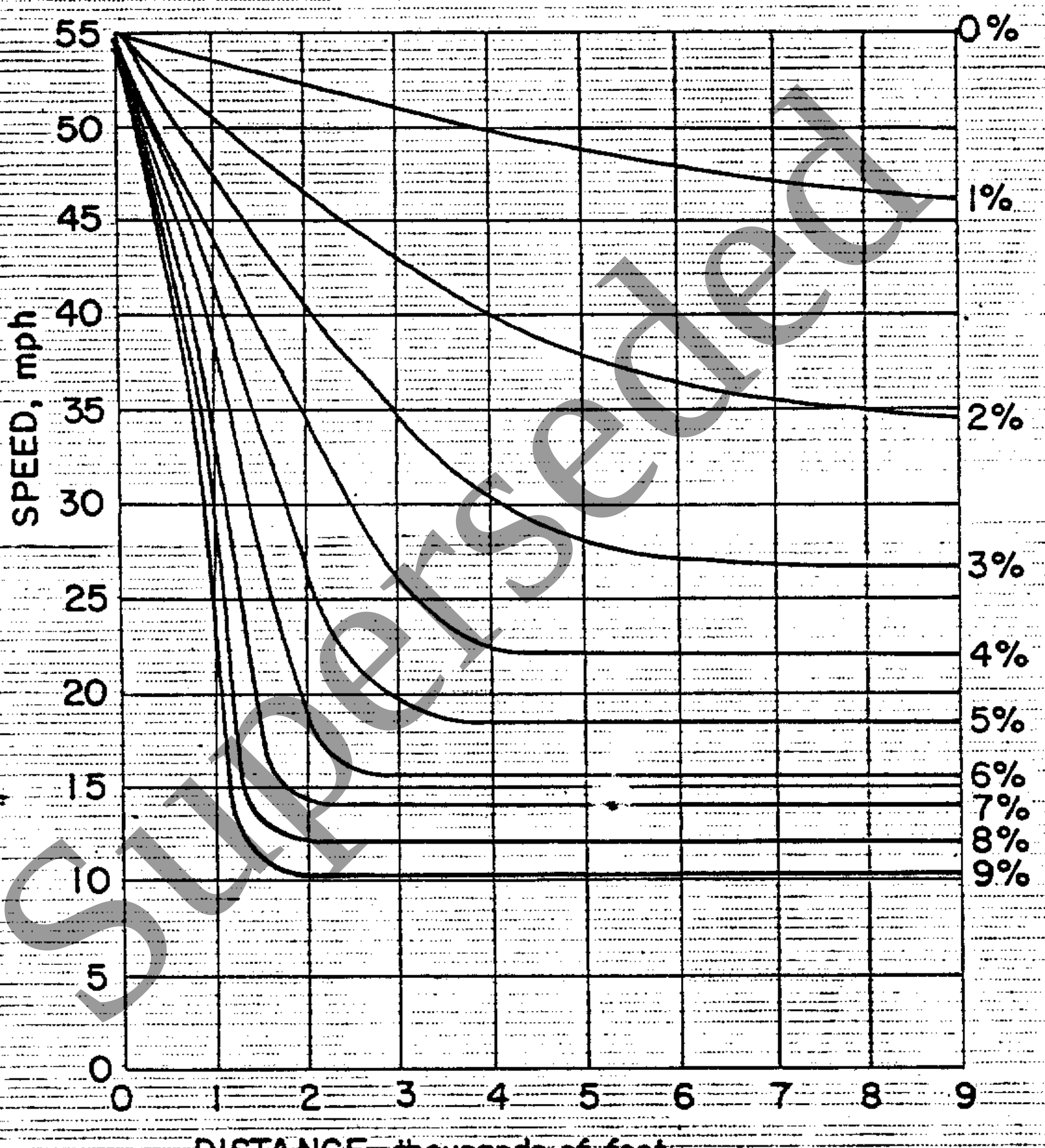
FIGURE 4-K



10/25/83

SPEED-DISTANCE CURVES

FIGURE 4-L

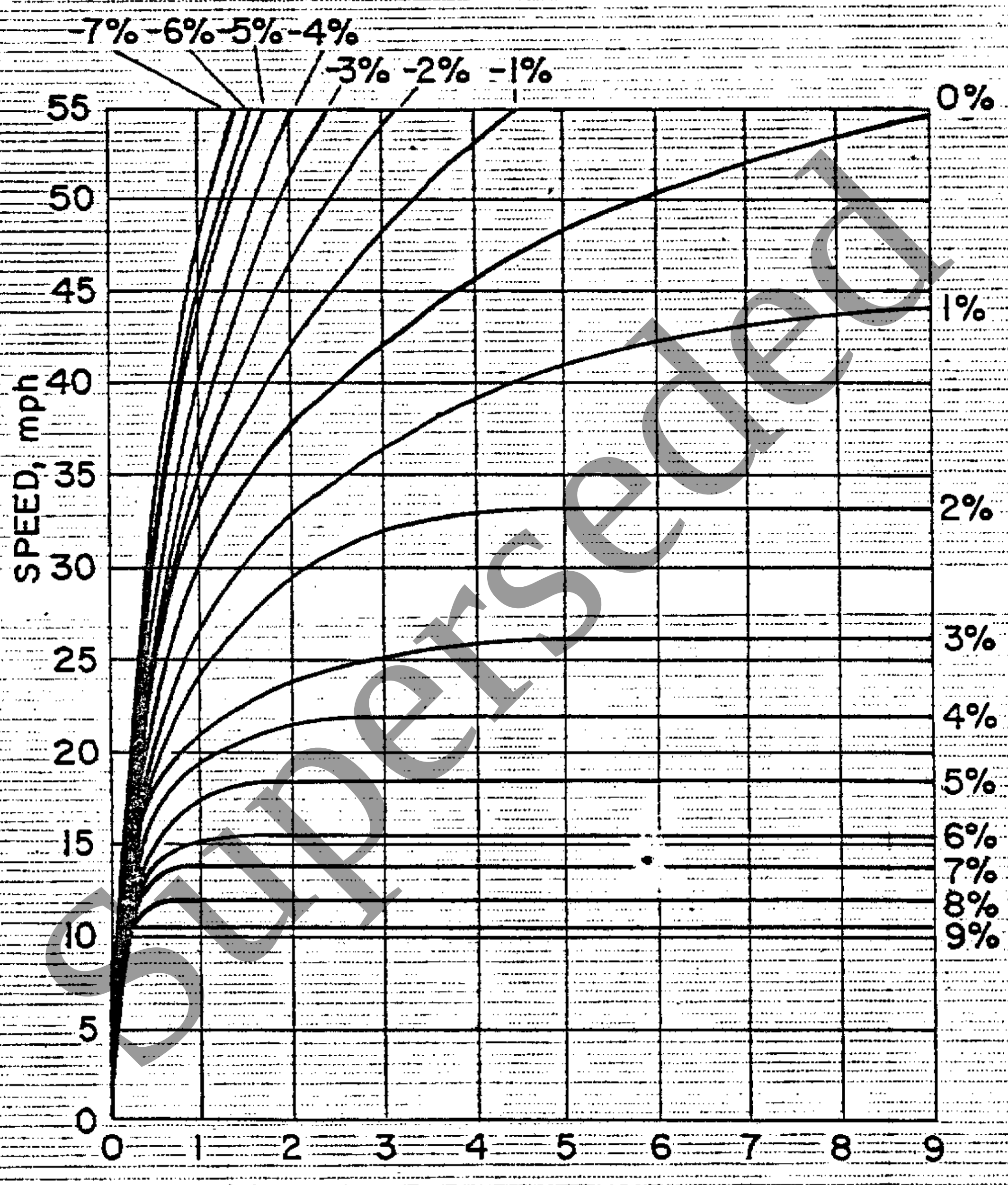


DECELERATION ON UPGRADES

Source: NCHRP 20-7 Review of Vehicle Weight / Horsepower Ratio as Related to Passing Lane Design Criteria. 11/3/83

SPEED-DISTANCE CURVES

FIGURE 4-M



ACCELERATION ON DESCENDING AND ASCENDING GRADES.

TYPICAL HEAVY TRUCK OF 300 LBS./HP.

Source: NCHRP 20-7 Review of Vehicle Weight / Horsepower Ratio as Related to Passing Lane Design Criteria. 11/3/83

The point of ending of a climbing lane shall be determined from Figure 4-M using 10 MPH less than the normal truck operating speed. The ending taper beyond this point shall be the highway design speed times the climbing lane width. Desirably passing sight distance should be available at the point of end of need. As a minimum, stopping sight shall be available.

A distance-speed profile should be developed for the area of a climbing lane. The profile should start at the bottom of the first long downgrade prior to the upgrade being considered for a climbing lané, speeds through long vertical curves can be approximated by using 100' cords.

4-06 PAVEMENT TRANSITION

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

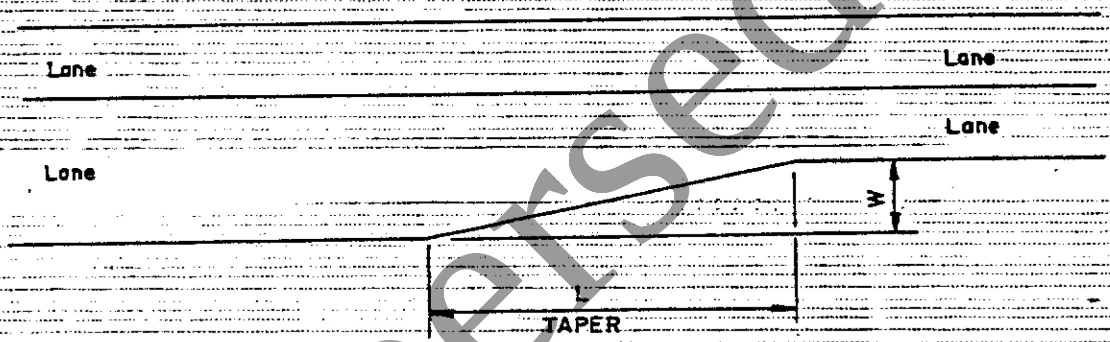
The design should be such that at-grade intersections within the transition are avoided.

Figure 4-N 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-N



FOR DESIGN SPEEDS GREATER THAN 40 MPH, $L = VW$.

FOR DESIGN SPEEDS EQUAL TO OR LESS THAN 40 MPH, $L = \frac{V^2W}{60}$.

V = DESIGN SPEED (MPH)

W = PAVEMENT WIDTH REDUCTION (FT.)

L = TAPER LENGTH (FT.)

11/4/83

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.

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 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 materials 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.

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 affect 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 traveled way and slope downward toward both edges.

The minimum cross slope for concrete pavement and bituminous pavement should be 1.5%. The cross slope shall be uniform across the pavement section, from the high point to the edge of lane. The cross slope in each successive lane should be increased by 0.5%.

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.

PAVEMENT CROSS SLOPE

FIGURE 5-A

EACH PAVEMENT SLOPES TWO WAYS

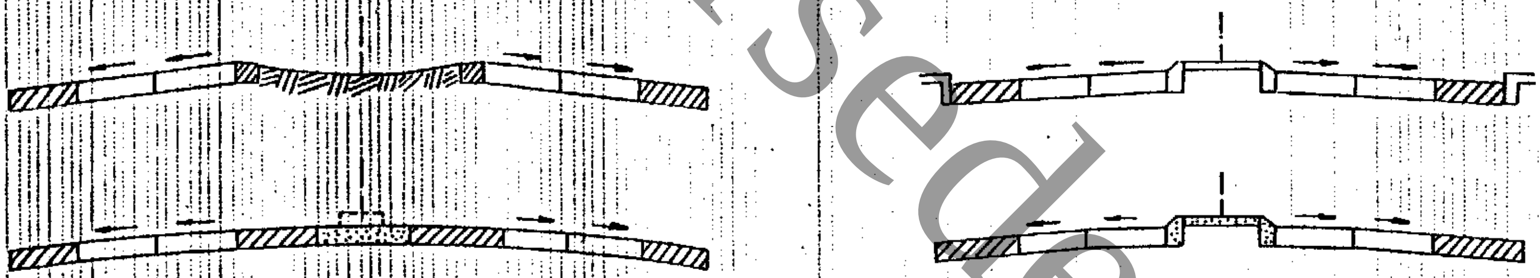


FIGURE 5-B

EACH PAVEMENT SLOPES ONE WAY

11/10/83

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 particulary 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, the drainage is concentrated next to and on 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 relationsips given in Section 4, BASIC GEOMETRIC DESIGN ELEMENTS.

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.

Although lane widths of 12 feet are desirable, there are circumstances that necessitate the use of lanes less than 12 feet wide. In urban areas, the use of 11 foot wide lanes is acceptable, and even 10 foot 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 wide, and desirably at least equal in width to the through lanes when constructed adjacent to a shoulder. When there is no shoulder adjacent to the auxiliary lane, the desirable width is one foot greater than the through lane width.

On freeways the desirable auxiliary lane treatment is to provide a 12 foot lane and 10 foot shoulder, the minimum treatment is to construct a 13 foot lane without a shoulder.

When resurfacing existing highways that have lane widths of 10 feet or less, the existing lanes should be widened to either 11 feet minimum or 12 feet desirable.

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.

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.

5-04.2 Width of Shoulders

Desirably, a vehicle stopped on the right shoulder should clear the pavement edge by at least 1 foot, preferably by 2 feet. This preference has led to the adoption of 10 feet 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 8 feet should be considered for the lowest type highway. Heavily traveled and high-speed highways, and those carrying large numbers of trucks, should have usable shoulders at least 10 feet and preferably 12 feet wide. Shoulder widths for specific types of highways are enumerated as parts of the typical sections illustrated at the end of this section.

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 when 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.

Left shoulders are preferred on all divided highways. The desirable median shoulder width on 4-lane and 6 to 8 lane highways is 5 feet and 10 feet respectively. The minimum shoulders widths are 3 feet and 5 feet respectively. On 6 to 8 - lane nonaccess control highways where the right-of-way is restricted, the minimum left shoulder width may be reduced to 3 feet.

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. 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.

On resurfacing projects where wider lanes are to be provided, the outside shoulder width may be reduced to a minimum of 8 feet.

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. The minimum cross slope should not be less than 4 percent to minimize ponding on the roadway.

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.

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 ROADSIDE OR BORDER

5-05.1 General

The area between the roadway and the highway right-of-way is referred to as the roadside or border. The term "roadside" generally applies to freeways and the term "border" applies to land service highways.

5-05.2 Width

The minimum right-of-way width on rural and urban freeways is 300 ft. and 150 respectively. Depending upon the median, travelled way and shoulder widths, the roadside width is in the range of 70 ft. for rural freeways and 25 ft. for urban freeways.

The desirable border width on land service highways is 15 ft. Where right-of-way acquisition costs or terrain features make the 15 ft. border width impractical a 10 ft. border may be used. Preferably the width of the border should be sufficiently wide to permit the placement of utilities poles and all fixed obstructions beyond the clear zone area. Normally, an additional 5 feet should be added to the clear zone distance to provide the necessary placement of utilities within the highway right-of-way yet beyond the clear zone recovery area.

See Section 8 for the required clear zone distance for various design speeds.

The border width on existing highways may be reduced to 8 feet to accommodate the widening of lanes and/or shoulders.

5-06 CURBS

5-06.1 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 the other acceptable means.

5-06.2 Types of Curbs

The two general classes of curbs are vertical curbs and sloping curbs. Each may be designed as a separate unit, or integrally with the pavement. Vertical and sloping curbs may be designed with a gutter to form a combination curb and gutter section.

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

Generally, vertical 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 our concrete barrier curb. A vertical curb may be used on bridges on urban streets, with a curb height the same height as the approach roadway curb. 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 travel 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 portions within 4 feet of the curb to minimize adverse driving effects. See Section 10 for the proper spacing of inlets.

5-06.3 Placement of Curbs

Vertical 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. Curbs introduced intermittently along streets should be offset 3 feet from the edge of pavement; where continuous, at least 1 foot.

5-06.4 Vertical Curb Height

When curb is warranted on local streets, land service highways, freeways and ramps, the height of the vertical curb face shall be no greater than six (6) inches.

All curb on bridge decks and curbs in front of retaining walls shall be constructed with a vertical curb face no greater than six (6) inches.

The vertical curb face may be increased when used in continuous medians or where required due to other special conditions but only if specifically approved by the Chief Engineer, Design.

Vertical curbs and safety walks may be desirable along the faces of long walls and tunnels, particularly if full shoulders are not provided.

Sloping curbs are designed so that vehicles can cross them readily when required. They are low with flat sloping faces. Mountable curbs can be used at median edges to outline channelizing islands in intersections areas, or sometimes at the outer edge of the shoulder.

When curbs are used for drainage purposes in conjunction with guiderail they should be installed directly under or 9 inches in front of the guiderail. Section 8, TRAFFIC BARRIERS, Guidelines for Guiderail Design and Median Barriers, discusses curb placement in conjunction with traffic barriers.

When resurfacing adjacent to curb, the curb should not be removed unless it is deteriorated or the curb face will be reduced to less than 3 inches. A curb face less than 3 inches is permissible, provided drainage calculations indicate the depth of flow in the gutter does not exceed the remaining curb reveal.

5-07 SIDEWALKS

5-07.1 General

Construction of new sidewalk will normally be the responsibility of the property owner fronting on the highway. The Department may construct sidewalks when required by local ordinance or to eliminate a known pedestrian vehicle accident prone location.

Sidewalk will be replaced when the existing sidewalk has been distrubed or removed by an improvement project undertaken by the Department.

Sidewalk widths are usually 4 feet wide and separated from the roadway by a planting strip 3 feet in width.

5-07.2 Curb Ramps for the Handicapped

Curb ramps for the physically handicapped shall be provided where curb or sidewalk is being constructed or reconstructed at intersections. However, where there is no sidewalk but curb is being constructed or reconstructed, it is not necessary to provide depressed curb for future ramps. Handicapped ramps may be placed at midblock crosswalks.

Figures 5-Q and 5-R illustrate the design standards for curb ramps for the physically handicapped.

5-08 DRIVEWAYS

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

Design requirements for driveways are included in the Department's publication, "Control of Access Driveways".

To determine the adequacy of the sight distance at driveways see SECTION 6 for sight distance at intersections.

5-09 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; a recovery area for out-of-control vehicles; a stopping area in case of emergencies; 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 medians in an urban area is that they 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 to a desirable dimension of 80 feet 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 betweem 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, 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 oprations.

Flush median 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 including emergency median openings on land service highways and freeways is discussed in SECTION 6, AT-GRADE INTERSECTIONS.

5-10 STANDARD TYPICAL SECTIONS

Typical sections should be developed to provide safe and aesthetically pleasing highway sections within reasonable economic limitations.

The typical sections shown in the plans should represent conditions that are "typical" or representative of the project. It is not necessary to show a separate typical section to ~~the~~ delineate relatively minor variations from the basic typical. The most common or predominant typical section on the project should be shown first in the plan sheets followed by sections of lesser significance.

Figures 5-C through 5-J inclusive illustrate the various control dimensions for single lane and multi-lane highways.

5-11 BRIDGES AND STRUCTURES

5-11.~~01~~ .1 General

Designers should make every effort during the early design phase to eliminate or minimize certain features such as, horizontal curves, vertical curves, variable horizontal widths and cross slopes, on bridge decks. Locating these features off the structure simplifies construction, is more economical and reduces future maintenance requirements.

For further information the designer should review Subsection 1.5.2. Geometrics on Bridges in the NJDOT Design Manual - Bridges and Structures.

5-11.~~02~~ .2 Lateral Clearances

It is desirable that the clear width on the bridge be as wide as the approach pavement plus shoulders.

On underpasses the desirable treatment is to maintain the entire roadway section including median, pavements, shoulders and clear roadside areas through the structure without change.

Minimum lateral clearances are illustrated in figures 5-K through 5-P inclusive.

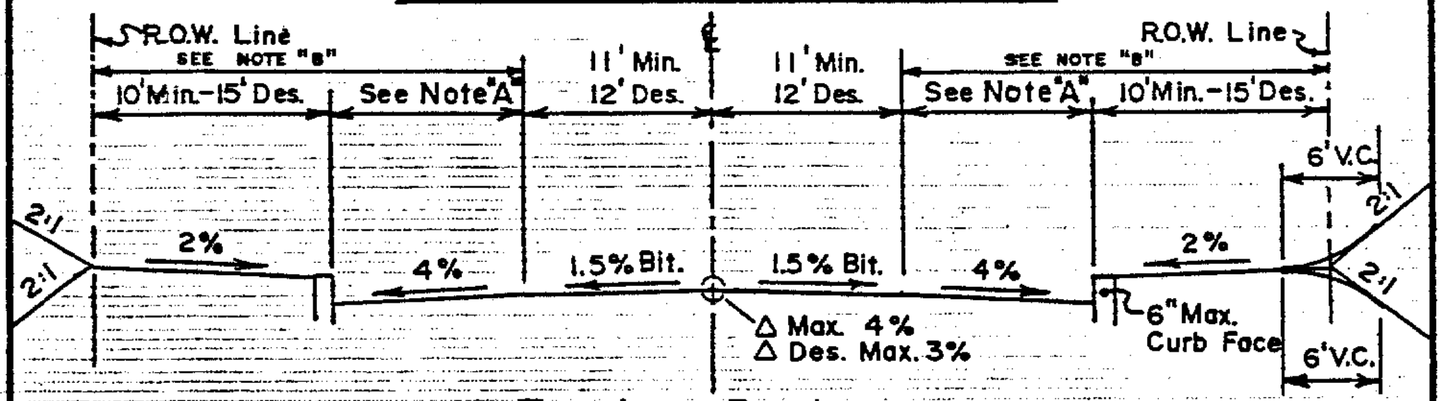
On divided highways where the median width is less than 30 feet consideration should be given to eliminating the parapets and decking the area between the structures.

5-11.03 Vertical Clearance

Vertical clearances for bridges and structures shall be in accordance with Subsection 1.3.1 of the NJDOT Design Manual - Bridges and Structures.

Bridges and Structures Design should be notified of all changes in bridge clearances.

LAND-SERVICE HIGHWAYS



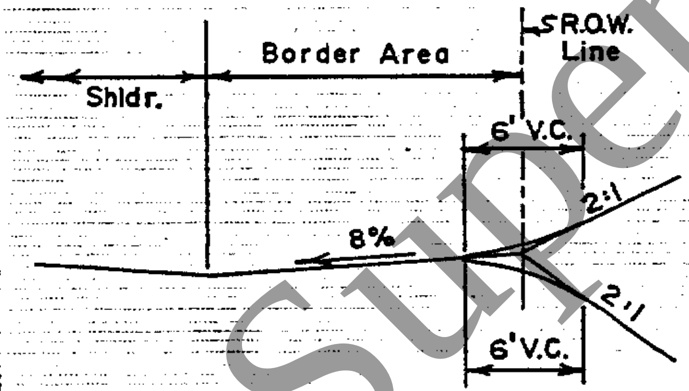
Two-Lane Roadway

Note "A": Shoulder width shall be 8' Absolute Minimum or 10' Minimum Desirable. Shoulder width may be increased to 12' maximum when a large volume of trucks are anticipated (250 DHV), when turning volumes are high or dualization is anticipated.

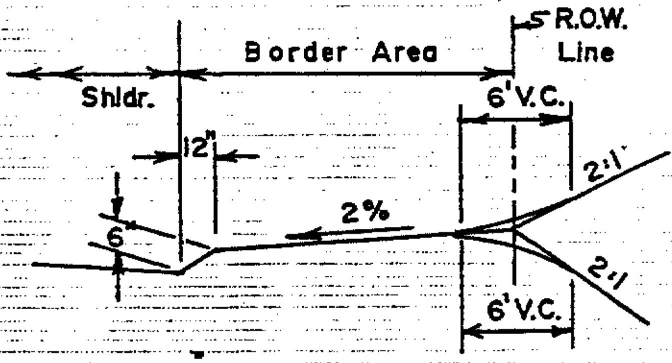
Note "B": Desirably the following clearances from the edge of thru lane to the right-of-way line for the corresponding design speeds should be provided: 60 mph, 35 ft.; 55 mph, 30 ft.; 50 mph or less, 25 ft.

Note "C": Curb Section may be used with or without sidewalk. Curb Section shall be used in built-up areas, where pedestrian traffic is anticipated or where necessary for drainage.

NOTE:
ALL UTILITY POLES SHALL BE LOCATED AS CLOSE TO THE RIGHT-OF-WAY LINE AS POSSIBLE

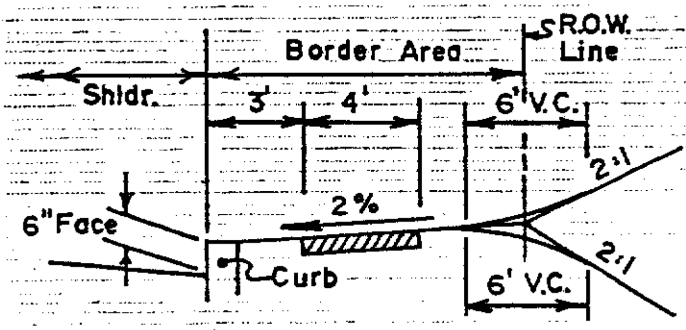


Sidewalk Area – Where pedestrians are not anticipated.



Sidewalk Area – Berm Section

Where pedestrians are to be accommodated; future sidewalk.



Sidewalk Area – Curb Section

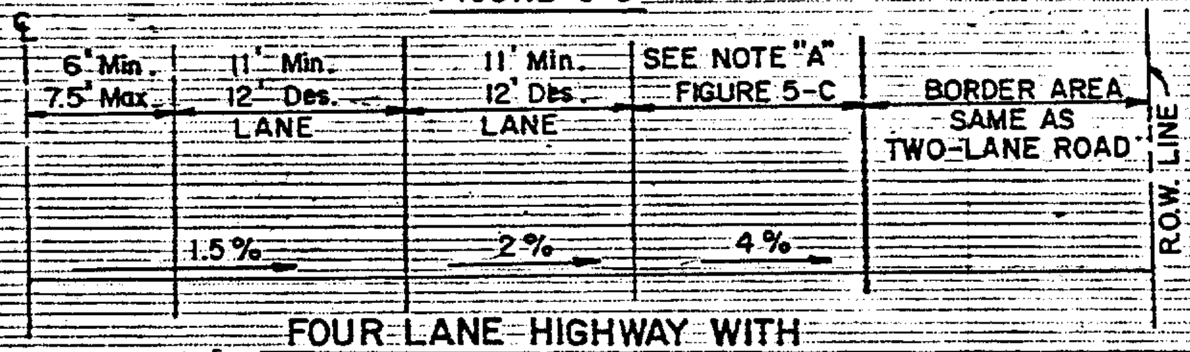
See Note "C"

FIG. 5-C

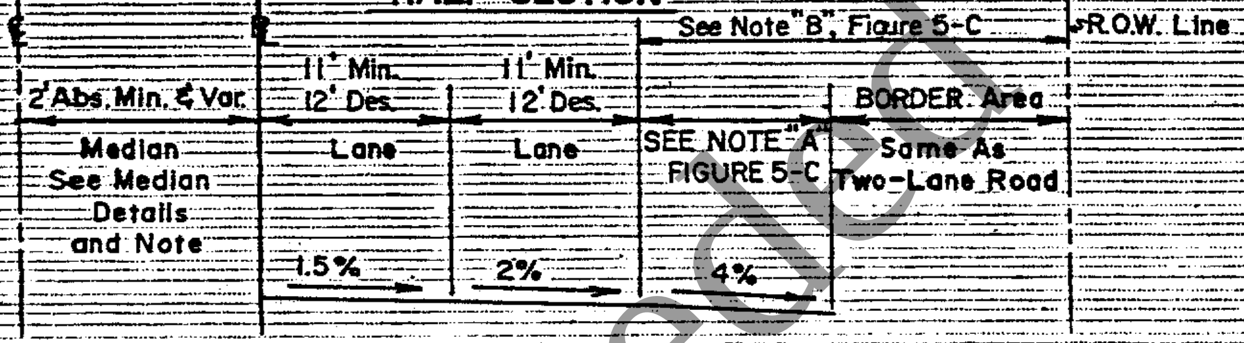
10/24/83

LAND-SERVICE HIGHWAYS

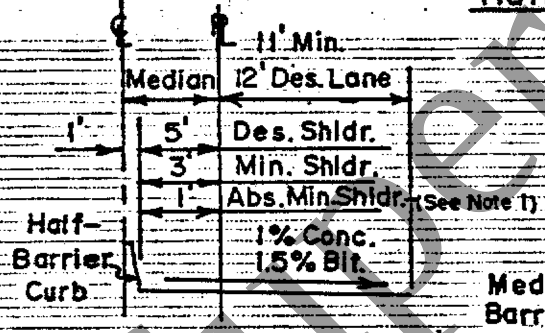
FIGURE 5-D



FOUR LANE HIGHWAY WITH TWO-WAY LEFT TURN LANE HALF SECTION

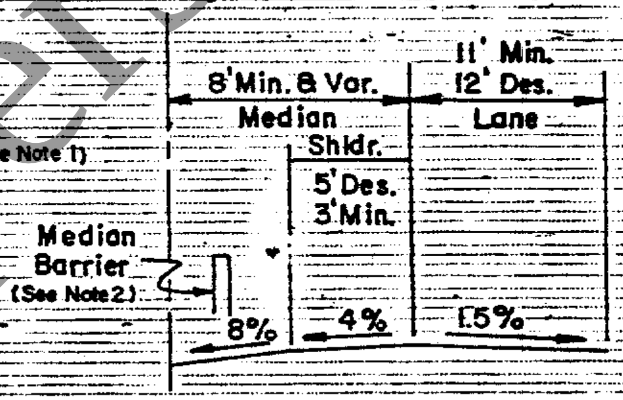


Four Lane-Divided Highway Half Section



Median Half-Section

No drainage in median.



Median Half Section

Drainage in median

NOTE 1: APPLICABLE TO EXISTING HIGHWAYS ONLY. USEAGE MUST HAVE PRIOR APPROVAL OF CHIEF ENGINEER.

NOTE 2: MEDIAN BARRIER MAY BE LOCATED EITHER SIDE OF LOW POINT.

10/25/83

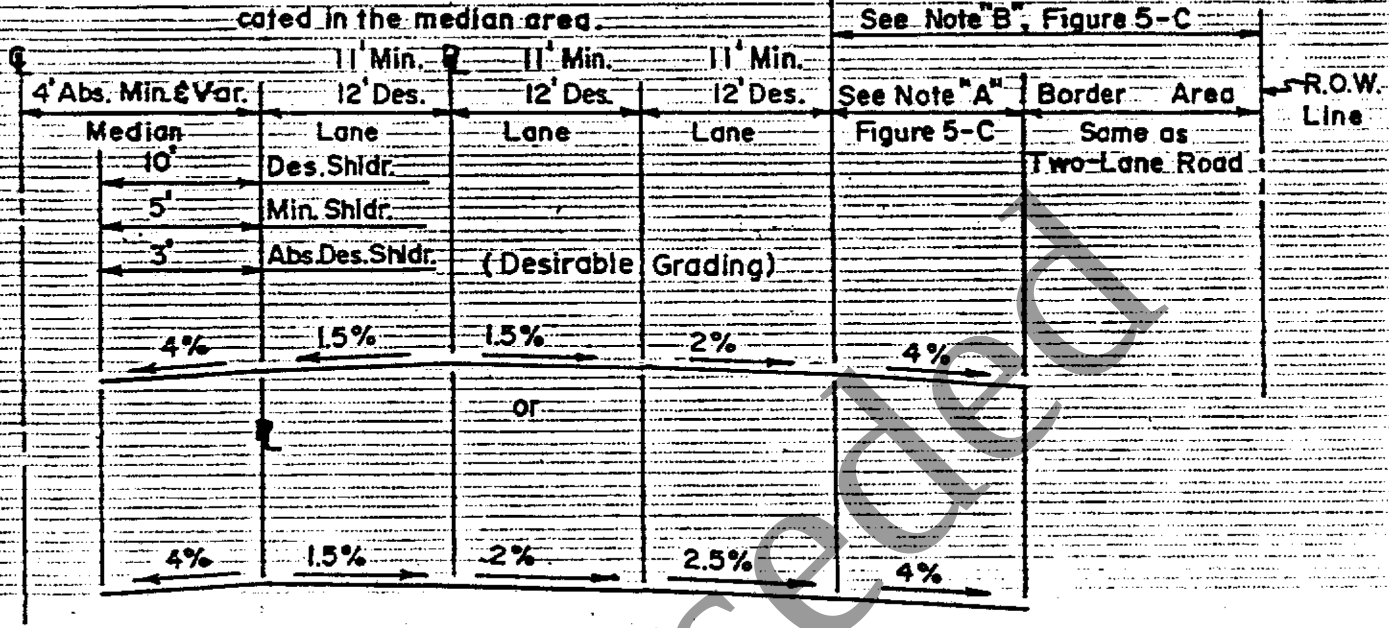
LAND-SERVICE HIGHWAYS

FIGURE 5-E

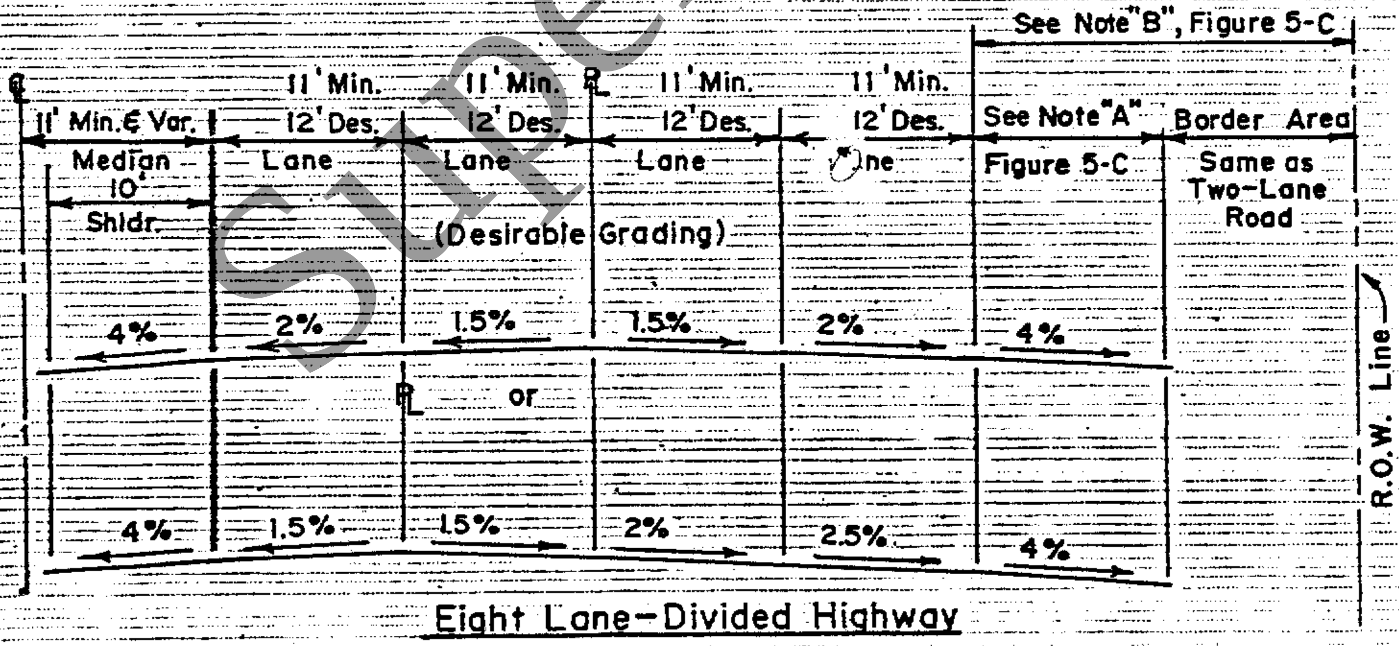
Note: 1) Median Barrier will be selected as per "Guide for Selecting, Locating and Designing Traffic Barriers", AASHTO 1977.

2) The kind of Median Barrier to be used is to be determined by the Design Engineer.

3) For medians over 30', Median Barrier use is optional. Drainage is to be located in the median area.



Six Lane-Divided Highway
Half-Section

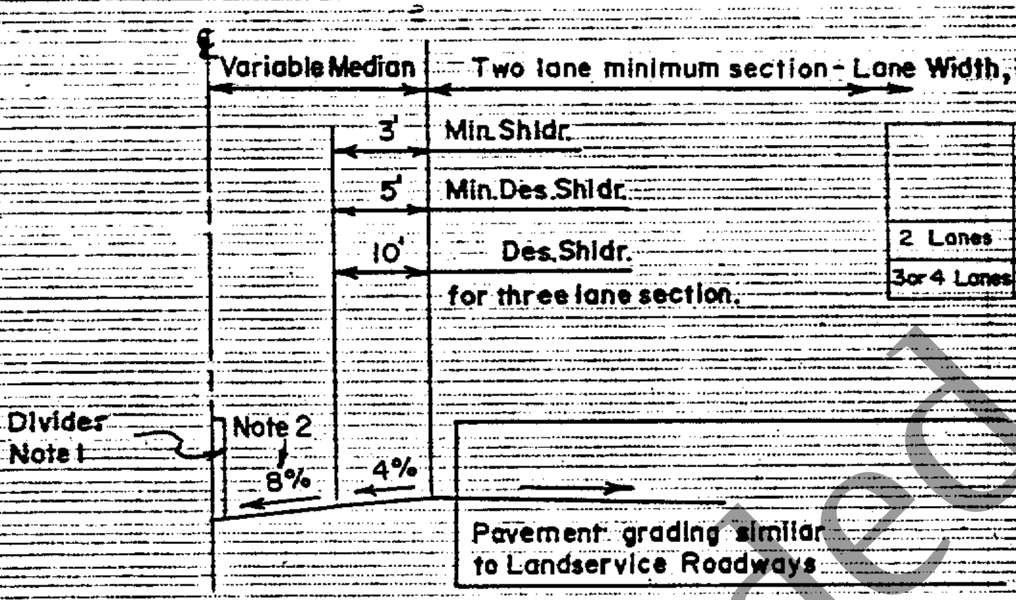


Eight Lane-Divided Highway
Half-Section

August 1979

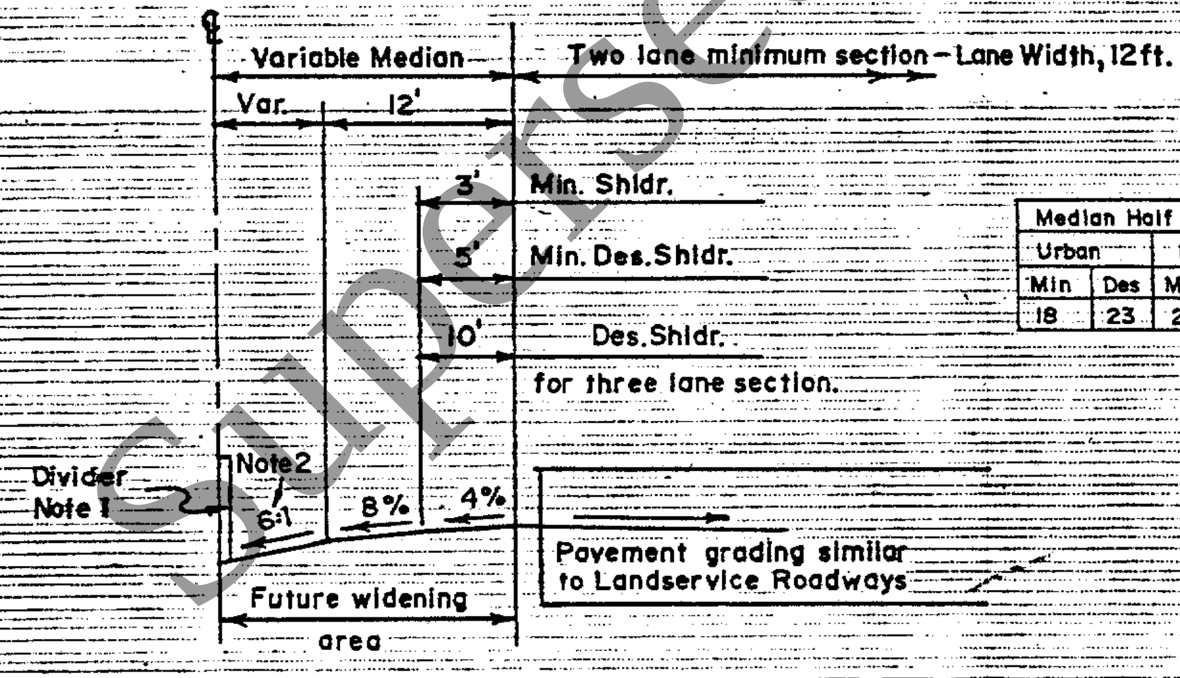
FREEWAY SECTIONS

FIGURE 5-F



	Median Half Width			
	Urban		Rural	
	Min	Des	Min	Des
2 Lanes	4	6	6	30
3 or 4 Lanes	6	11	11	30

Half Section (No Expansion)



Median Half Width			
Urban		Rural	
Min	Des	Min	Des
18	23	23	42

Half Section (Future Expansion)

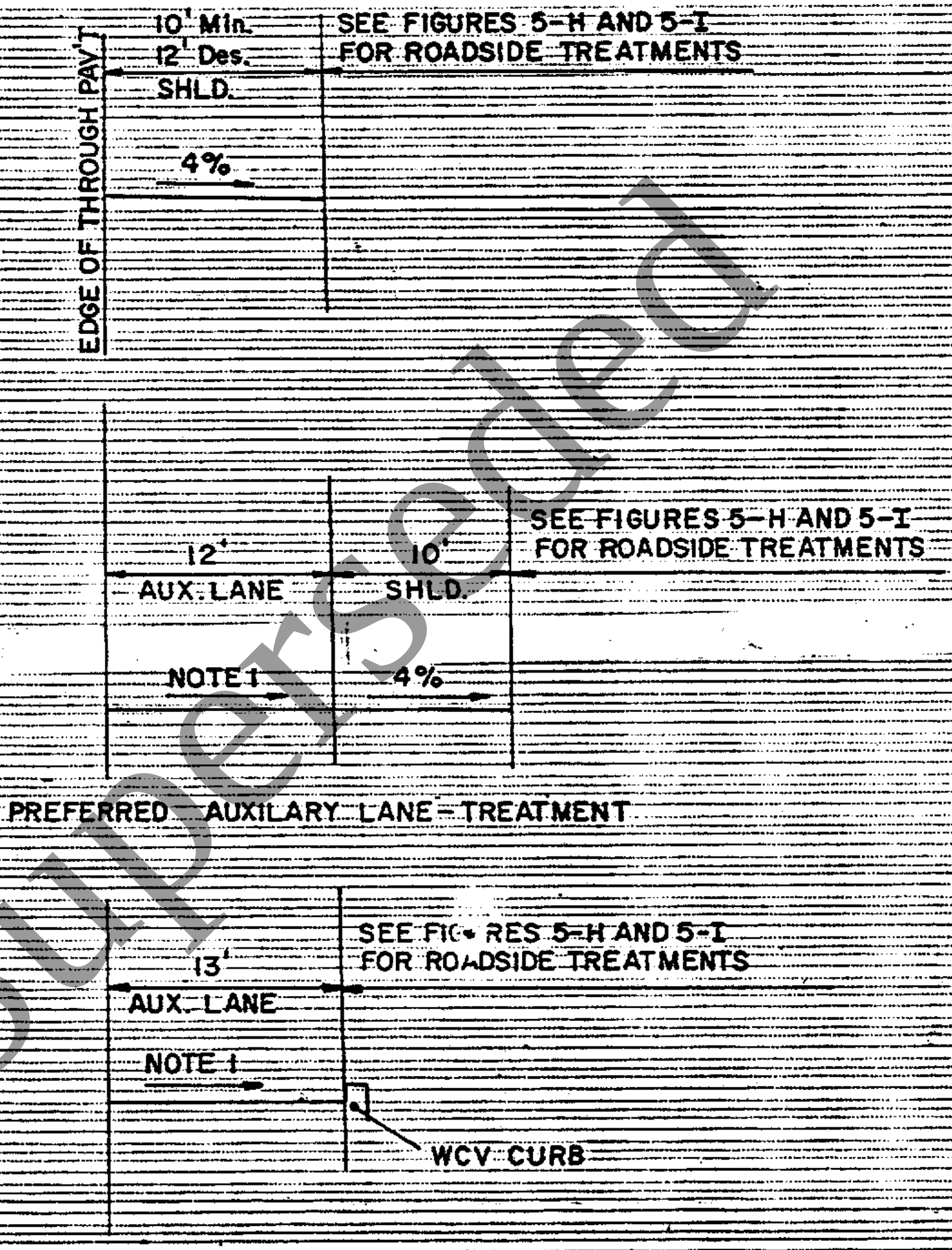
NOTE 1 — FOR MEDIAN BARRIER WARRANTS SEE SECTION 8 FIGURE 8-V.

NOTE 2 : MAXIMUM SIDESLOPE ADJACENT TO A MEDIAN BARRIER IS 10:1.

11/9/83

FREEWAY SECTION

FIGURE 5-G

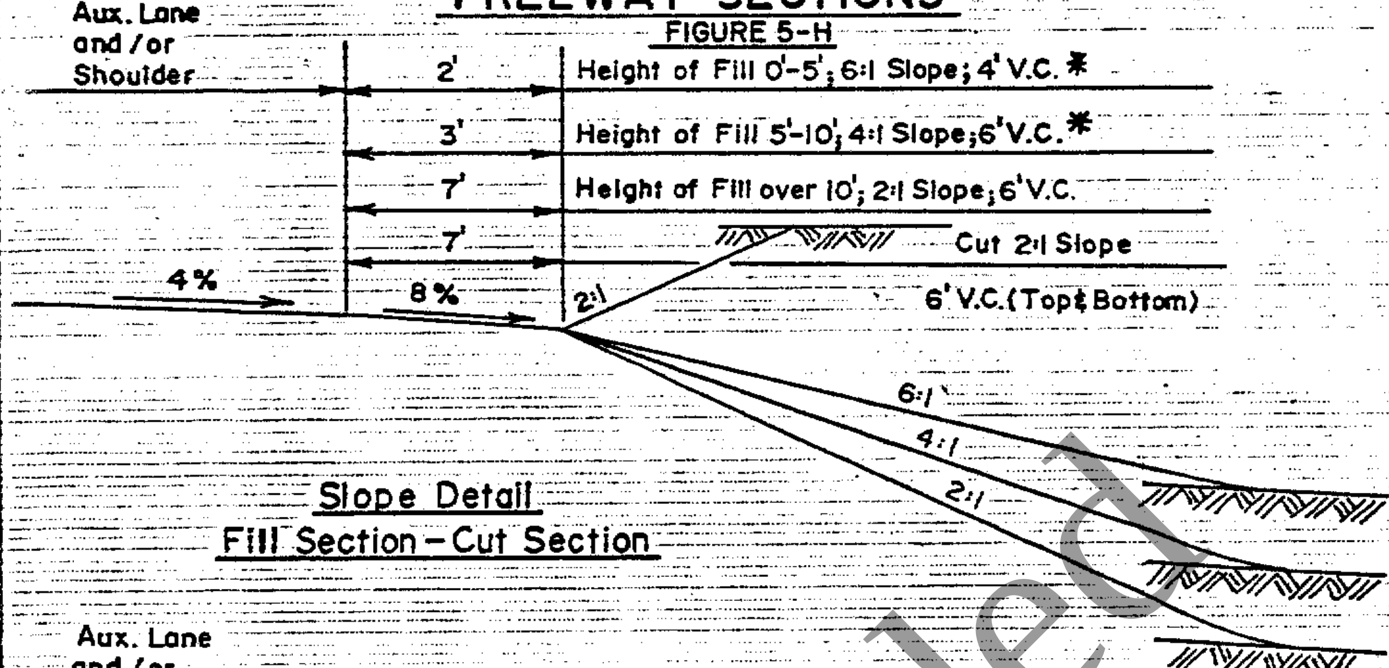


NOTE 1. CROSS SLOPE SHOULD BE 1/2% GREATER THAN ADJACENT THROUGH LANE. MAXIMUM CROSS SLOPE SHOULD NOT EXCEED 2.5%.

11/4/83

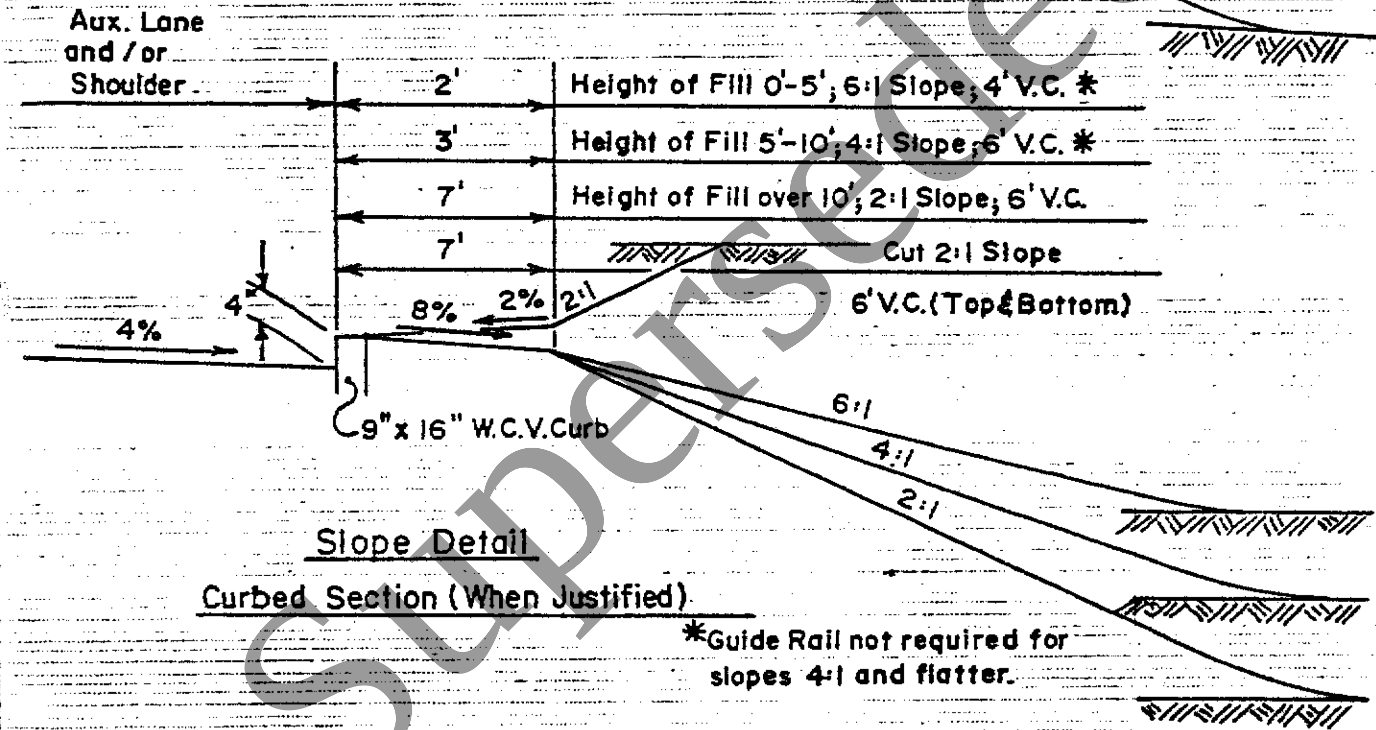
FREEWAY SECTIONS

FIGURE 5-H



Slope Detail

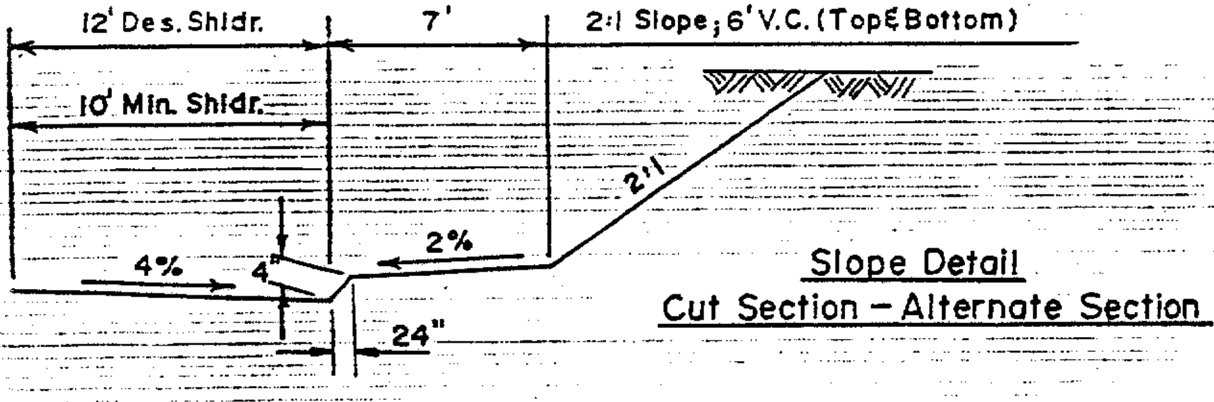
Fill Section – Cut Section



Slope Detail

Curbed Section (When Justified)

*Guide Rail not required for slopes 4:1 and flatter.



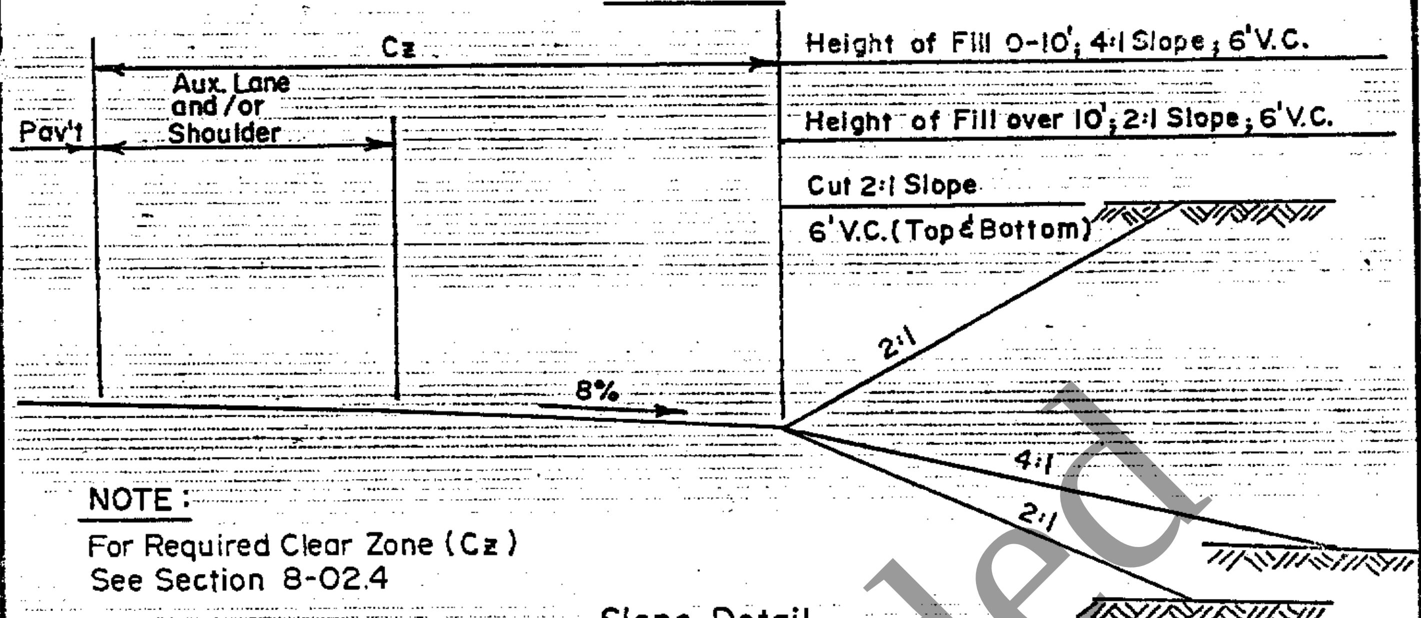
Slope Detail

Cut Section – Alternate Section

11/15/83

FREEWAY SECTIONS

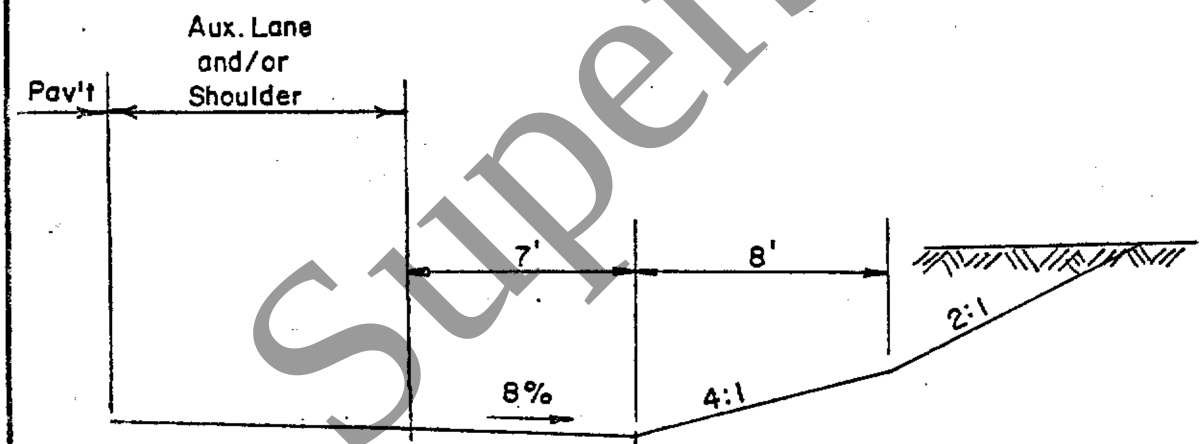
FIGURE 5-I



NOTE:

For Required Clear Zone (Cz)
See Section 8-02.4

Slope Detail
Fill Safety Section – Cut Safety Section

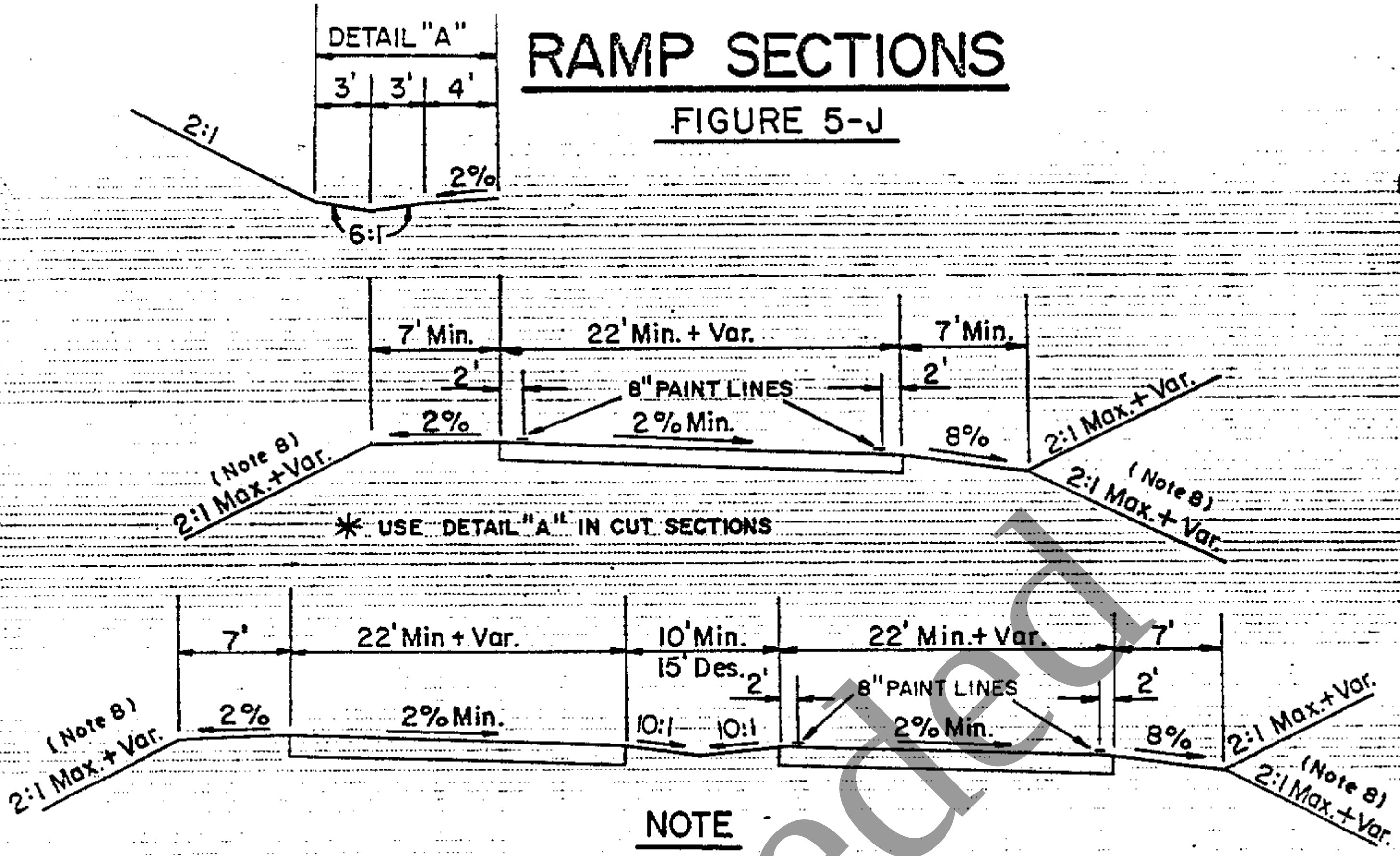


Slope Detail
Alternate– Cut Safety Section

11/15/83

RAMP SECTIONS

FIGURE 5-J



NOTE

1. THE MINIMUM RAMP WIDTH IS 22 FEET. THE WIDTH SHOULD BE ADJUSTED BASED ON VARIOUS OPERATING CONDITIONS, DESIGN VEHICLE AND CURVATURE. THE REQUIRED WIDTH SHOULD BE BASED ON THE SMALLEST RADIUS OF THE RAMP PROPER AND IS APPLICABLE THOUGHOUT THE FULL LENGTH OF THE RAMP. (FIGURE 7-B)

2. SUPERELEVATION SHOULD BE PROVIDED ON RAMPS.

3. SIDE SLOPES WHERE PRACTICAL SHOULD BE FLATTENED TO ELIMINATE THE NEED FOR GUIDE RAIL.

4. CURB MAY BE PROVIDED ON RAMPS WHEN REQUIRED FOR DRAINAGE CONTROL OR ACCESS CONTROL. MAXIMUM CURB HEIGHTS 6".

5. THE MEDIAN WIDTH ON OPPOSING RAMPS MAY BE REDUCED TO 4' WHERE CURB IS PROVIDED AND RAMP SPEEDS ARE 25 MPH OR LESS.

6. WHERE BARRIER CURB IS PROVIDED TO SEPARATE OPPOSING DIRECTIONS OF TRAVEL, THE MEDIAN WIDTH SHOULD BE 8'.

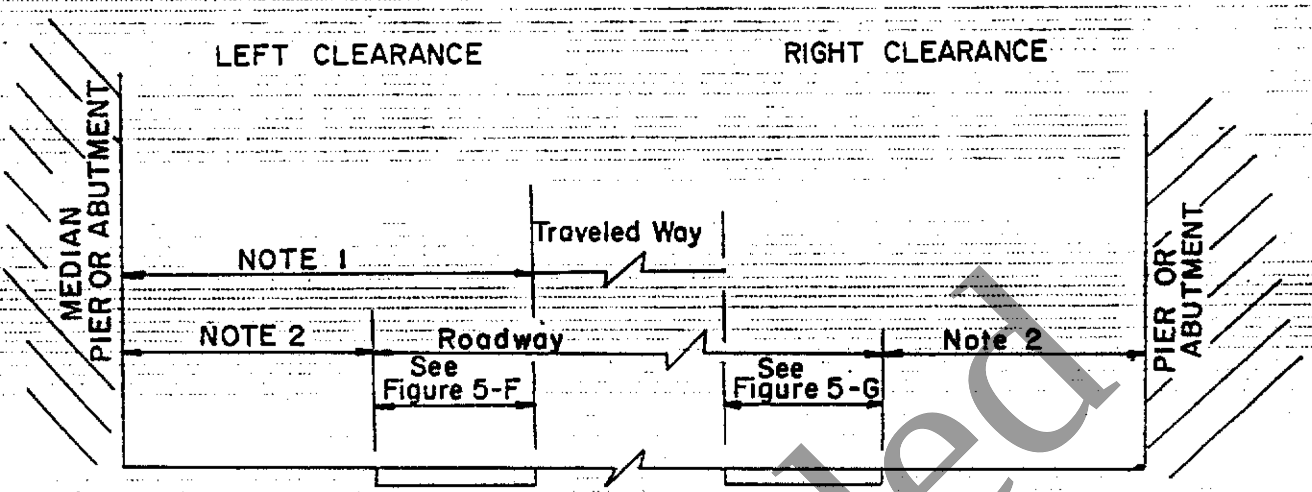
7. GUIDE RAIL SHOULD BE LOCATED ACCORDING TO THE "GUIDELINES FOR GUIDE RAIL DESIGN AND MEDIAN BARRIES", SECTION 8.

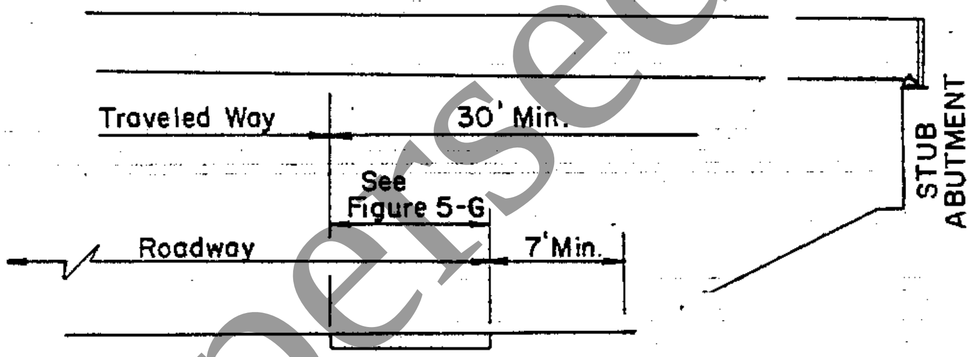
8. INTERIOR SIDE FILL SLOPES ON RAMPS SHOULD BE 4:1.

9. 2' PAINT LINE OFFSET PROVIDED FOR (1) INLET PLACEMENT (2) MINIMIZE COVERING OF LINE WITH DEBRIS (DIRT, GRASS CLIPPINGS, ETC.)

LATERAL BRIDGE CLEARANCES

FIGURE 5-K





INTERSTATE OR FREEWAY UNDERPASS

NOTE 1. WHEN PRACTICAL, PLACE PIER AT CENTERLINE OF MEDIAN. PROVISION FOR ADDITIONAL LANES SHOULD BE CONSIDERED WHEN, DETERMINING PIER OR ABUTMENT LOCATION. IF THERE IS A CONTINOUS MEDIAN BARRIER THE OFFSET SHOULD BE SUFFICIENT TO CONSTRUCT THE BARRIER IN FRONT OF THE PIER WITHOUT REDUCING THE SHOULDER WIDTH.

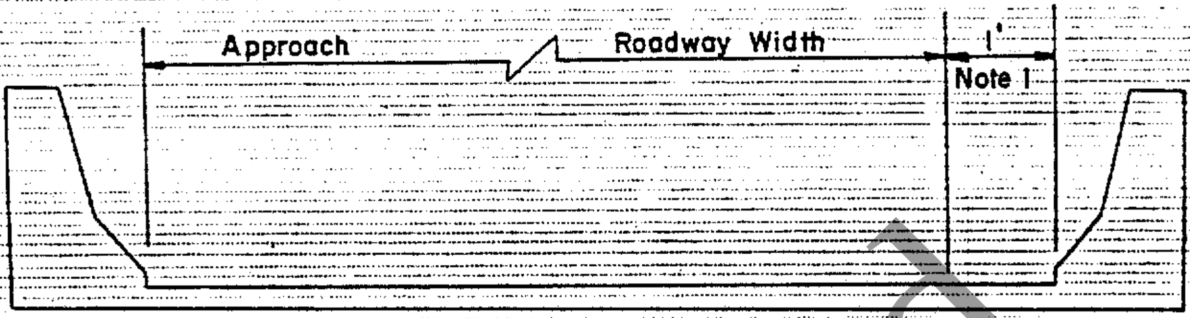
NOTE 2. MINIMUM OFFSET TO PIER OR ABUTMENT IS 8'-0" AND 4'-9" RESPECTIVELY.

NOTE: THESE DIMENSIONS ARE MINIMUMS. DESIGNS WHICH ELIMINATE THE NEED FOR A LONGITUDINAL BARRIER ARE PREFERRED WHEN PRACTICAL.

11/22/83

LATERAL BRIDGE CLEARANCES

FIGURE 5-L



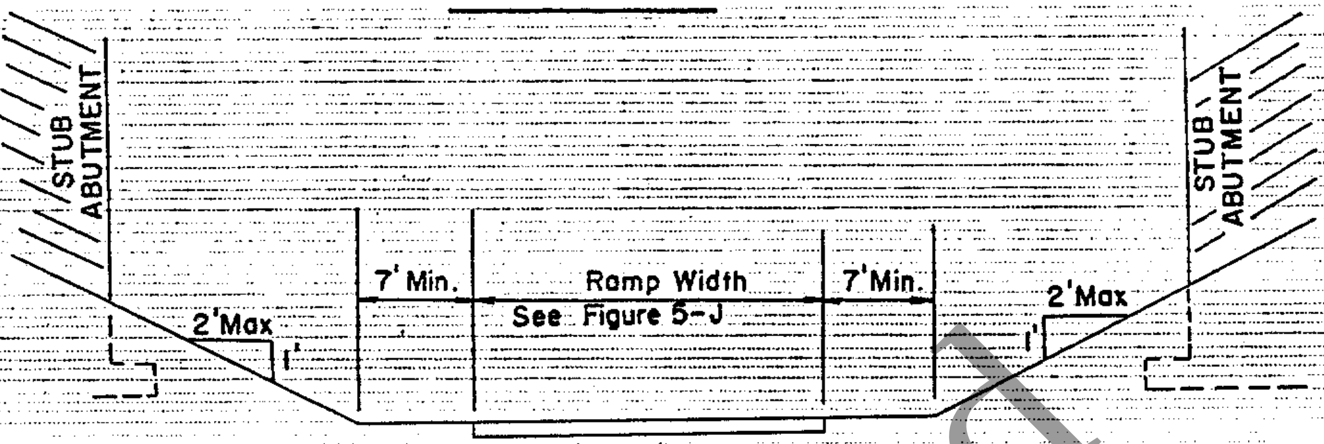
INTERSTATE OR FREEWAY OVERPASS

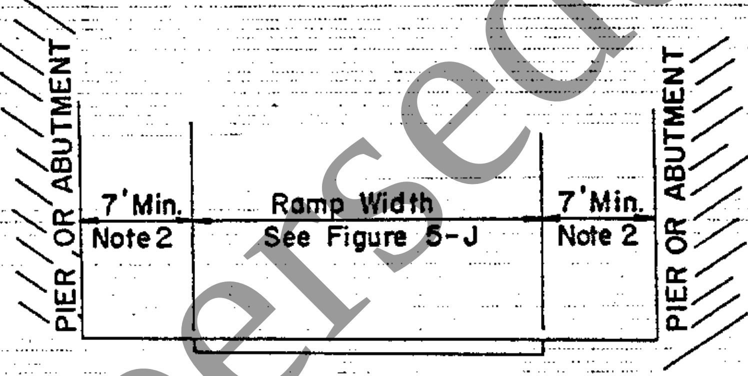
NOTE 1. A 2FT. OFFSET TO THE RIGHT SIDE PARAPET IS ACCEPTABLE ON PROJECTS UNDER DESIGN PRIOR TO JANUARY 1, 1984.

11/23/83

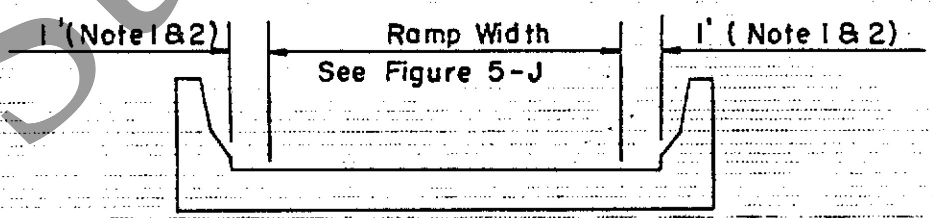
LATERAL BRIDGE CLEARANCES

FIGURE 5-M





RAMP UNDERPASS



RAMP OVERPASS

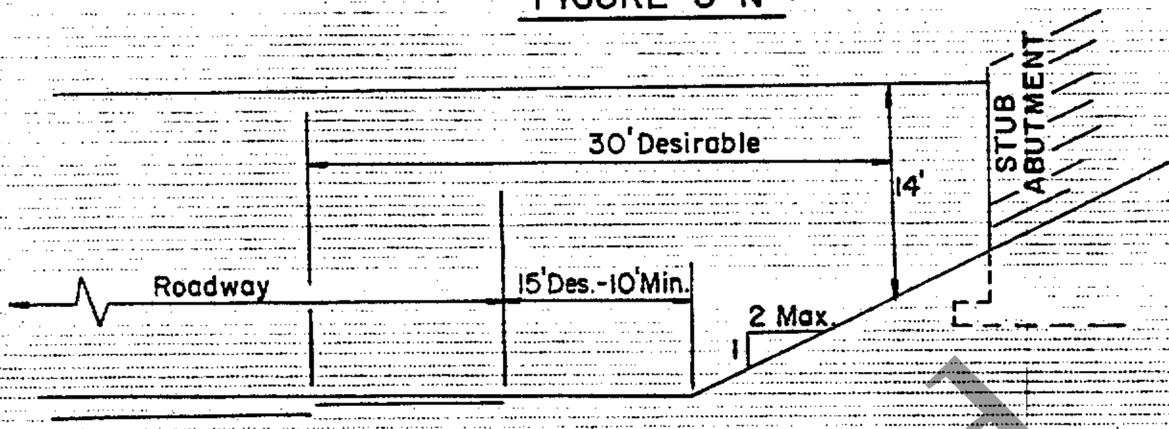
NOTE 1. A 2 FT. BARRIER OFFSET IS ACCEPTABLE ON PROJECTS WHICH WERE UNDER DESIGN PRIOR TO JANUARY 1, 1984.

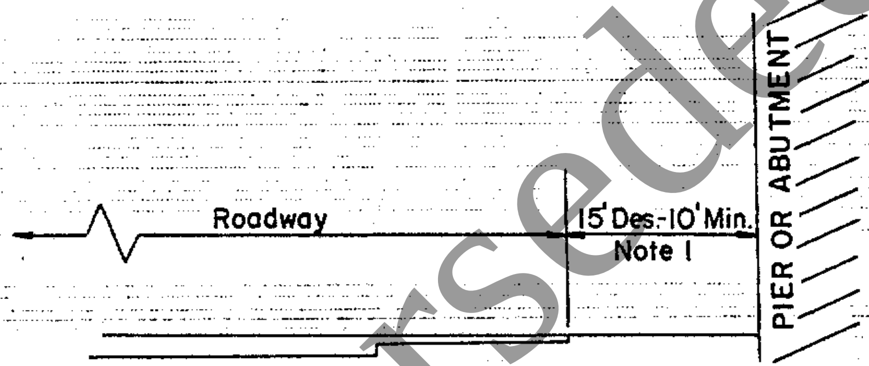
2. STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES GOVERNS (SEE FIGURE 4-A)

11/23/83

LATERAL BRIDGE CLEARANCES

FIGURE 5-N





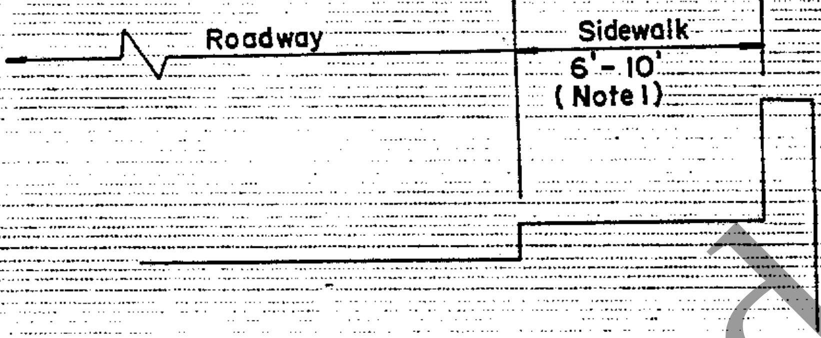
STATE HIGHWAY UNDERPASS

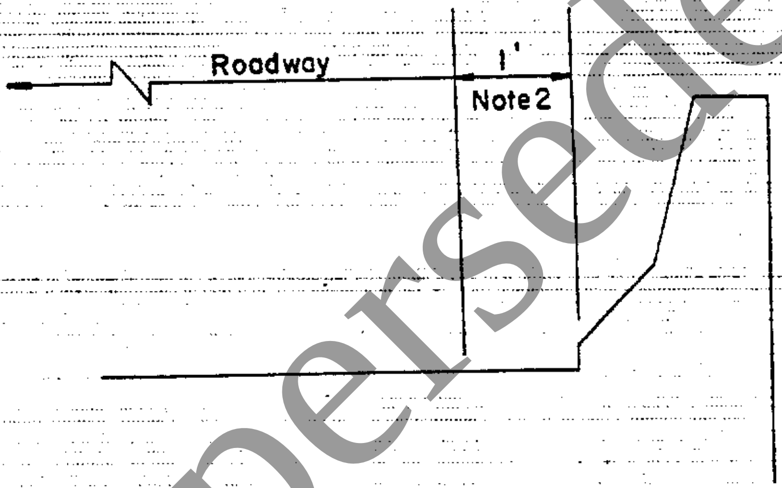
NOTE 1. STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES GOVERNS. (SEE FIGURE 4-A.)

11/23/83

LATERAL BRIDGE CLEARANCE

FIGURE 5-0





STATE HIGHWAYS AND LOCAL ROAD OVERPASS

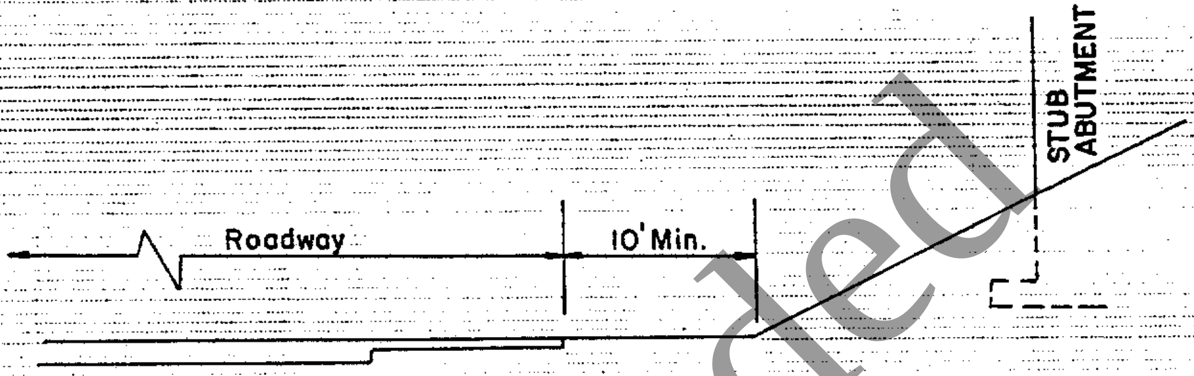
NOTE 1. SIDEWALKS SHOULD BE PROVIDED ON BOTH SIDES OF AN OVERPASS STRUCTURE.

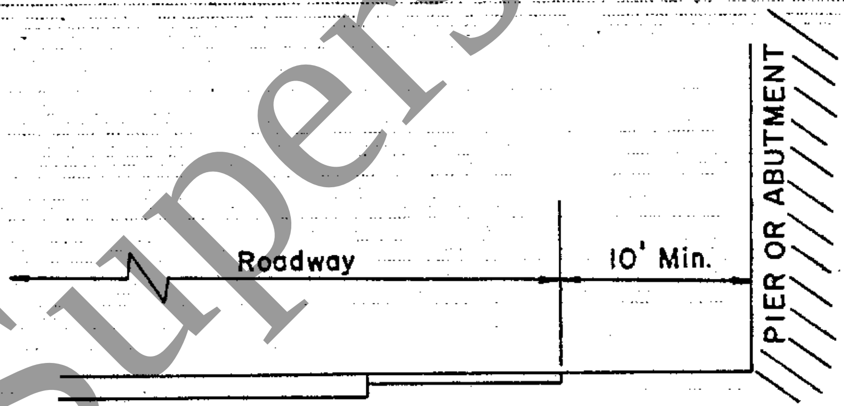
2. BARRIER CURB PARAPET SHOULD BE USED ONLY WHEN A SIDEWALK CANNOT BE JUSTIFIED ON BOTH SIDES OF A ROADWAY.

11/23/83

LATERAL BRIDGE CLEARANCES

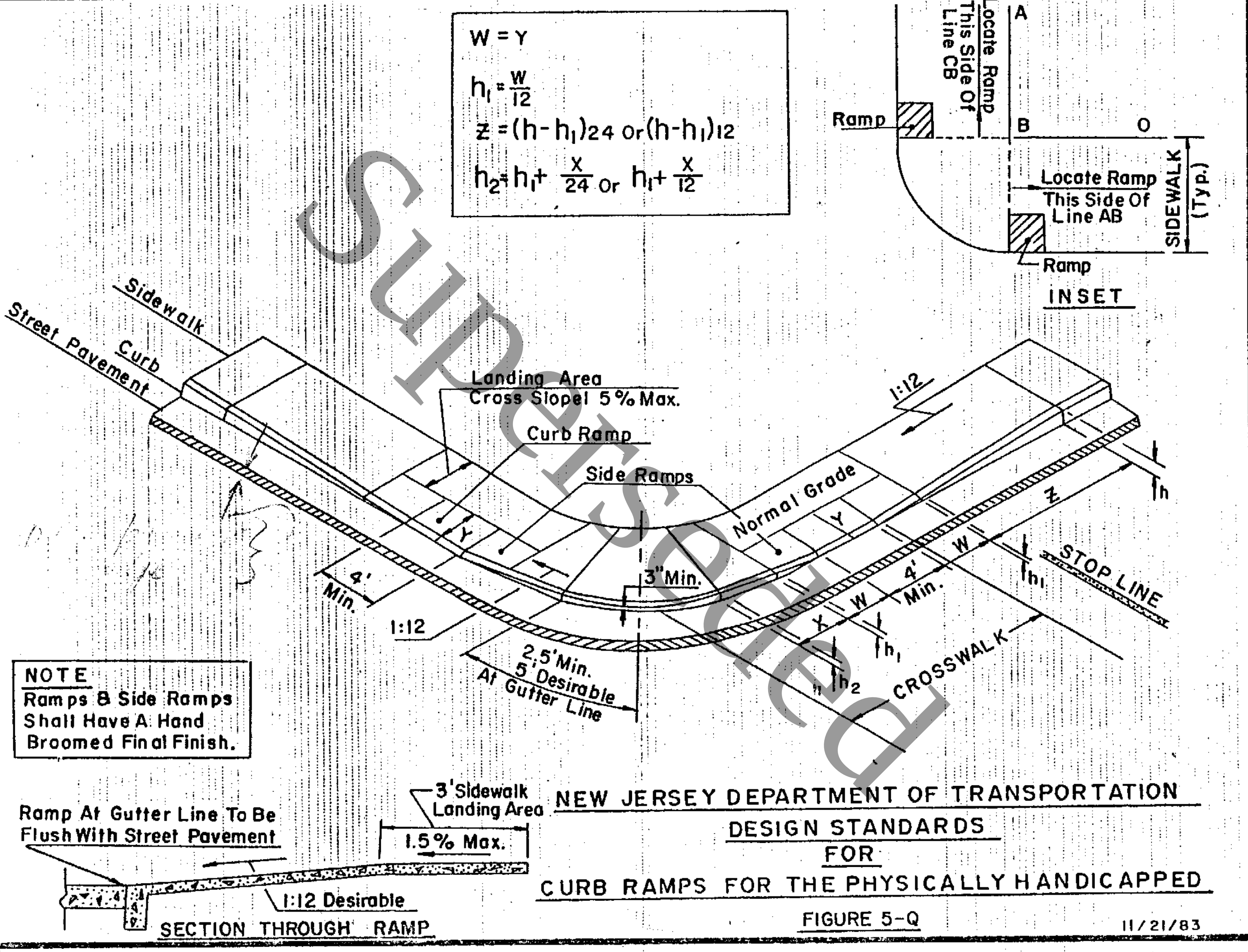
FIGURE 5-P





LOCAL ROAD UNDERPASS

11/23/83



NEW JERSEY DEPARTMENT OF TRANSPORTATION

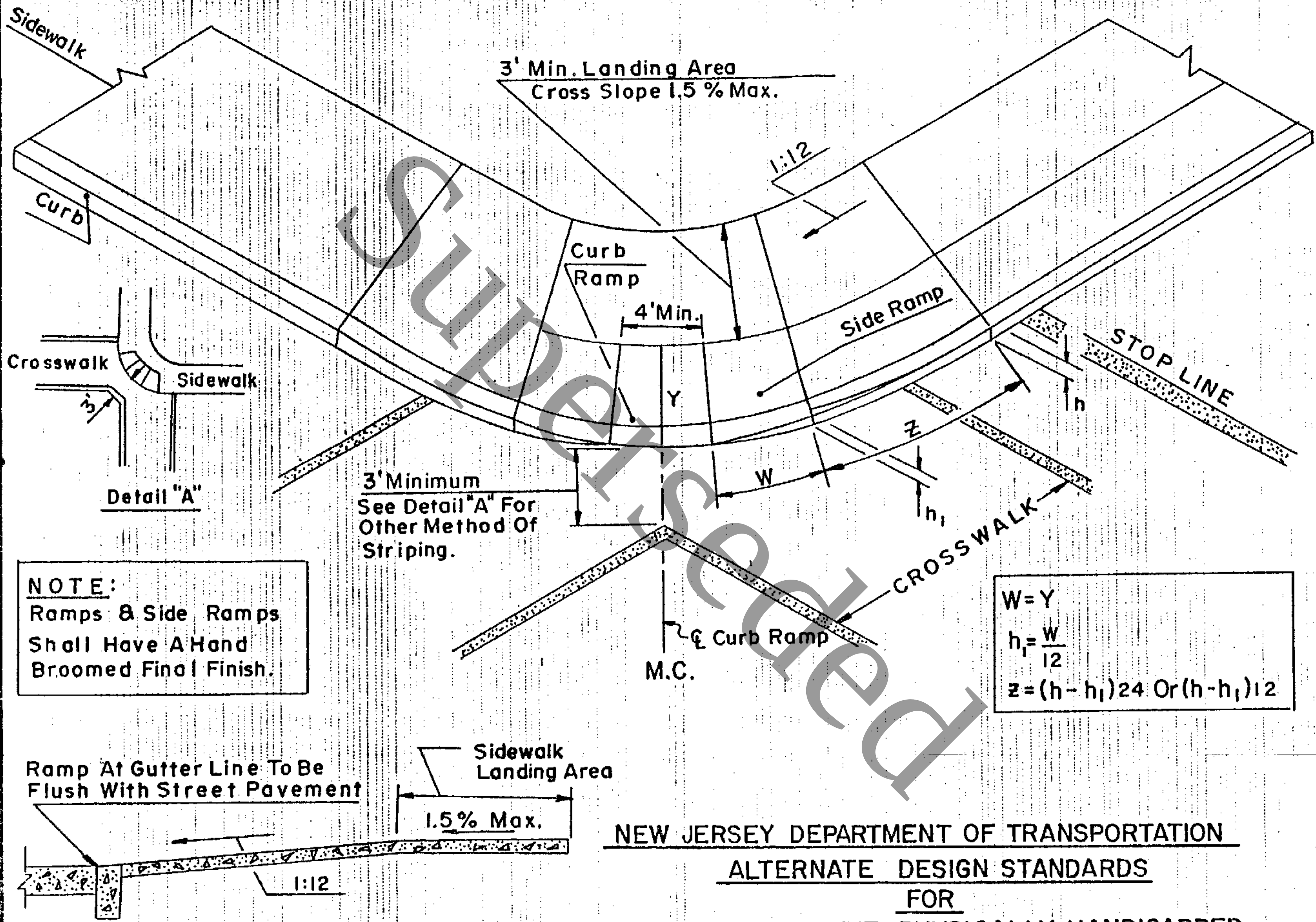
DESIGN STANDARDS

FOR

CURB RAMPS FOR THE PHYSICALLY HANDICAPPED

FIGURE 5-Q

11/21/83



NEW JERSEY DEPARTMENT OF TRANSPORTATION

ALTERNATE DESIGN STANDARDS

FOR

CURB RAMPS FOR THE PHYSICALLY HANDICAPPED

FIGURE 5-R

11/22/83

SECTION 6.

AT-GRADE INTERSECTIONS

6-01 GENERAL

Most highways intersect at grade. To minimize the resulting conflicts and to provide adequately for the anticipated crossings and turning movements, the geometric design of the intersection at grade must be given careful consideration.

Although intersections have many common factors, they are not subject to a set treatment, and must be looked upon as individual problems.

In varying degrees, four basic factors enter into the design 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 on the proper evaluation of the foregoing factors. In the redesign of an existing intersection, standards sometimes must be compromised due to the high cost of existing development or to the necessity of meeting rigid physical controls. In the design of a new intersection, however, such controls frequently can be avoided by a shift in line or grade of one or both of the intersecting highways.

6-02 GENERAL DESIGN CONSIDERATIONS

6-02.1 Capacity Analysis

Capacity analysis is one of the most important considerations in the design of intesections. 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 1965 Highway Capacity Manual and to Transportation Research Board Circular Number 212, Interim Materials on Highway Capacity, January 1980, for procedures in performing capacity computations.

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, signalized intersections should be located no closer than 1200 feet apart. In rural areas, the minimum spacing of intersections should be one-half mile.

6-02.3 Alignment and Profile

Intersections are points of conflict between vehicles, and between vehicles and pedestrians 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 flat 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. Intersection angles less than 60 degrees normally warrant realignment closer to 90 degree. 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 profile 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 grades 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 50 feet, should be provided for minor streets where stop sign control is employed and the approach grade is up towards the intersection. Such slopes should desirably be less than one percent and no more than 3 percent.

The profile 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 profile line of the major highway should be carried through the intersection, and that of the cross road adjusted to it. Intersections with a minor road crossing a multi-lane divided highway with narrow median and superelevated curve should be avoided whenever possible because of the difficulty in adjusting grades to provide a suitable crossing. Profile lines of separate turning roadways should be designed to fit the cross slopes and longitudinal grades of the intersection legs.

As a rule, the horizontal and vertical alignment are subject to greater restrictions 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.

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.

6-03 SIGHT DISTANCE

6-03.1 General

There must be unobstructed sight along both roads at an intersection and across their included corner for distances sufficient to allow the operators of vehicles approaching the intersection or stopped at the intersection to carry out whatever manuevers may be required to negotiate the intersection.

Any object within the sight triangle high enough above the elevation of the adjacent roadways to constitute a sight obstruction should be removed or lowered. Such objects include but are not limited to cut slopes, hedges, bushes, tall crops, signs, buildings, parked vehicles, etc.

6-03.2 Stop Control on Cross Street

Intersection designs should provide sufficient sight distances to avoid potential conflicts between vehicles turning onto or crossing a highway from a stopped position and vehicles on the through highway operating at the design speed.

Figure 6-A indicates the required sight distance along the major roadway for various design vehicles to cross safely. Where the median width on a divided highway is equal to or greater than the vehicle length, the crossing can be accomplished in 2 steps. The vehicle crosses the first pavement, stops within the median opening, 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 must be included as part of the roadway width (w).

Figure 6-B indicates the sight distance requirement to permit passenger vehicles to turn left or right onto a 2-lane highway from a stopped position.

6-03.3 Yield Control

When an intersection is controlled by a yield sign, the sight triangle is governed by the design speed on the main highway and that of the approach highway or ramp.

Suggested approach speeds on the yield controlled approach are 15 mph for urban conditions and 20 to 25 mph for rural conditions. Where two major highways intersect and one leg is controlled by a yield sign, the design speed on both highways should be used in determining the minimum sight triangle.

Figure 6-C illustrates the method for establishing the minimum sight triangle for yield controlled intersections.

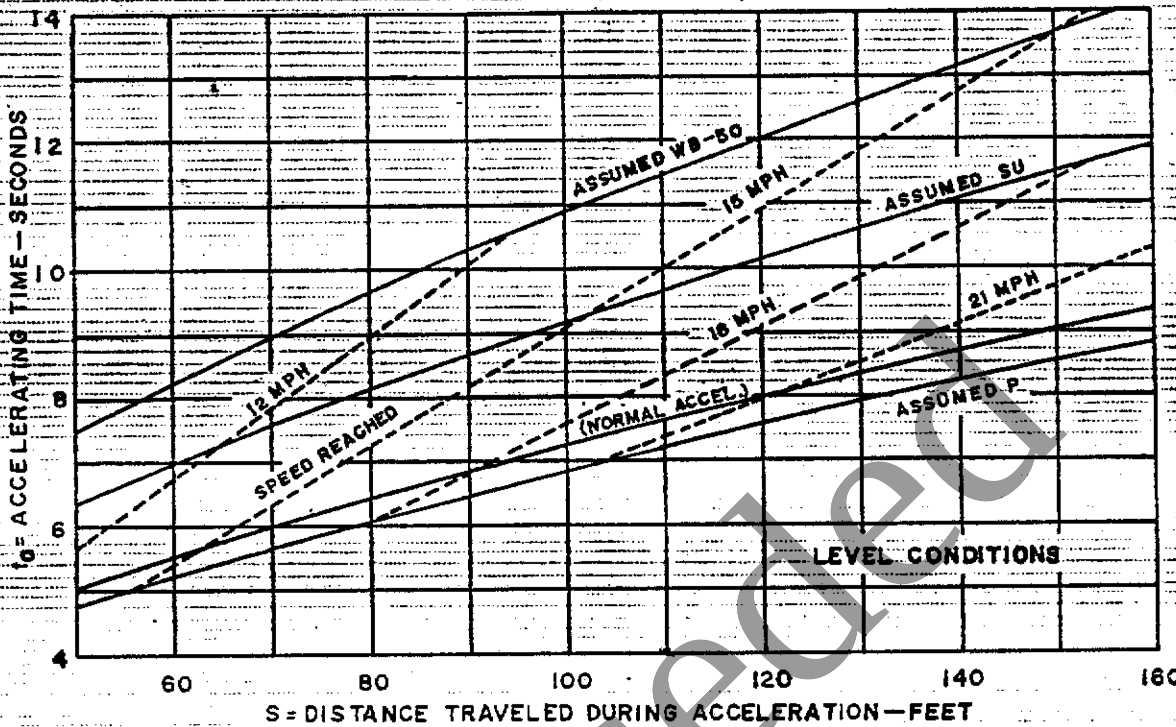
6-03.4 Sight Distance at Signalized Intersections

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 signal should consider the effect of parked vechicles. As a minimum, stopping sight distance should be provided.

6-04 TURNING MOVEMENTS

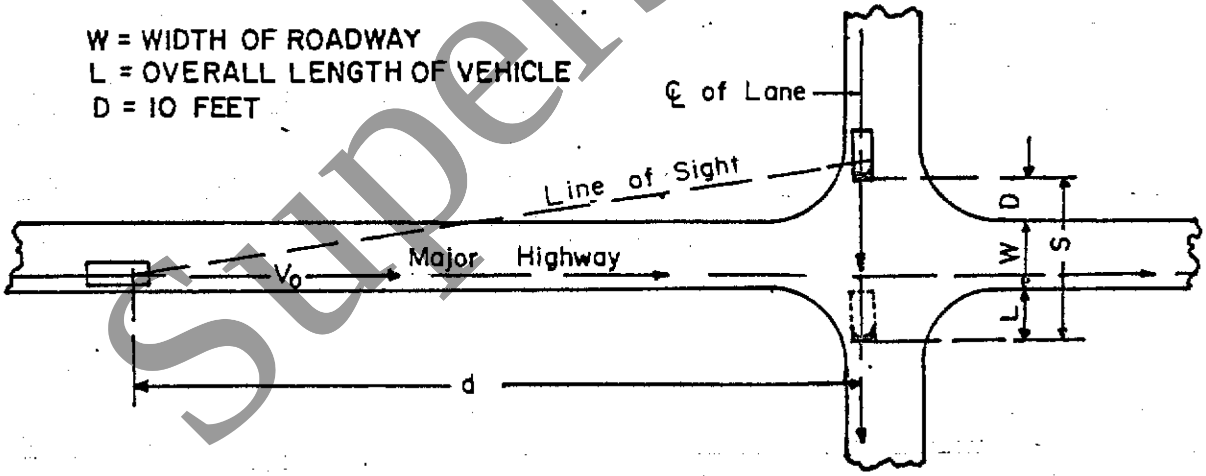
6-04.1 General

One of the primary concerns of intersection design is to provide adequately for left and right turning movements. The pavement and



SIGHT DISTANCE AT INTERSECTIONS
DATA ON ACCELERATION FROM STOP

W = WIDTH OF ROADWAY
L = OVERALL LENGTH OF VEHICLE
D = 10 FEET



STOP CONTROL ON MINOR ROAD

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

where, V = design speed on major highway, mph

J = perception and reaction time, assumed 2 seconds.

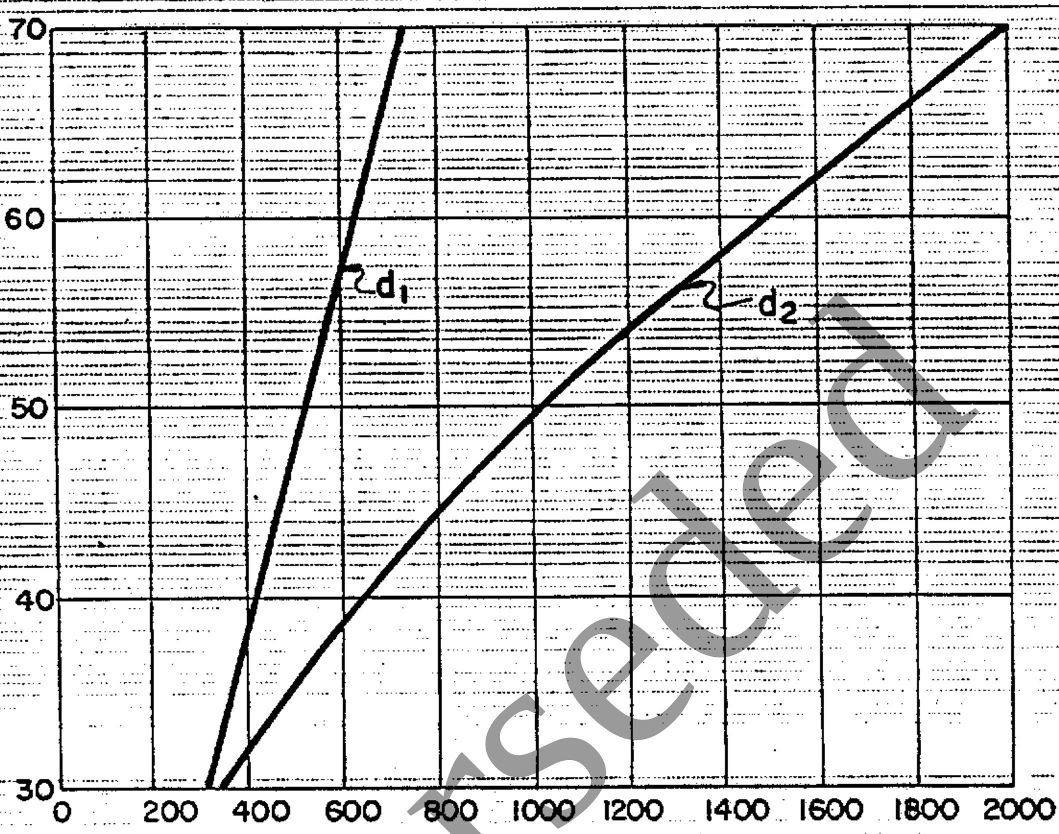
t_a = time required to traverse distance $S = D + W + L$

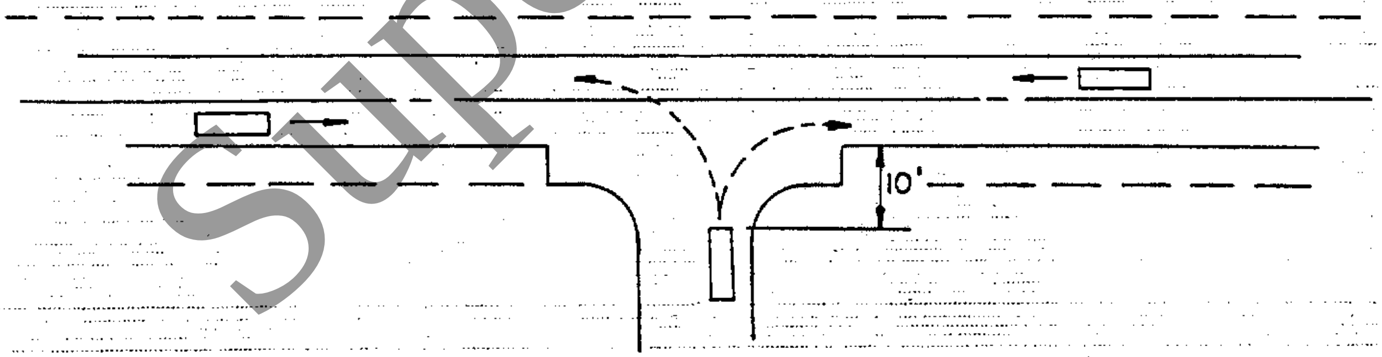
Source: A Policy on Geometric Design of Rural Highways: AASHTO 1965

SIGHT DISTANCE AT INTERSECTIONS LEFT OR RIGHT TURNING VEHICLES

FIGURE: 6-B

DATE: 12/83





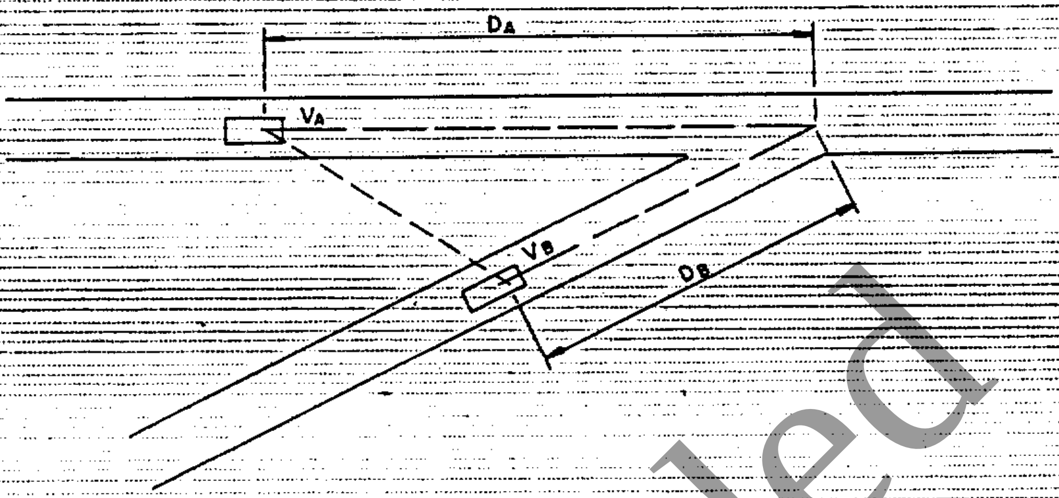
d_1, Desirable Sight Distance Along The Highway To Permit A Passenager Vehicle To Turn Left And Clear The Vehicle Approaching From The Left.

d_2, Desirable Sight Distance Along The Highway To Permit A Passenager Vehicle To Turn Left Or Right Before Being Over Taken By A Vehicle Travelling in The Same Direction.

YIELD CONTROL

FIGURE: 6-C

DATE: 11/83



WITH ACCELERATION LANE

DESIGN SPEED (MPH) V_A OR V_B	DISTANCE (FT.) D_A OR D_B
20	90
30	130
40	180
50	220
60	260
70	310

WITHOUT ACCELERATION LANE

DESIGN SPEED	DISTANCE DA'	DISTANCE DB'
20	240	120
30	400	200
40	550	275
50	700	350
60	950	475
70	1200	600

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 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 and should be used on all intersection designs to assure that encroachments into opposing lanes are eliminated or minimized.

6-0~~3~~4.3 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. Generally, radii of 15 to 25 feet are adequate for passenger vehicles. Radii of 30 feet or more should be provided to allow an occasional truck or bus to turn without much encroachment. Radii of 50 feet or more should be provided where large truck combinations and buses turn frequently.

When provisions must be made for the larger truck units, and the angle of turn exceeds 90 degrees, a 3-centered compound curve may be used in lieu of a single circular arc with a large radius.

Figure 6-J indicates the minimum treatment at unchannelized intersections.

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

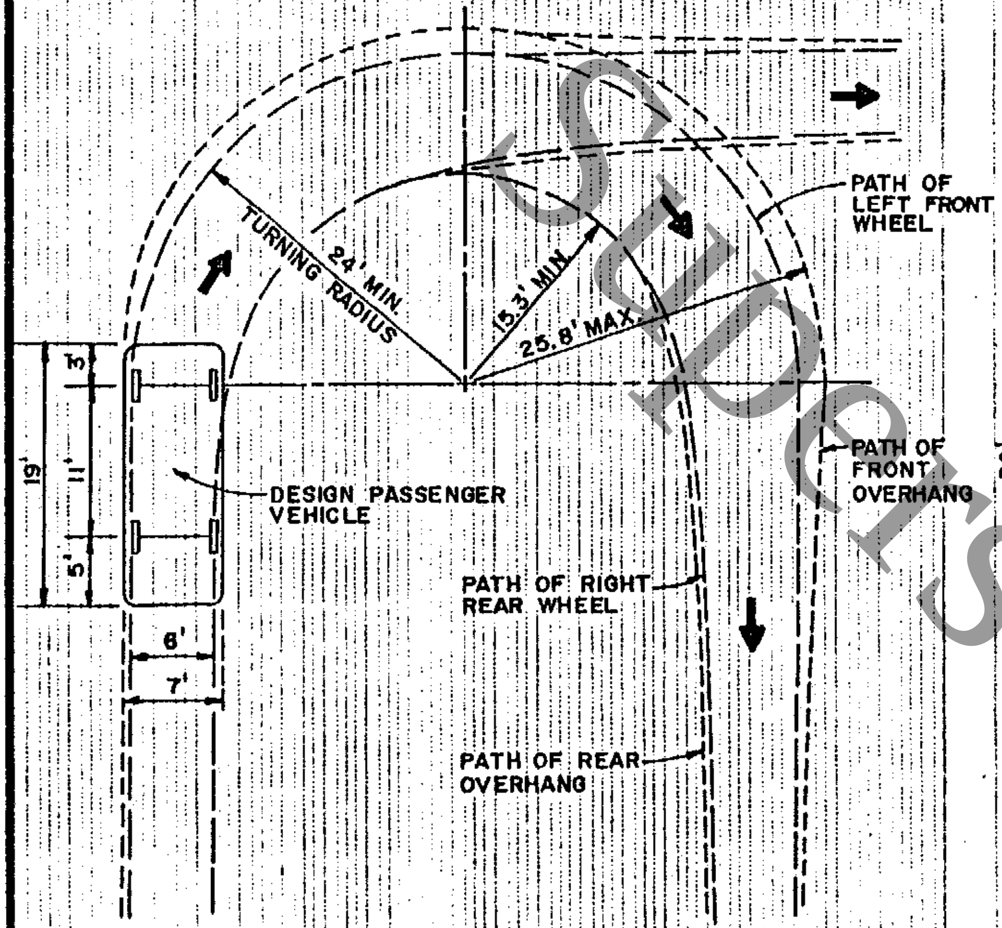


FIGURE 6-D

MINIMUM TURNING PATH FOR
P DESIGN VEHICLE

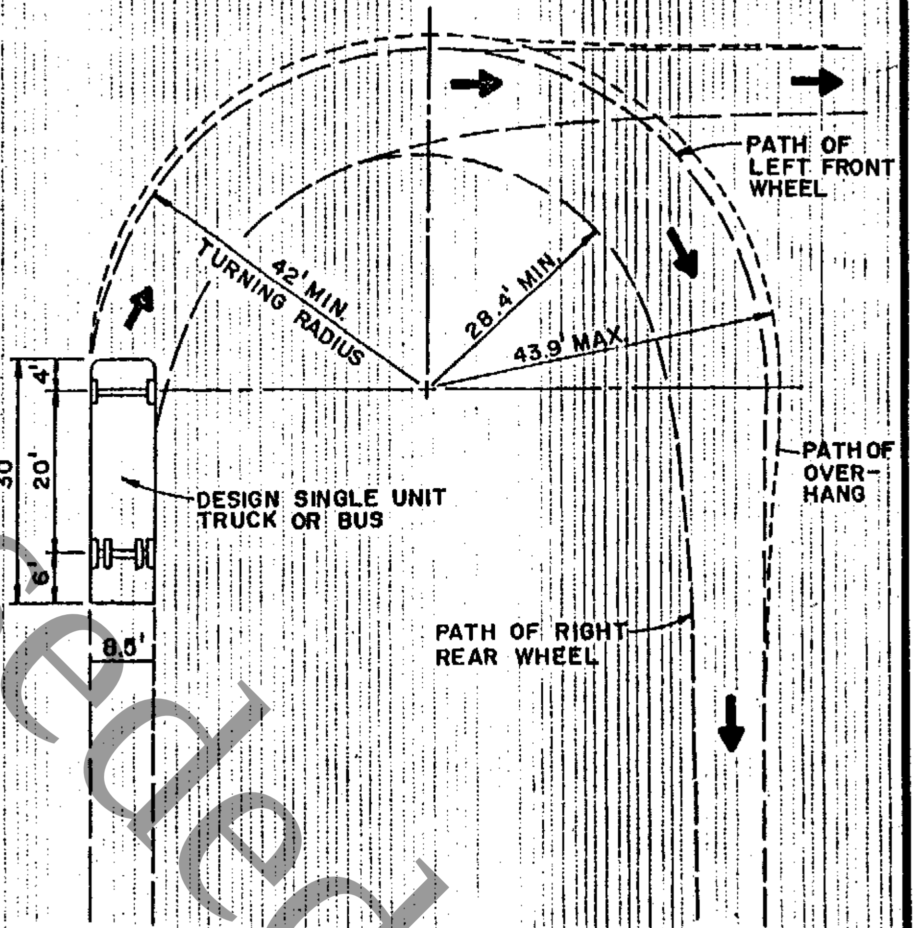


FIGURE 6-E

MINIMUM TURNING PATH FOR
SU DESIGN VEHICLE

11/12/83

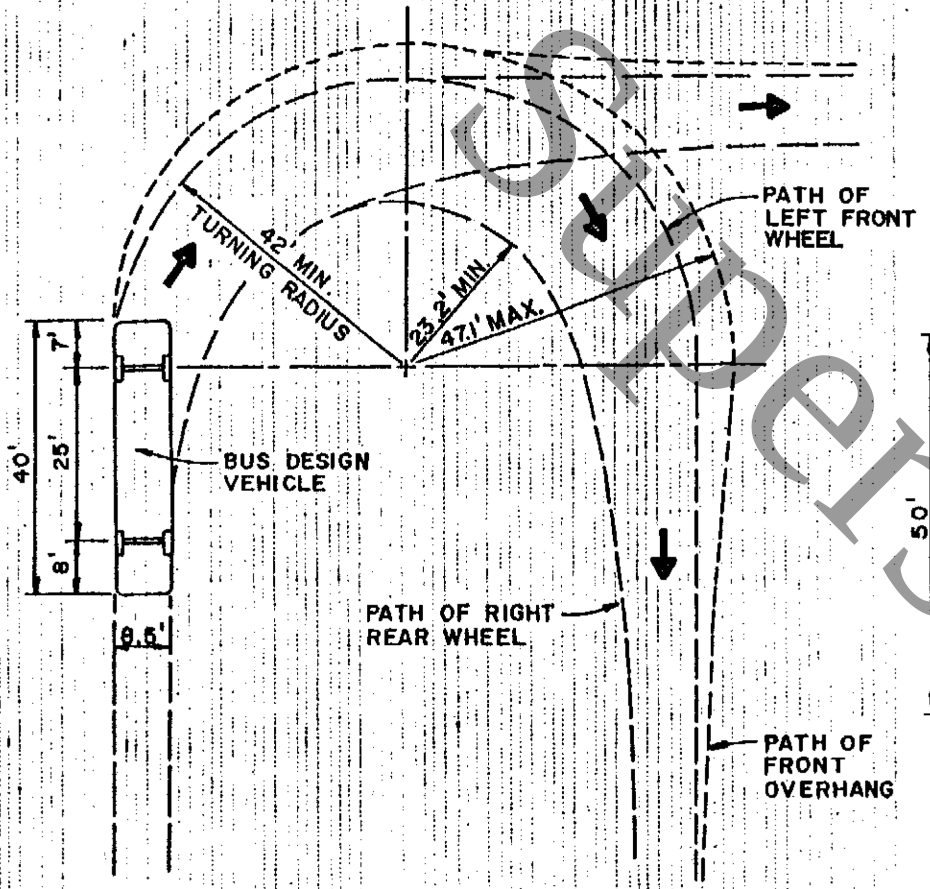


FIGURE 6-F

MINIMUM TURNING PATH FOR BUS DESIGN VEHICLE

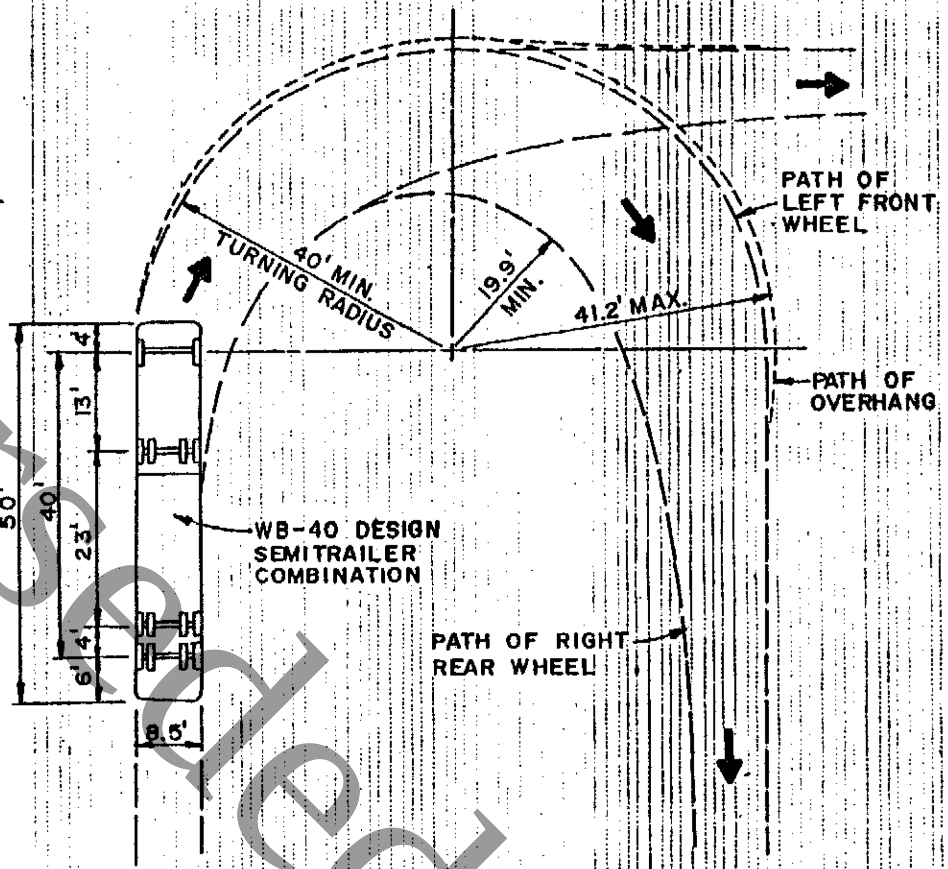


FIGURE 6-G

MINIMUM TURNING PATH FOR WB-40 DESIGN VEHICLE

11/12/83

Superseded

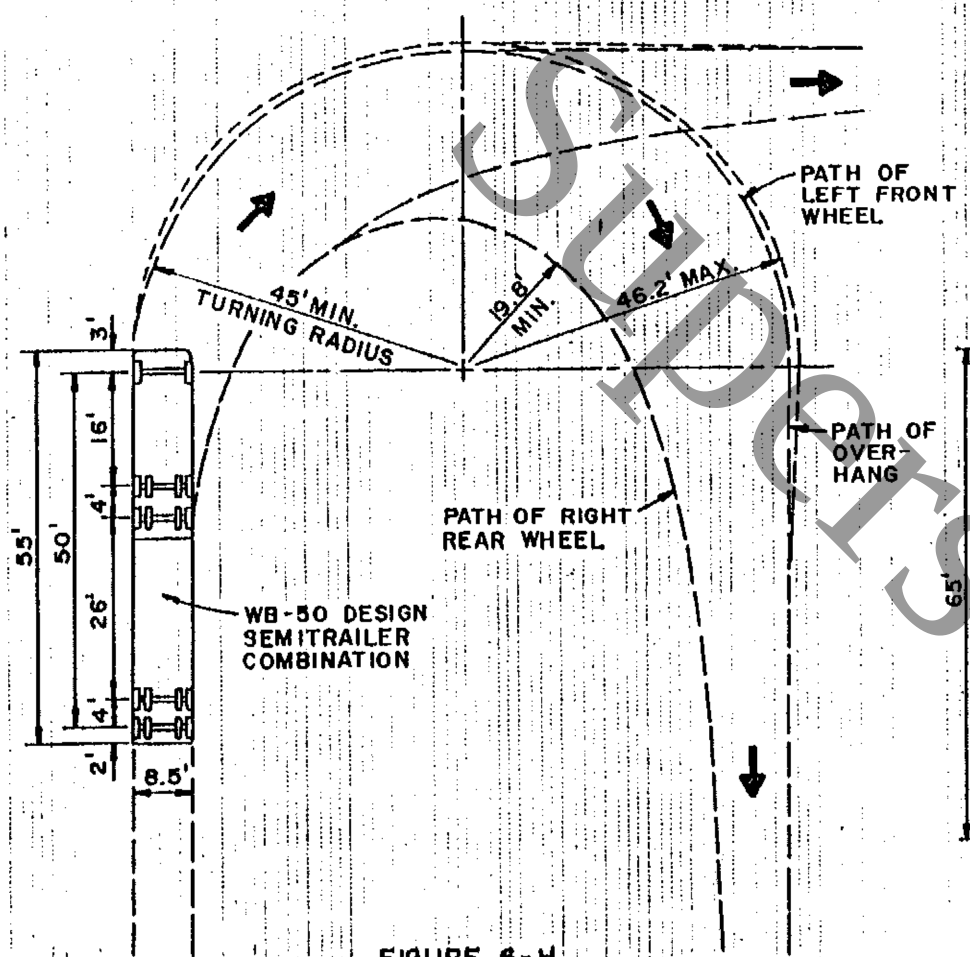


FIGURE 6-H

MINIMUM TURNING PATH FOR WB-50 DESIGN VEHICLE

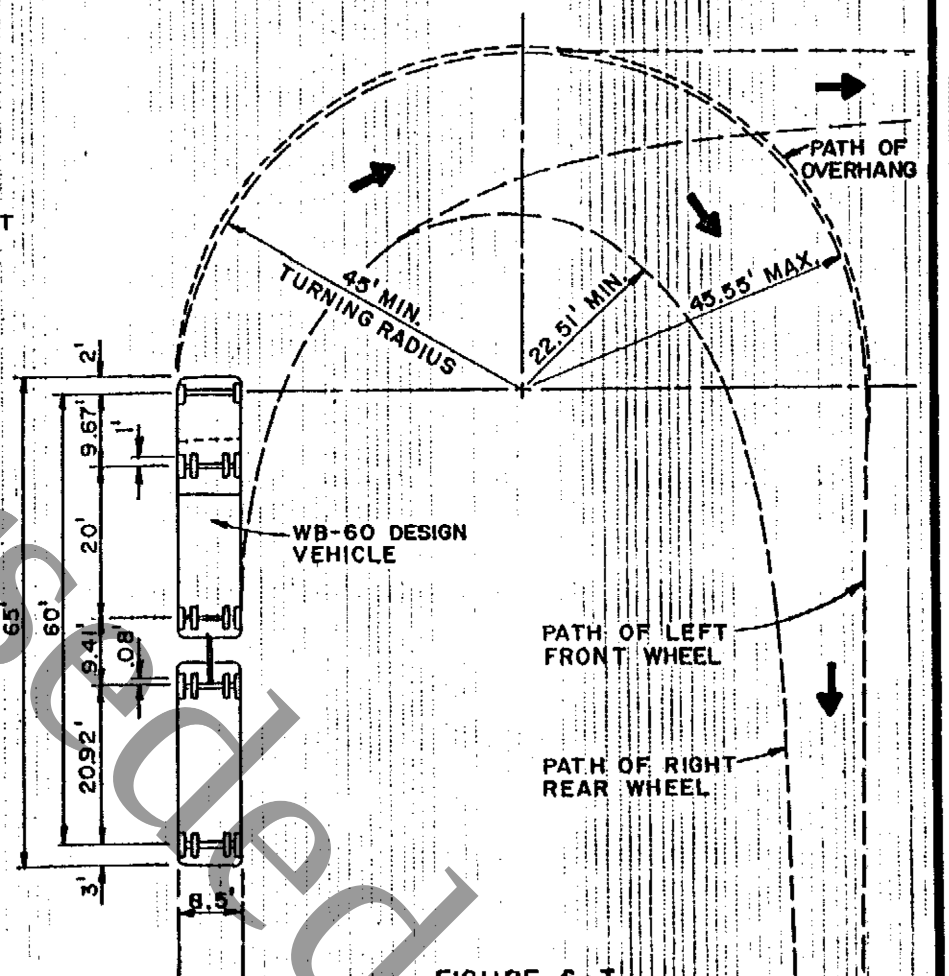
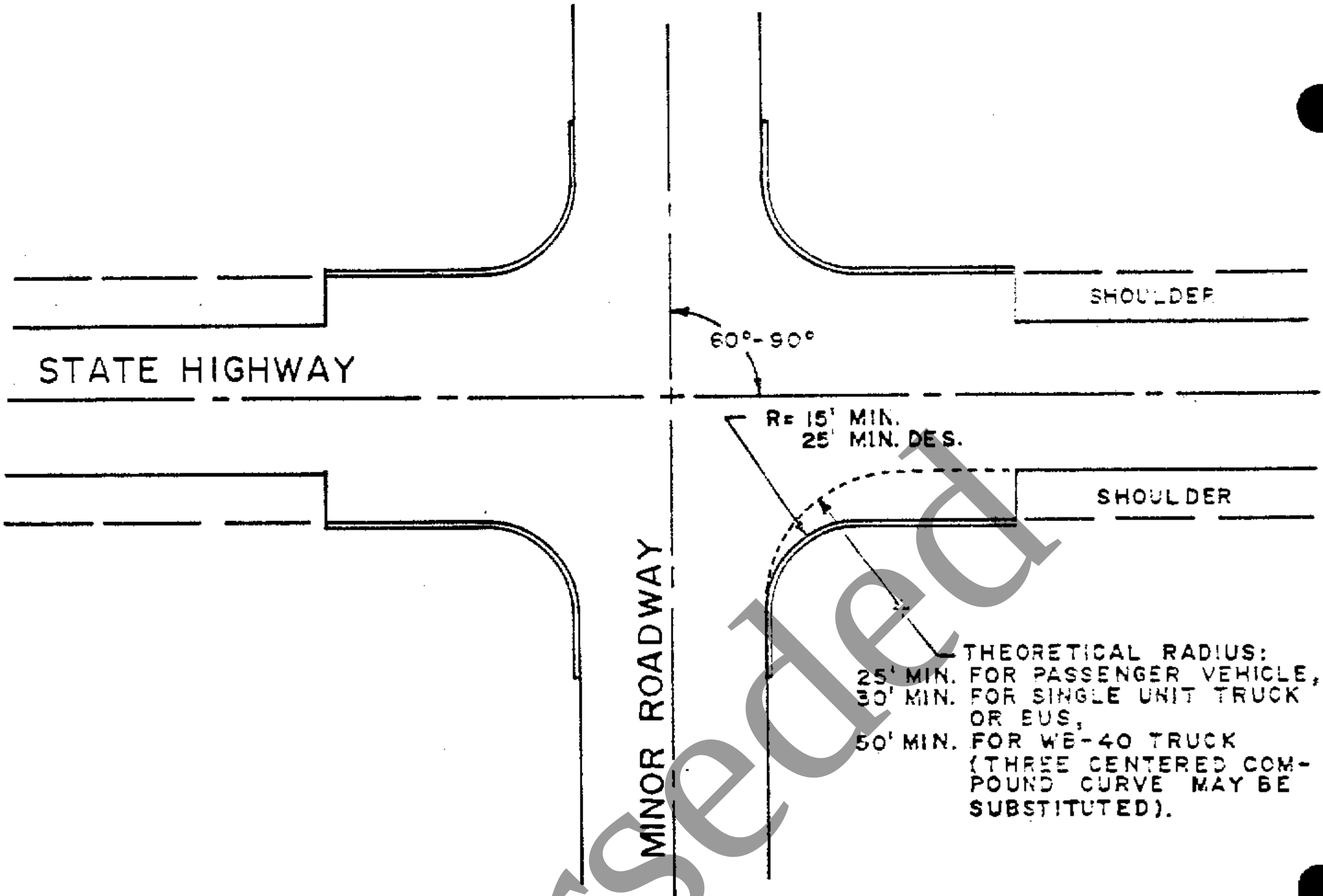


FIGURE 6-I

MINIMUM TURNING PATH FOR WB-60 DESIGN VEHICLE

11/12/83



DESIGN GUIDELINES

1. PHYSICAL CURB RETURN SHOULD BE CLEAR OF THEORETICAL RADIUS.
2. TRUCK VOLUMES DICTATE THE THEORETICAL RADIUS TO BE USED. WHERE TRUCK TRAFFIC IS LIGHT, A SU TRUCK RADIUS SHOULD BE USED WHERE POSSIBLE.
3. A TURNING TEMPLATE FOR THE APPROPRIATE DESIGN VEHICLE SHOULD BE USED TO CHECK THE ADEQUACY OF RADII RETURNS.
4. FOR INTERSECTION SKEW ANGLES LESS THAN 60°, CHANNELIZATION SHOULD BE PROVIDED.
5. WHERE TURNING VOLUMES ARE HIGH, AUXILARY LANES THROUGH THE INTERSECTION MAY BE WARRANTED.
6. CHECK APPLICABLE SIGHT DISTANCES.

INTERSECTION TURNING RADII

FIGURE 6-J
NOT TO SCALE

D.B. 11/22/82

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.

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 a layout designed to include islands.

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 curbs to a pavement area marked by paint.

Islands generally are included in intersection design 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 enchance driver expectancy. The designer should also refer to Part V of the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) for 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

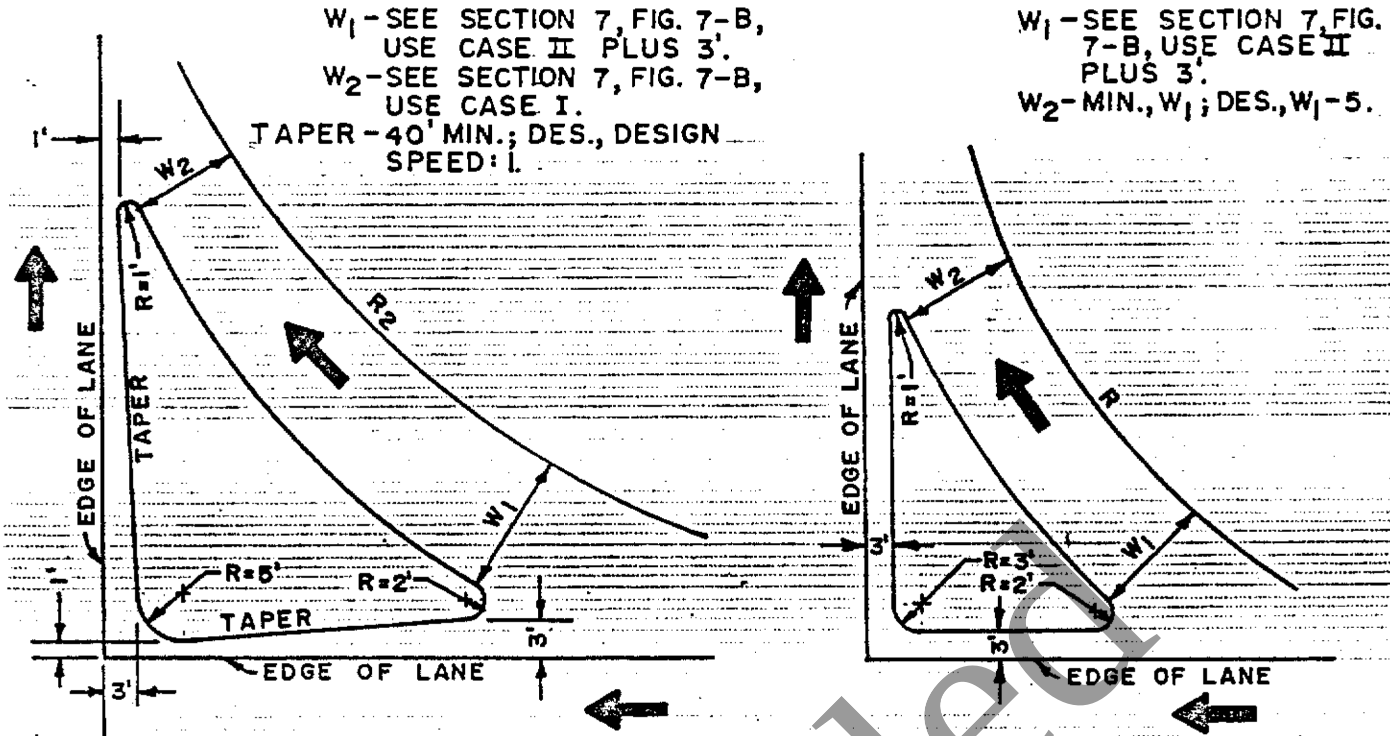
   Island sizes and shapes vary materially from one intersection to another. Islands should be large enough to command attention. The smallest curbed island that normally should be considered is one that has an area of approximately 50 square feet for urban streets, and 75 square feet for rural intersection.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

2. Approach-End Treatment

   The approach end of a curbed island should be consipicuous 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 3 feet from the normal edge of through pavement and 2 to 3 feet from the pavement edge of the turning roadway. Figure 6-L 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, Figure 6-L.



Large Islands

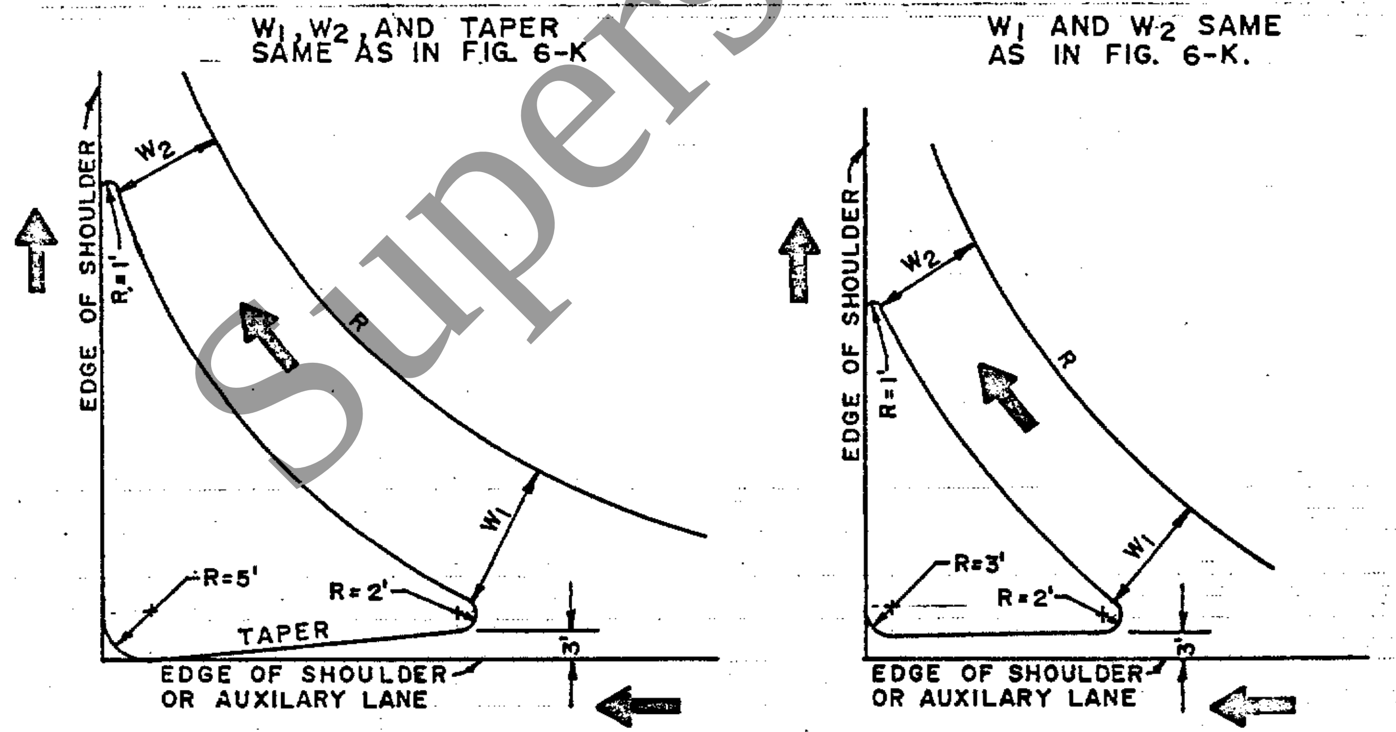
Small & Intermediate Islands

ISLANDS WITH NO SHOULDERS

FIGURE 6-K

NOT TO SCALE

D.B. 11/23/82



Large Islands

Small & Intermediate Islands

ISLANDS WITH SHOULDERS OR AUXILARY LANES

FIGURE 6-L

NOT TO SCALE

D.B. 11/24/82

3. Divisional Islands

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

6-05.3 Auxilary Lanes

Auxilary lanes at intersections serve a wide range of purposes including space for deceleration and acceleration, bus stops, and storage for turning vehicles.

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 with yield 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.

An auxilary 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. See Figure 6-N for desirable lengths of auxilary lanes.

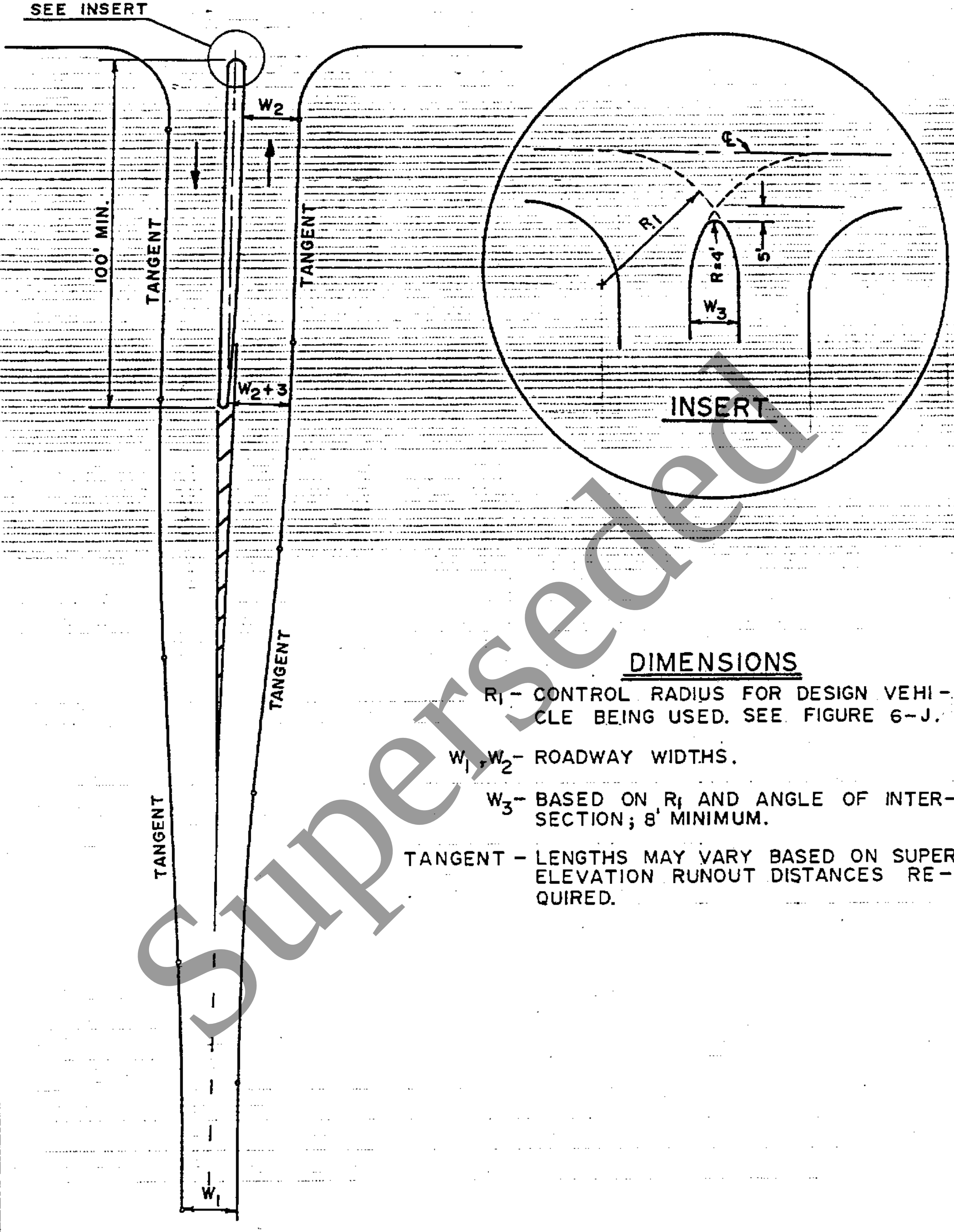
6-05.4 Median Openings

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

The length (L) of the median opening desirably should equal the full roadway of the cross road, shoulder to shoulder plus 5 feet on both sides. The control radius (R1) is determined by the design vehicle as follows:

P and SU	- 40 feet
SU, BUS, WB-40	- 50 feet
WB-50	- 75 feet

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



DIMENSIONS

R_1 - CONTROL RADIUS FOR DESIGN VEHICLE BEING USED. SEE FIGURE 6-J.

W_1, W_2 - ROADWAY WIDTHS.

W_3 - BASED ON R_1 AND ANGLE OF INTERSECTION; 8' MINIMUM.

TANGENT - LENGTHS MAY VARY BASED ON SUPERELEVATION RUNOUT DISTANCES REQUIRED.

DIVISIONAL ISLAND TREATMENT

FIGURE 6-M

NOT TO SCALE

D.B. 12/21/82

AUXILARY LANE LENGTHS

FIGURE 6-N

HIGHWAY DESIGN SPEED MPH (V)	L = LENTH OF DECELERATION LANE - FEET								
	FOR DESIGN SPEED OF EXIT CURVE - MPH (V')								
	STOP CONDITION	15 50'R	20 90'R	25 150'R	30 230'R	35 310'R	40 430'R	45 550'R	50 690'R
	FOR AVERAGE RUNNING SPEED ON EXIT CURVE - MPH (V'a)								
	0	14	18	22	26	30	36	40	44
30	235	185	160	140	---	---	---	---	---
40	315	295	265	235	185	155	---	---	---
50	435	405	385	355	315	285	225	175	---
60	530	500	490	460	430	410	340	300	240
65	570	540	530	490	480	430	380	330	280

LENGTH OF DECELERATION LANES

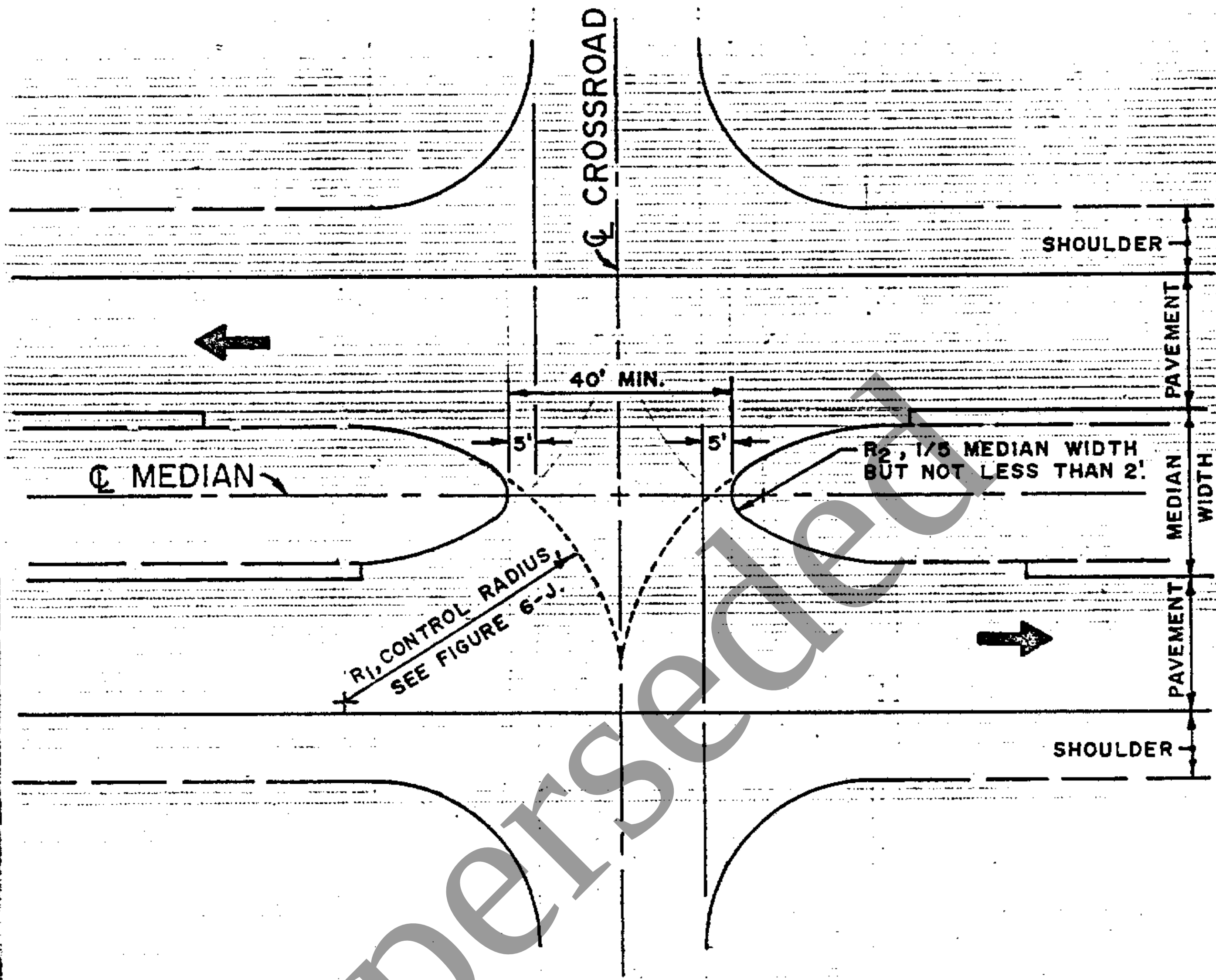
HIGHWAY DESIGN SPEED MPH (V)	L = LENTH OF ACCELERATION LANE - FEET								
	FOR DESIGN SPEED OF EXIT CURVE - MPH (V')								
	STOP CONDITION	15 50'R	20 90'R	25 150'R	30 230'R	35 310'R	40 430'R	45 550'R	50 690'R
	AND INITIAL SPEED - MPH (V'a)								
	0	14	18	22	26	30	36	40	44
30	190	---	---	---	---	---	---	---	---
40	380	320	250	220	140	---	---	---	---
50	760	700	630	580	500	380	160	---	---
60	1,170	1,120	1,070	1,000	910	800	590	400	170
70	1,590	1,540	1,500	1,410	1,330	1,230	1,010	830	580

LENGTH OF ACCELERATION LANES

NOT TO SCALE

SOURCE: A Policy on Geometric Design of Rural Highways, AASHTO 1963.

D.B. 12/17/82



MEDIAN OPENING

FIGURE 6-O

NOT TO SCALE

D.D. 11/15/82

rural divided roadways where intersections are spaced in excess of one-half mile apart. Median widths in such cases should be at least 20 feet and desirably 30 feet to provide adequate protection for the vehicle executing the U-turn movement from the median.

6-05.5 Median Openings For Emergency Vehicles

Although it is desirable to require all U-turns by official vehicles be accomplishd at intersections or interchanges, experience demonstrates that some emergency median openings are necessary for proper law enforcement, fire-fighting apparatus, ambulances and maintenance activates. Where median openings are provided for use by official vehicles only, they shall be limited in number and carefully located.

On freeways and Interstate highways where the spacing of interchanges is greater than approximately 3 miles, a U-turn median opening may be provided at a favorable location halfway between the interchanges. Where the spacing of interchanges is greater than about 6 miles, U-turn median openings may be provided so that the distance between such openings or interchange is not greater than about 3 miles.

U-turn median openings in general should not be provided on urban freeways due to the close spacing of interchanges. Generally due to the close proximity of intersection on divided arterials emergency U-turn median openings are not provided. However when emergency facilities are located between intersections there may be a need for direct access to the highway.

See figures 6-P & 6-Q for typical emergency median opening treatments.

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 may 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.

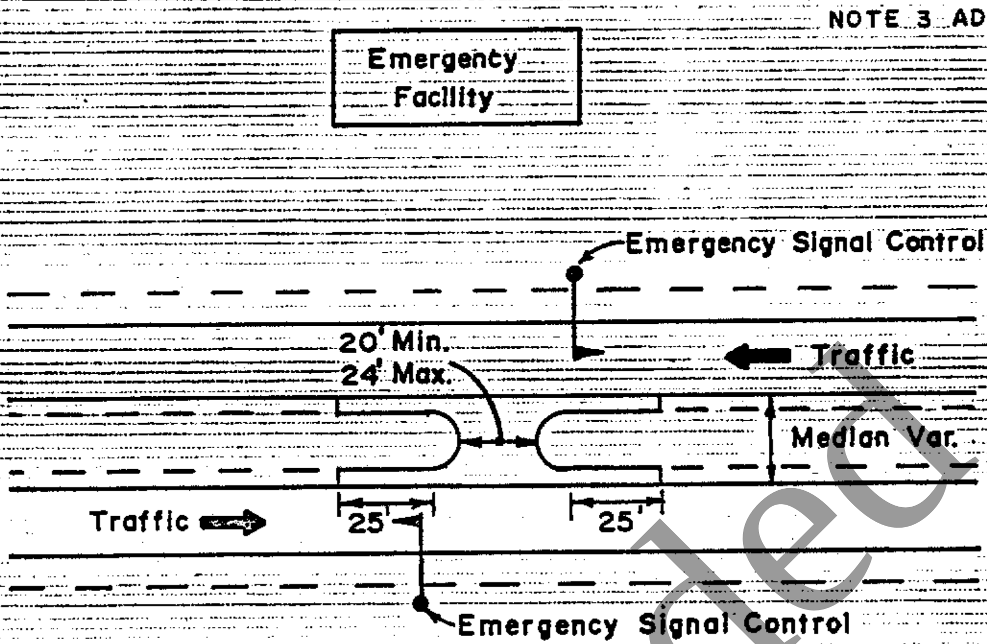
Figure 6-R shows a typical median left turn lane.

6-06.2 Lane Width

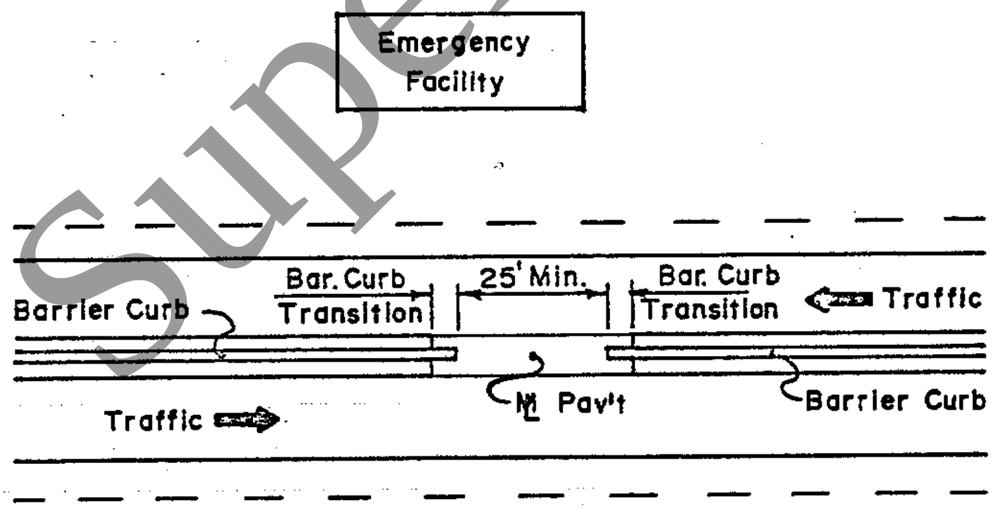
Left-turn lane should be 11 feet wide and desirably 14 feet wide. The lane width is measured from the curb face to the edge of thru lane.

EMERGENCY MEDIAN OPENINGS ON LAND-SERVICE ROADWAYS	FIQURE 6-P
	DATE: 8/79

NOTE 3 ADDED, 12/21/82 D.B.



Note: 1.) Grading to be done at 8% around median opening.
2.) If necessary, ponding water is to be eliminated by providing an "E" inlet in the median ℄, and connecting to existing drainage line.
3.) Adequate stopping sight distance must be available.



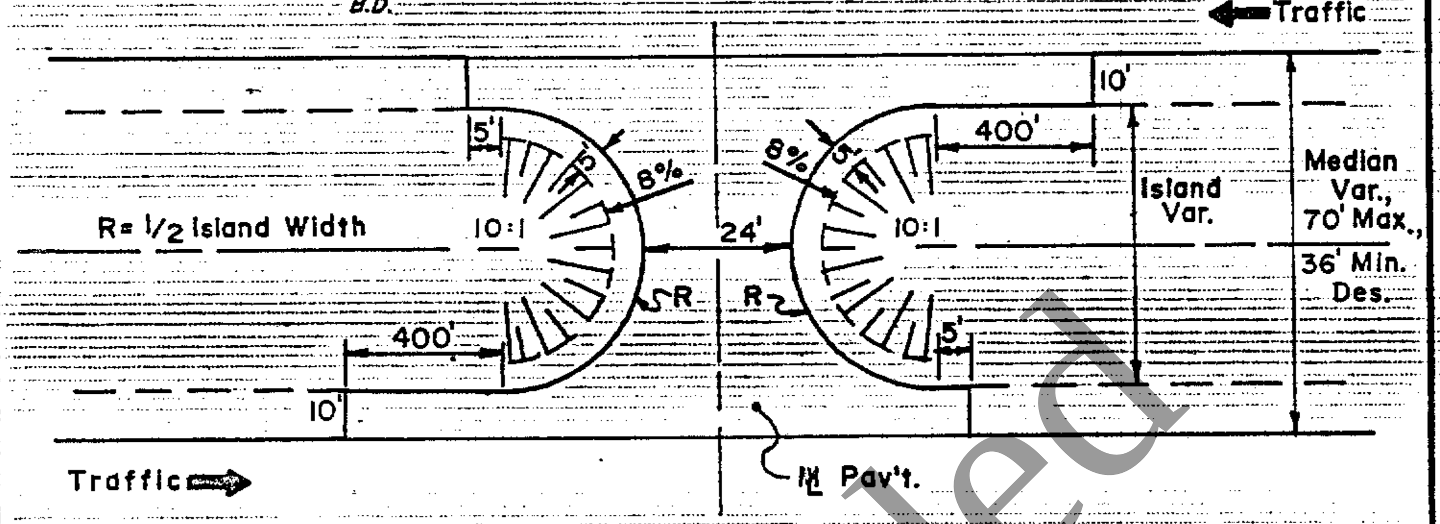
Note: 1.) Emergency Signal Control may be placed in Barrier Curb, see Standard Details, or outside the shoulder area, as above.
2.) Barrier Curb opening is a special case and requires an approval by the Chief of Design.
3.) Adequate stopping sight distance must be available.

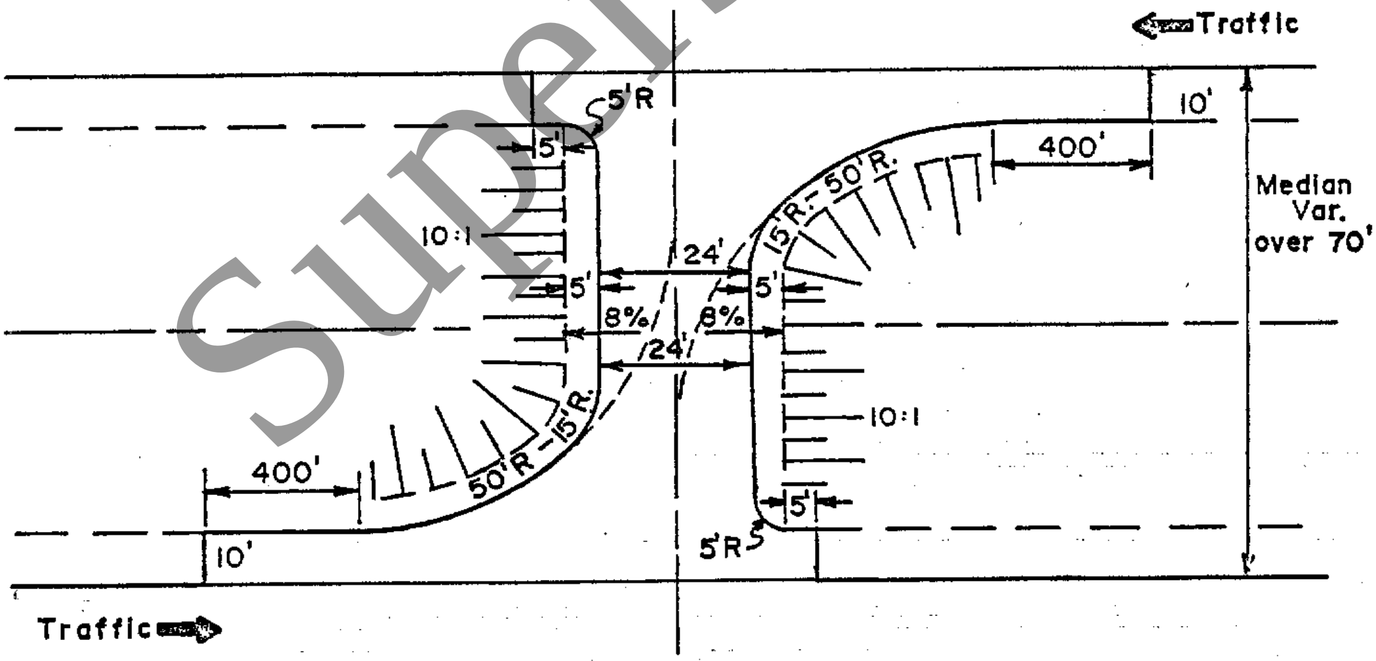
EMERGENCY MEDIAN OPENINGS ON INTERSTATE OR FREEWAY HIGHWAYS

FIQURE 6-Q

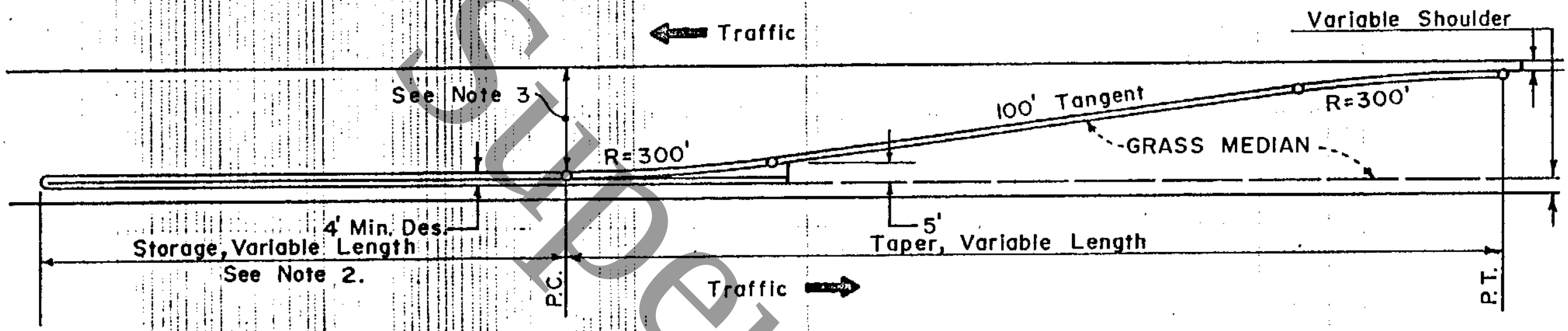
DATE: 9/79

DECEL. LENGTH CHANGED, 9/21/83 B.D. SLOPE CHANGED, NOTE 3 ADDED 12/27/82 D.B.





Note: 1.) Ponding of run-off is to be eliminated by conventional means.
2.) The median opening is to be located where adequate stopping sight distance may be provided.
3.) The median opening should be located 1/2 mile from any freeway underpass and at least one mile from any ramp terminal.



DESIGN GUIDELINES

1. MAXIMUM CURB SIZE: 9" x 18".
2. MINIMUM LENGTH: 100' FOR EACH 100 DHV (TURNING VEHICLES) AT UNSIGNALIZED INTERSECTIONS. FOR SIGNALIZED INTERSECTIONS, CHECK STORAGE LENGTH WITH TRAFFIC ENGINEERING.
3. LEFT TURN LANE WIDTH: 14' DESIRABLE, 11' MINIMUM.
4. THIS DESIGN IS INTENDED FOR USE WITH A PHYSICAL MEDIAN AND IS NOT TO BE EMPLOYED WITH PAINT STRIPES.
5. LEFT TURN LANE SHALL NOT BE CONSTRUCTED ADJACENT TO BARRIER CURB.

TYPICAL LEFT TURN SLOT

FIGURE 6-R
NOT TO SCALE

D.B. 12/30/82

Median widths of 20 to 25 feet or more are desirable at intersections with single left-turning lane, but widths of 15 feet to 18 feet are acceptable.

6-06.3 Length

The total length of the left-turn lane is the sum of storage length and entering taper.

1. 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.

2. Taper

   The entering taper treatment is illustrated in figure 6-R

6-07 CONTINUOUS TWO-WAY LEFT-TURN MEDIAN LANE

6-07.1 General

A continuous two-way left-turn median lane provides a common space for speed changes and storage for left-turn vehicles travelling in either direction and allows turning movements at any locations along a two-way roadway.

Continuous two-way left-turn median lanes are an effective means of providing an increased level of service on many urban arterials. They are especially effective in locations of strip commercial development and frequent driveway openings experiencing moderate left-turn demands.

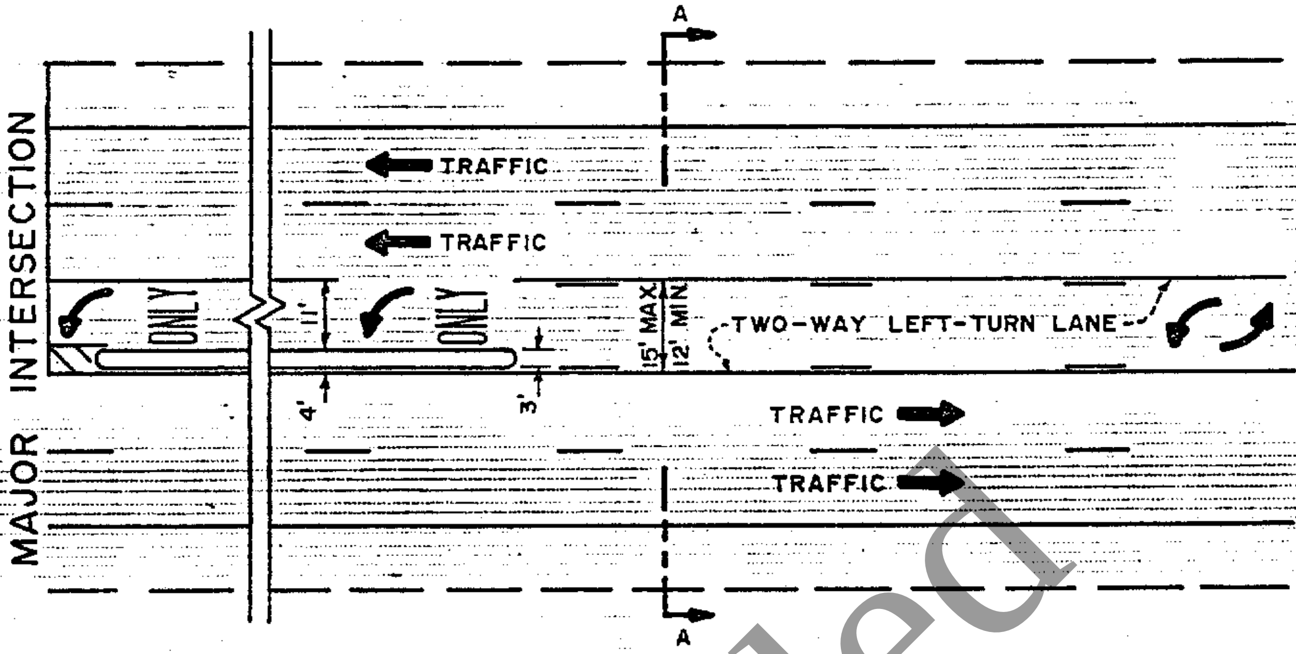
Figure 6-S shows a typical two-way left-turn median lane.

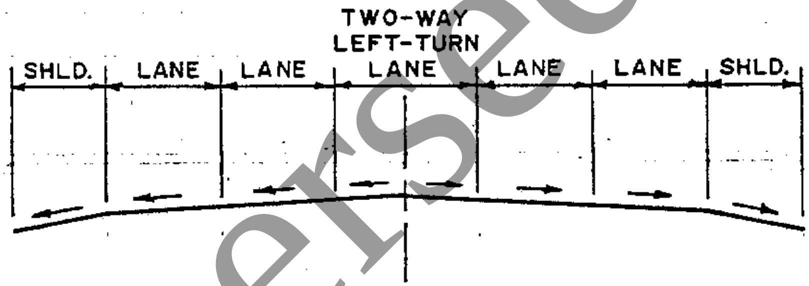
6-07.2 Lane Width

Lane widths for continuous two-way left turn median lanes range from 10 to 16 feet. The wider pavement width should be used only when raised islands are provided at major intersection with high left-turn demands. A median lane width of 12 feet is desirable where raised islands are not provided at major intersections.

6-07.3 Cross Slope

Generally the crown line should be located in the center of the median turn lane. The slope of pavement from the crown line should be the same as the cross slope on the thru lane adjacent to the median lane.





SECTION A-A

DESIGN GUIDELINES

1. PASSING SIGHT DISTANCE SHOULD BE AVAILABLE THROUGHOUT LENGTH OF TWO-WAY LEFT-TURN LANE.
2. FOR PROPER SIGNING AND PAINT STRIPING CONSULT THE "MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES", SECTIONS 3B-12 AND 2B-19.
3. TWO-WAY LEFT-TURN LANE MAY BE PROVIDED WHERE THERE IS ONLY ONE THRU LANE IN EACH DIRECTION. IT IS NOT RECOMMENDED WHERE THE NUMBER OF THRU LANES EXCEEDS TWO LANES IN EACH DIRECTION.
4. DIVISIONAL ISLAND USED ONLY WHEN MEDIAN WIDTH IS 15' MAX.

TWO-WAY LEFT-TURN LANE

FIGURE 6-S

NOT TO SCALE

D.F 1/13/83

6-08 JUGHANDLES

6-08.1 General

A "jughandle" is an at grade ramp provided at or between intersection to permit the motorists to make indirect left-turns or U-turns.

These ramps exit from the right lane of the highway in advance of the intersection, or past the intersection and convey traffic across the main highway under traffic signal control. This movement eliminates all turns within active traffic lanes and, in addition to providing greater safety, reduces delays to the through traffic that left turns usually create.

6-08.2 Ramp Width

Ramp widths are based on figure 7-B in SECTION 7. The minimum width for a one lane ramp should not be less than 22 feet. Ramps may have more than one lane when required to accomodate the anticipated traffic volumes.

6-08.3 Standard Jughandle Designs

Figures 6-T through 6-V illustrate the three basic jughandle designs.

6-09 OTHER CONSIDERATIONS

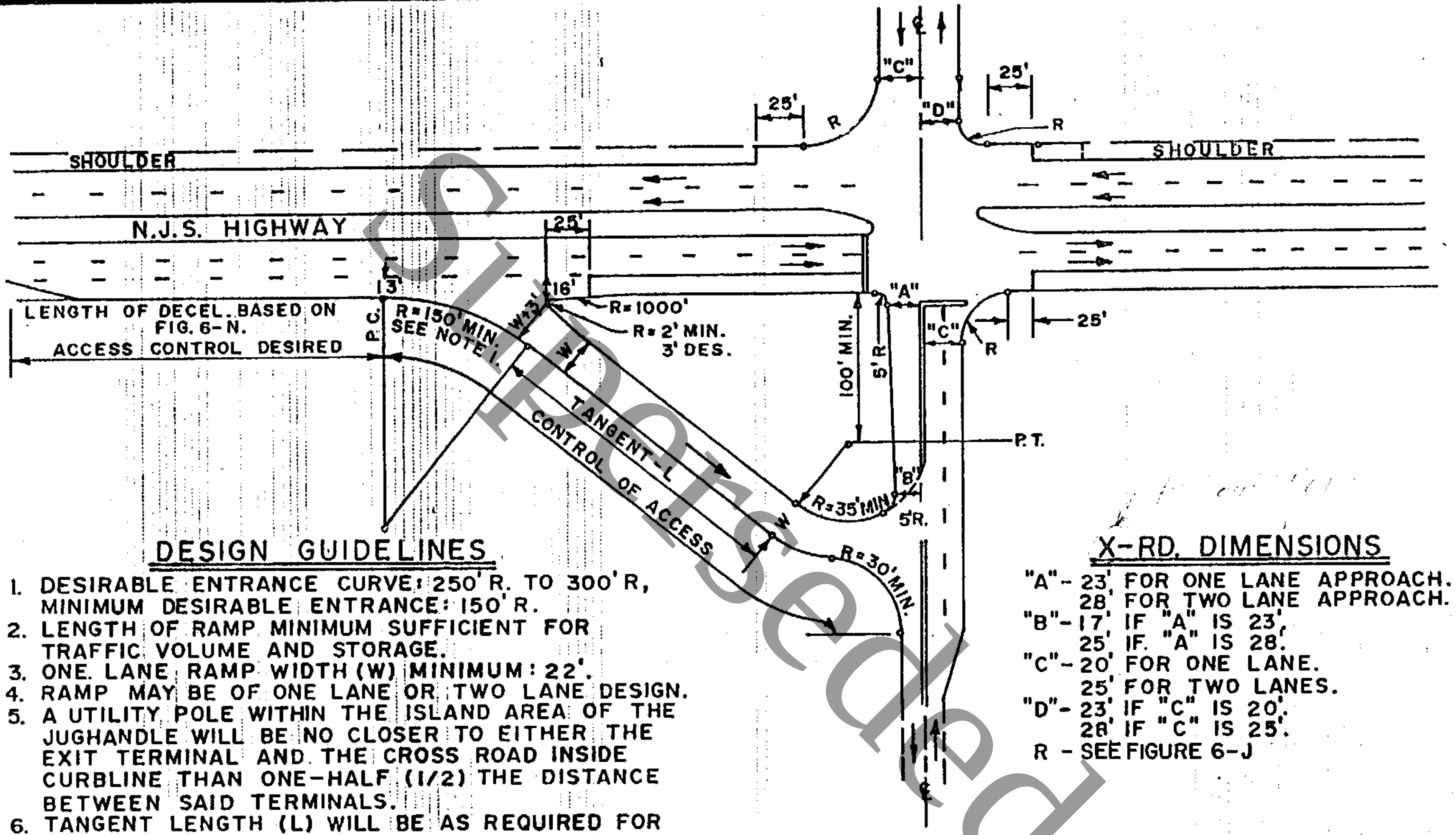
6-09.1 Parking Restrictions At Intersections

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

6-09.2 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 period.

Intersections with channelization, particularly with multiple-road goemetrics, 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



DESIGN GUIDELINES

1. DESIRABLE ENTRANCE CURVE: 250'R. TO 300'R, MINIMUM DESIRABLE ENTRANCE: 150'R.
2. LENGTH OF RAMP MINIMUM SUFFICIENT FOR TRAFFIC VOLUME AND STORAGE.
3. ONE LANE RAMP WIDTH (W) MINIMUM: 22'.
4. RAMP MAY BE OF ONE LANE OR TWO LANE DESIGN.
5. A UTILITY POLE WITHIN THE ISLAND AREA OF THE JUGHANDLE WILL BE NO CLOSER TO EITHER THE EXIT TERMINAL AND THE CROSS ROAD INSIDE CURBLINE THAN ONE-HALF (1/2) THE DISTANCE BETWEEN SAID TERMINALS.
6. TANGENT LENGTH (L) WILL BE AS REQUIRED FOR SUPERELEVATION TRANSITION.
7. CONTROL OF ACCESS SHOULD CONTINUE ALONG DECELERATION LANE WHERE POSSIBLE.

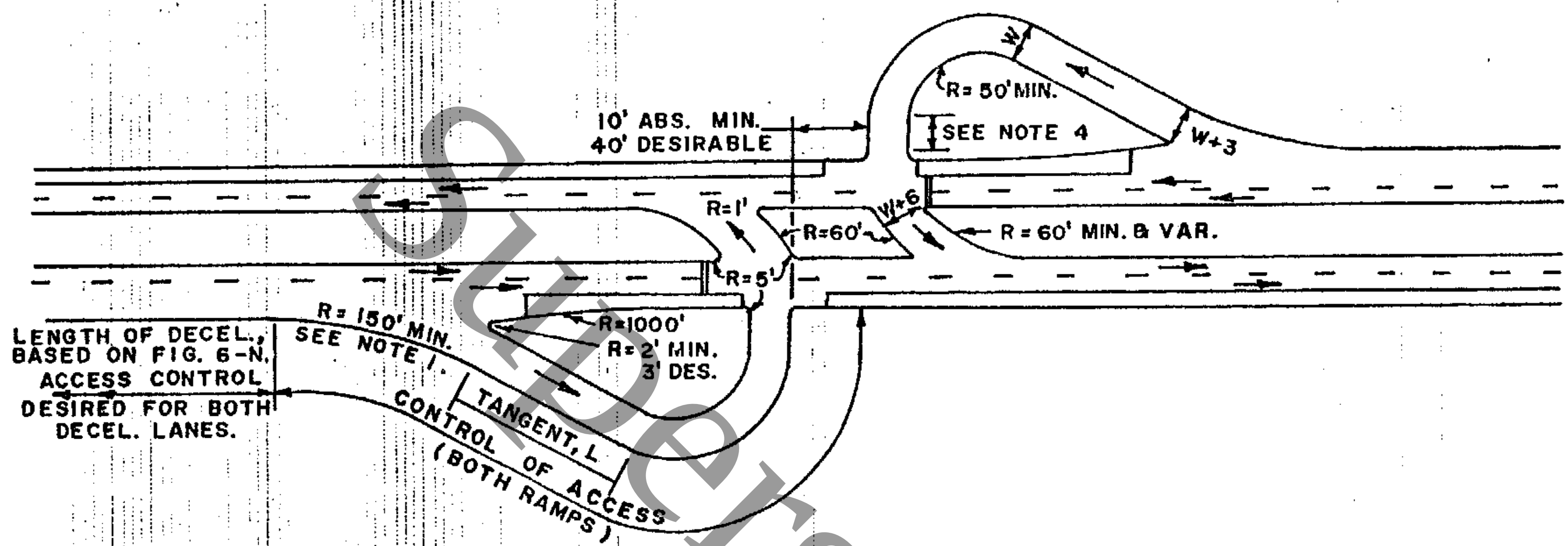
X-RD. DIMENSIONS

"A" - 23' FOR ONE LANE APPROACH.
28' FOR TWO LANE APPROACH.
"B" - 17' IF "A" IS 23',
25' IF "A" IS 28'.
"C" - 20' FOR ONE LANE.
25' FOR TWO LANES.
"D" - 23' IF "C" IS 20'.
28' IF "C" IS 25'.
R - SEE FIGURE 6-J

TYPICAL TYPE "A" JUGHANDLE

FIGURE 6-T
NOT TO SCALE

D.B. 11/12/82



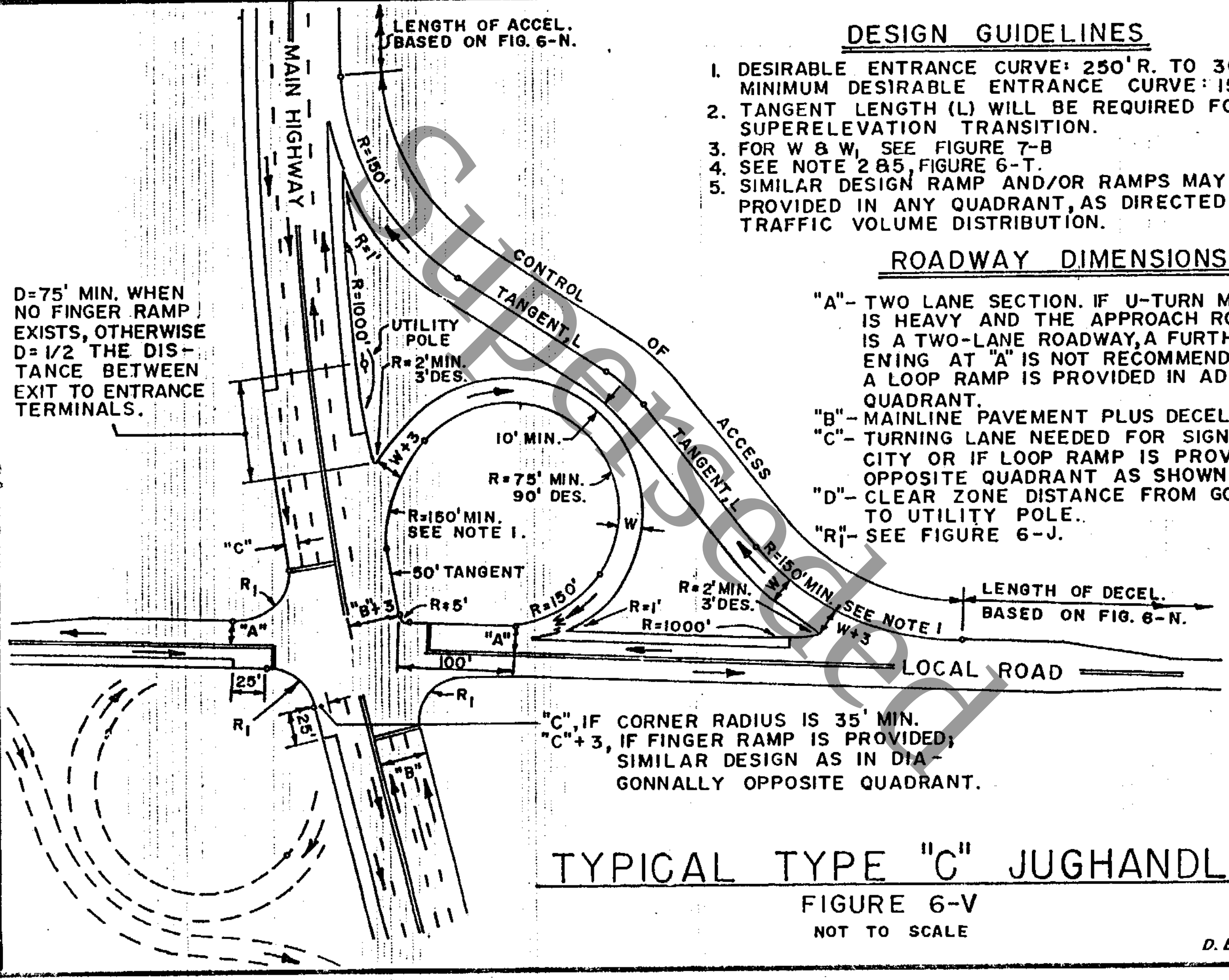
DESIGN GUIDELINES

1. DESIRABLE ENTRANCE CURVE: 250' R. TO 300' R., MINIMUM DESIRABLE ENTRANCE CURVE: 150' R.
2. LENGTH OF RAMP MINIMUM SUFFICIENT FOR TRAFFIC VOLUME AND STORAGE.
3. SEE NOTE 5, FIGURE 6-T.
4. TANGENT DISTANCE: 25' ABS. MINIMUM, 100' DESIRABLE.
5. RAMP MAY BE OF ONE LANE OR TWO LANE DESIGN.
6. TANGENT LENGTH (L) WILL BE AS REQUIRED FOR SUPER-ELEVATION TRANSITION.
7. CONTROL OF ACCESS SHOULD CONTINUE ALONG DECEL. LANE, WHERE POSSIBLE.

TYPICAL TYPE "B" JUGHANDLE

FIGURE 6-U
NOT TO SCALE

D.B. 11/15/82



DESIGN GUIDELINES

1. DESIRABLE ENTRANCE CURVE: 250' R. TO 300' R., MINIMUM DESIRABLE ENTRANCE CURVE: 150' R.
2. TANGENT LENGTH (L) WILL BE REQUIRED FOR SUPERELEVATION TRANSITION.
3. FOR W & W1 SEE FIGURE 7-B
4. SEE NOTE 2 & 5, FIGURE 6-T.
5. SIMILAR DESIGN RAMP AND/OR RAMPS MAY BE PROVIDED IN ANY QUADRANT, AS DIRECTED BY TRAFFIC VOLUME DISTRIBUTION.

ROADWAY DIMENSIONS

"A" - TWO LANE SECTION. IF U-TURN MOVEMENT IS HEAVY AND THE APPROACH ROADWAY IS A TWO-LANE ROADWAY, A FURTHER WIDENING AT "A" IS NOT RECOMMENDED UNLESS A LOOP RAMP IS PROVIDED IN ADJACENT QUADRANT.

"B" - MAINLINE PAVEMENT PLUS DECEL. LANE.

"C" - TURNING LANE NEEDED FOR SIGNAL CAPACITY OR IF LOOP RAMP IS PROVIDED IN OPPOSITE QUADRANT AS SHOWN DASHED.

"D" - CLEAR ZONE DISTANCE FROM GORE AREA TO UTILITY POLE.

"R1" - SEE FIGURE 6-J.

TYPICAL TYPE "C" JUGHANDLE

FIGURE 6-V

NOT TO SCALE

D.B. 11/18/82

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.

INTERCHANGES

7-01 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.

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations, providing for the movement of traffic between two or more roadways on different levels. Safety and traffic capacity are increased by the provision of traffic interchange. Crossing conflicts are eliminated by grade separations. Turning conflicts are either eliminated or minimized, depending upon the type of interchange design.

7-02 WARRANTS FOR INTERCHANGES

7-02.1 Freeways & Interstate Highways

Interchanges should be provided on Interstate highways and freeways at all intersections where access is to be permitted. Other intersecting roads or streets are either grade separated, terminated, or rerouted.

7-02.2 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 safety at such intersections cannot be improved by more inexpensive methods, construction of an interchange facility may be warranted.

3. Site Topography

At some sites, the topographic conditions may be such that the provisions 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 satisfactorily serve, without undue delay and congestion, the traffic volumes and turning movements expected.

7-03 INTERCHANGE TYPES

7-03.1 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.

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.

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 basis types.

7-04 INTERCHANGE DESIGN ELEMENTS

7-04.1 General

Geometric design for all interchange roadways should follow the design guides as covered in SECTION 4, BASIC GEOMETRIC DESIGN ELEMENTS.

7-04.2 Spacing

The minimum spacing of interchanges for proper signing on the main road should be at least one mile between urban crossroads and three miles along rural sections. Closely spaced interchanges interfere with free traffic flow and safety, even with the addition of extra lanes, because of insufficient distance for weaving maneuvers. During the early design stage, the Bureau of Traffic Engineering should be consulted to assure that proper signing of the interchange is possible.

7-04.3 Sight Distance

Sight distance along the through roadways and all ramps should be at least equal to the minimum safe stopping sight distance and preferrably longer for the applicable design speed. See Section 4 & 6 for sight distance requirements.

7-04.4 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.

7-04.5 Ramps

1. General

The term "ramp includes all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange. The components of a ramp are a terminal at each end and a connecting road, usually with some curvature, and on a grade. Ramps are one way roadways.

2. Ramp Capacity

The capacity of a ramp is generally controlled by one of its terminals. Occasionally the ramp proper determines the capacity, particularly where speeds may be significantly affected by curvature, grades, and truck operations. Figure 7-A shows the basic values (Service Volumes) for the ramp proper on single lane ramps.

CAPACITY OF RAMP PROPER

FIGURE : 7-A

DATE : 11/83

Single-Lane Operation

DESIGN CONDITION	T %TRUCKS DURING PEAK HOUR	Design Speed V ≤ 20 mph R=90' min - 125' Des. Rate of Upgrade %			Design Speed V = 25 mph R = 150' Rate of Upgrade %			Design Speed V = 30-40 R = 230'-430' Rate of Upgrade %			Design Speed V ≥ 50 R = 690' Rate of Upgrade%		
		0-2	3-4	≥ 5	0-2	3-4	≥ 5	0-2	3-4	≥ 5	0-2	3-4	≥ 5
SERVICE LEVEL B	0	800	800	800	1000	1000	1000	1100	1100	1100	1220	1220	1220
	5	760	720	700	950	900	870	1050	1000	950	1140	1090	1040
	10	720	670	610	910	830	770	1000	920	850	1090	1000	920
	20	670	570	500	830	720	620	920	780	690	1000	860	750
	30	610	500	420	770	620	530	850	690	580	920	750	630
SERVICE LEVEL C	0	1000	1000	1000	1250	1250	1250	1400	1400	1400	1500	1500	1500
	5	950	900	870	1190	1140	1090	1330	1270	1220	1420	1360	1300
	10	910	830	770	1140	1040	960	1270	1170	1080	1360	1250	1150
	20	830	720	620	1040	890	780	1170	1000	870	1250	1070	940
	30	770	620	530	960	780	660	1080	880	740	1150	940	790

Adapted from FHWA Report on "Capacity Analysis for Design and Operation of Freeway Facilities" 1974

NOTES:

1. For 2 lane ramps multiply tabular values as follows: 1.7 for 20 mph or less, 1.8 for 25 mph, 1.9 for 30-40 mph, 2.0 for 50 mph or more.
2. For down grades, use same values as for 0-2% upgrade.
3. To approximate level of service E, multiply above values by 1.25.
4. Minimum ramp radius on interstate highways should not be less than 150'.

3. Design Speed

It is not practical to provide design speeds on ramps that are comparable to those on the through roadways. Ramp design speeds however should not be less than 25 mph. On cloverleaf interchanges, the outer connections should desirably be designed for 35 mph.

Minimum ramp design speeds for various ramp configurations are as follows: Loop ramps, 25 mph; semidirect, 30; and direct connections, 40 mph.

4. Grades

Ramp grades should be as flat as feasible to minimize driving effort required in maneuvering from one road to another. The following guidelines should be used in designing ramp profiles:

1. Ramp gradients should be limited to a maximum upgrade of 7% (desirable maximum of 5%) and 5% on downgrades.

2. Minimum ramp grades should not be less than 0.5%.

3. When the ramp is to be used predominately by truck traffic (many heavy trucks), upgrades should be limited to 4%.

5. Sight Distance

On ramps, no planting of vegetation that would restrict the sight distance to less than the minimum for the applicable design speed shall be permitted.

6. Ramp Widths

Table 7-B illustrates the desired ramp widths for various ramp curvatures. Single lane ramp widths will be based on Case II for the ramp proper and Case I at the entrance terminal. Case III should be used in determining ramp widths on two lane ramps. See SECTION 5, Figure 5-J for typical single and two lane ramp sections.

7. 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.

DESIGN WIDTHS OF PAVEMENT FOR TURNING ROADWAYS

FIGURE 7-B

DATE 11/83

R Radius on Inner Edge of Pavement, Feet	(w) Pavement width in feet for: Case I Entrance Terminal Width	Case II Ramp Proper Width 1-Lane, One Way Operation	Case III Ramp Proper Width 2-Lane, One Way or two-way Operation
50	20	26	-
75	19	24	-
100	18	23	Note 4
150	18	22	32
200	18	22	31
300	17	22	30
400	17	22	30
500	17	22	30
Tangent	17	22	39

Note: 1. Ramps widths are applicable for ramps with or without curb.

Note: 2. Minimum ramp radii will be used to determine ramp width. Width will be applied through entire ramp except at the ramp terminals.

Note: 3. On 2-lane ramps where shoulders 4 ft. or wider are provided, reduce ramp pavement width by 4 ft.

Note: 4. 2-lane operation should not be considered on ramps with radii less than 150'.

For left 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. See SECTION 6 for sight distance at intersections.

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.

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 a local parallel two-way street should also be discouraged because of limited sight distance usually encountered at the merge with the local street traffic.

7-05 SUPERELEVATION FOR RAMPS

The factors controlling superelevation rates for main line conditions discussed in SECTION 4 apply also to ramps. Standard superelevation with maximum rate of 6% should be provided on ramp curves. Ramp alignment which precludes the attainment of standard superelevation within a reasonable transition distance should be avoided.

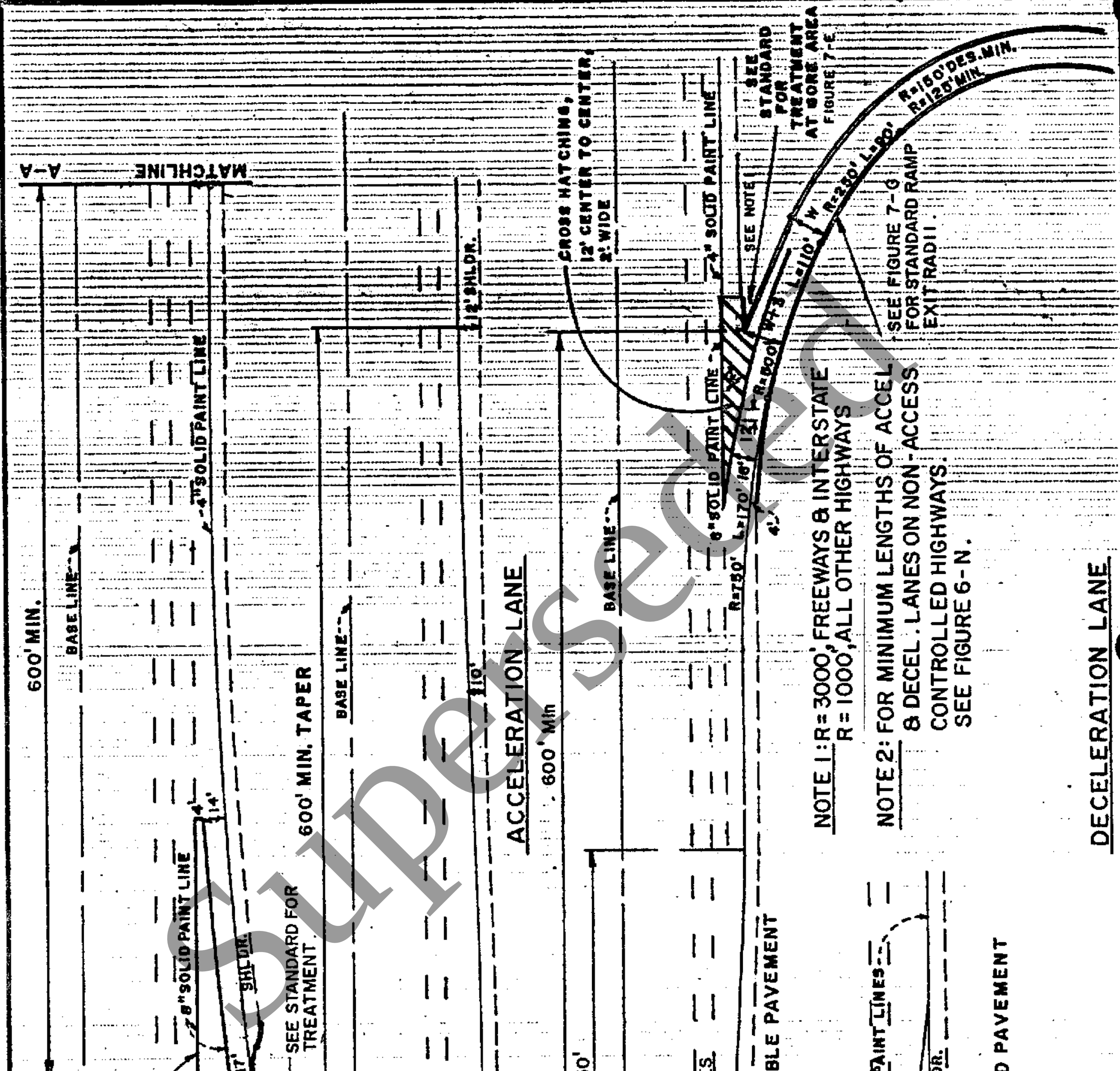
Exceptions to the use of the full superelevation are at street intersections where a stop or reduced speed condition is in effect and, under some conditions, at ramp junctions. Edge of pavement profiles should be drawn at ramp junctions to assure a smooth transition.

7-06 FREEWAY ENTRANCES AND EXITS

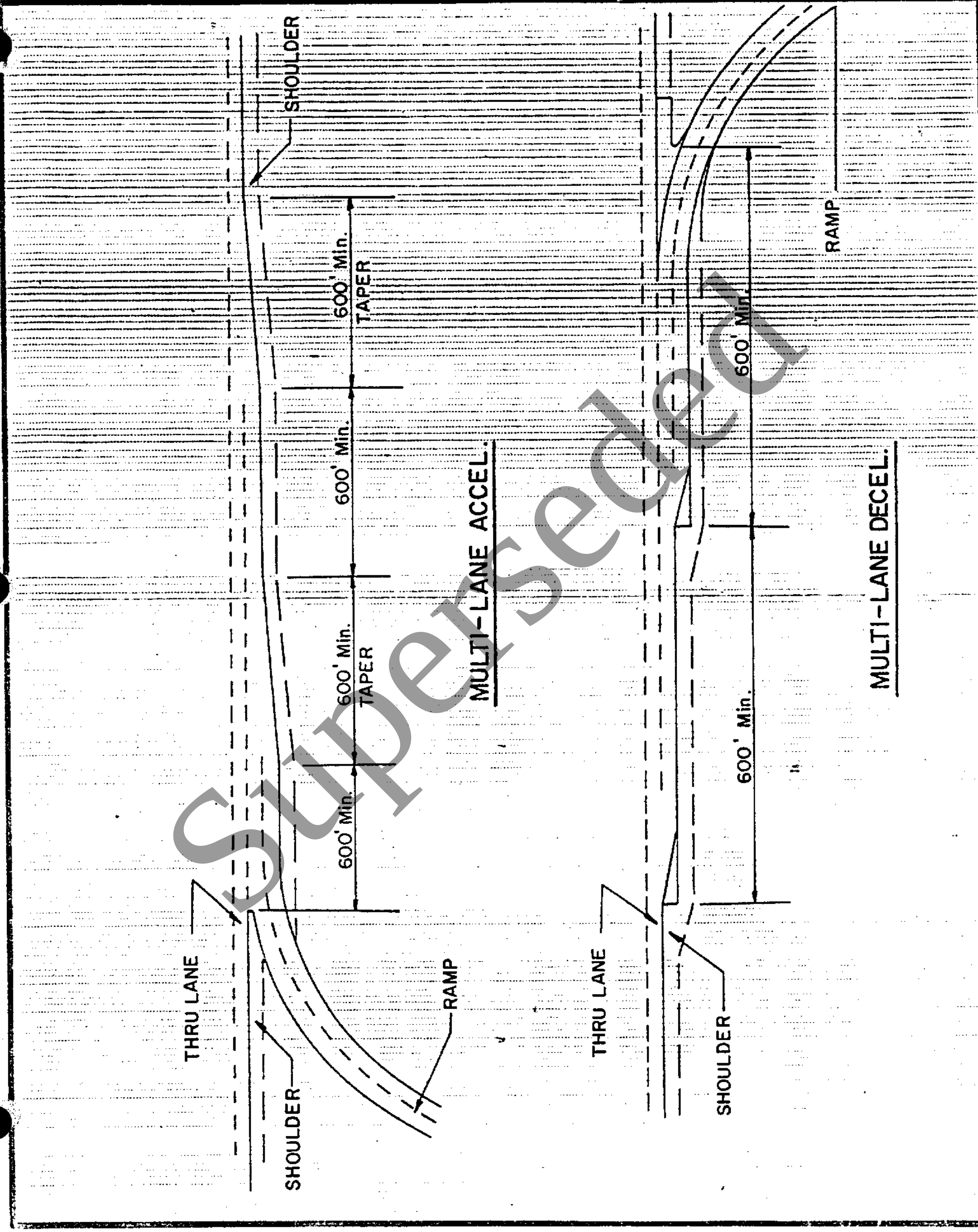
7-06.1 Basic Policy

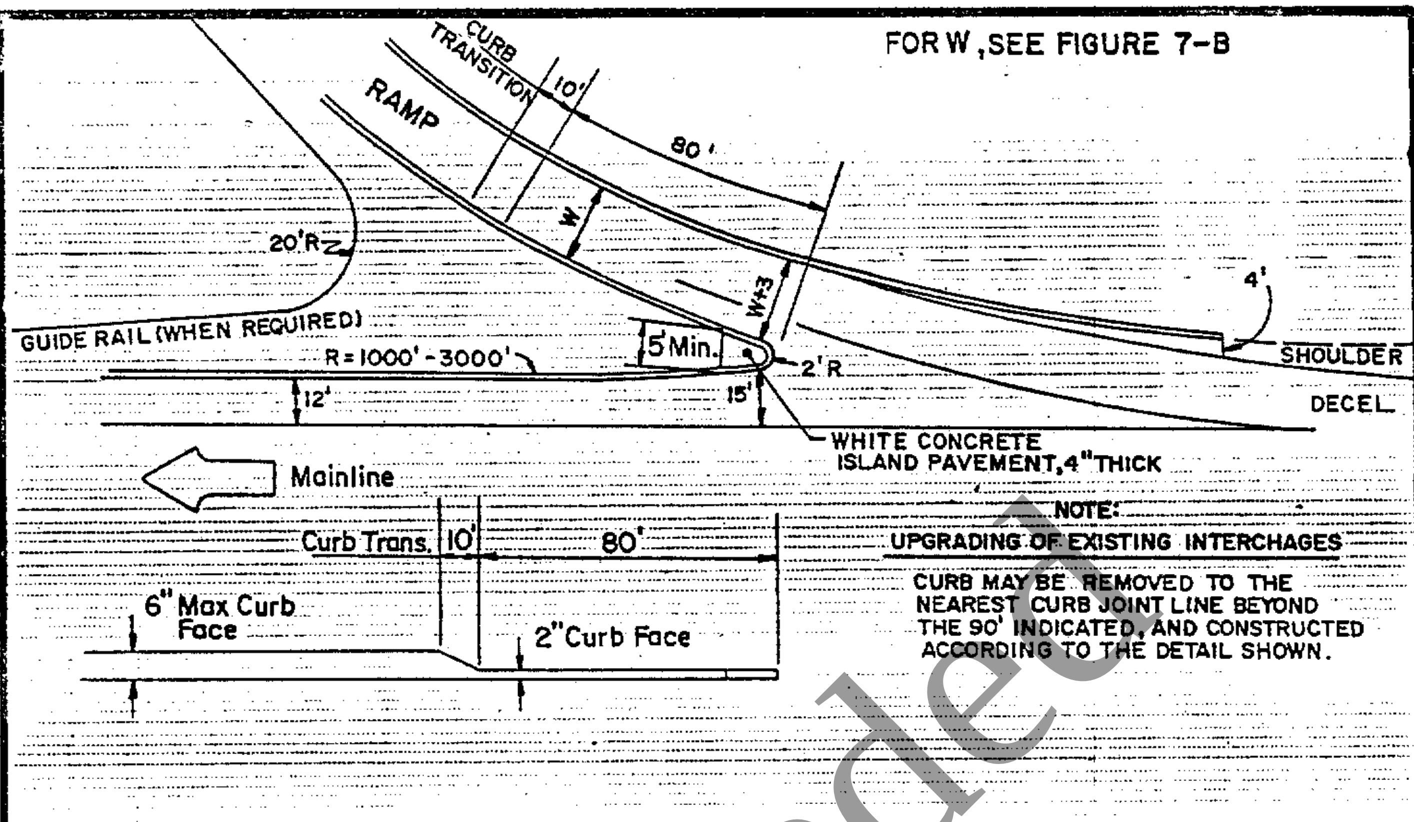
Desirably all interchange entrances and exits should connect at the right of through traffic. Freeway entrances and exits should be located on tangent sections where possible in order to provide maximum sight distance and optimum traffic operation.

INTERSTATE AND FREEWAY RAMP TERMINAL TREATMENT SINGLE LANE RAMP	FIGURE-7-C
	DATE: 9/79

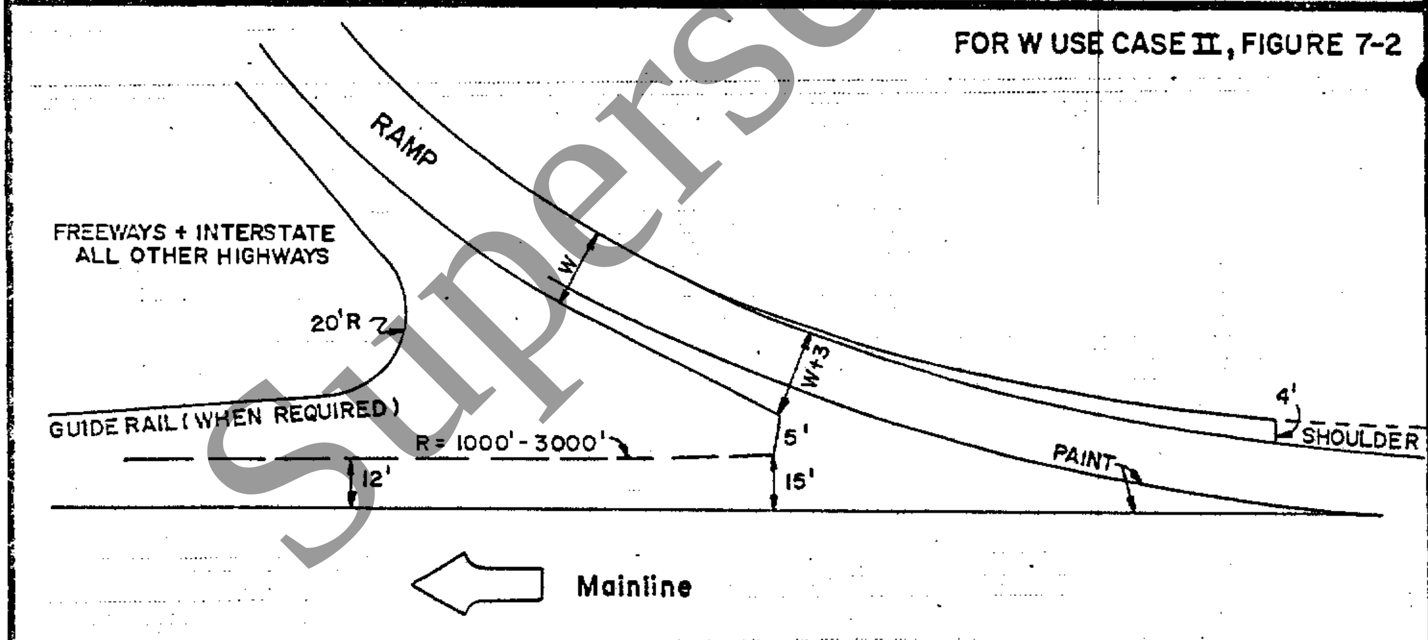


INTERSTATE AND FREEWAY RAMP TERMINAL TREATMENT MULTI-LANE RAMP	FIGURE-7-D
	DATE: 10/83





EXIT TERMINAL-CURBED



EXIST TERMINAL - NO CURB

TREATMENT AT GORE AREAS

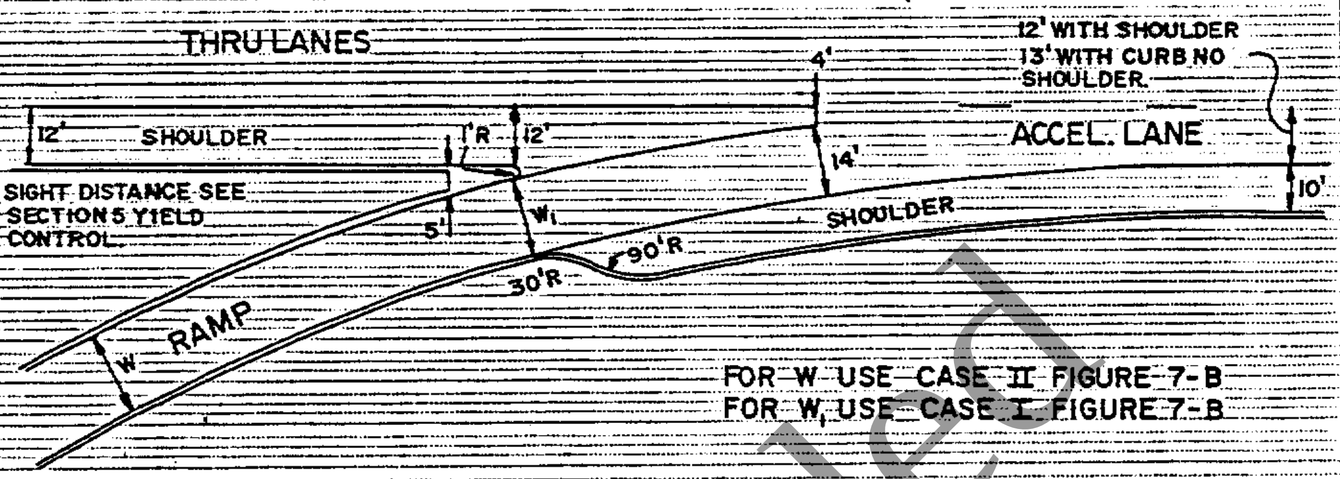
FIGURE 7-E

DATE 10/83

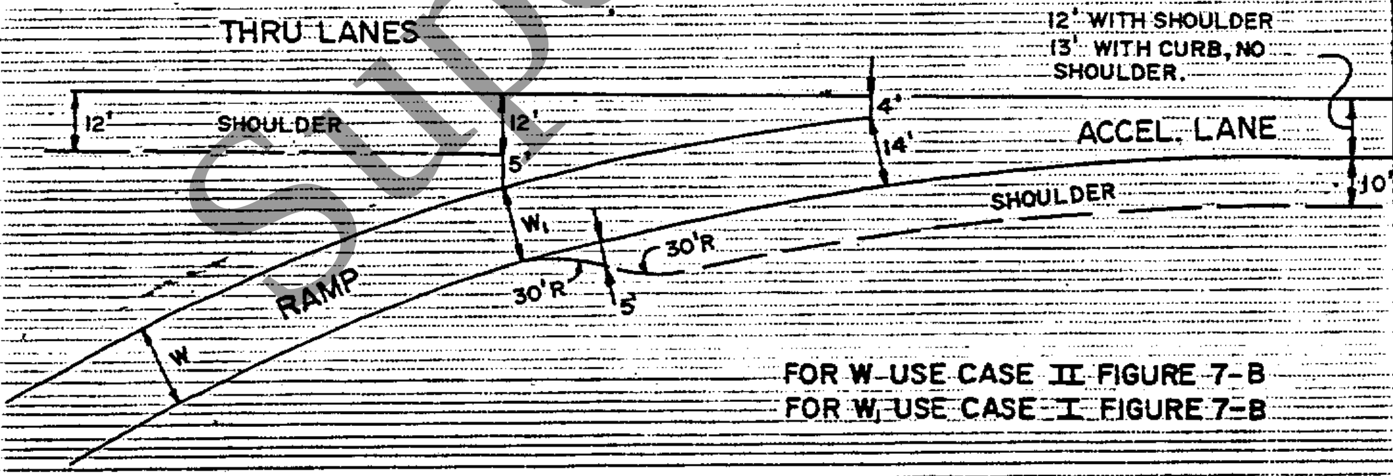
ENTRANCE TERMINAL TREATMENT

FIGURE 7-F

DATE 10/83

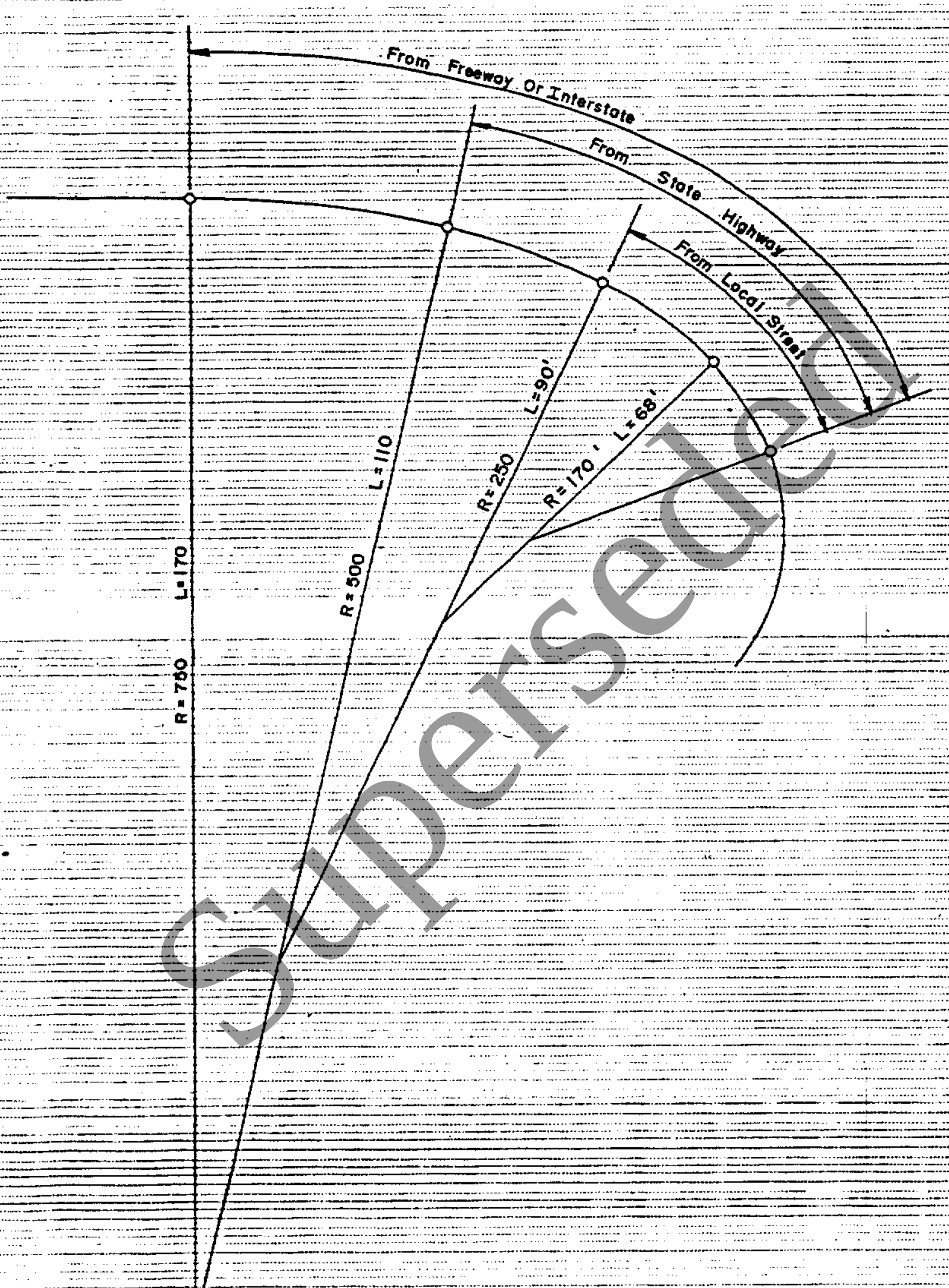


ENTRANCE TERMINAL- CURBED



ENTRANCE TERMINAL - NO CURB

MINIMUM EXIT RAMP RADII	FIGURE : 7-G
	DATE : 11/83



7-06.2 Ramp Terminals

The ramp terminal is the portion of the ramp adjacent to the through lanes and includes the speed change lanes, tapers, gore areas, and merging ends. Figures 7-C through 7-F illustrate the various ramp terminal treatments.

7-06.3 Distance Between Successive Exits

At interchanges there are frequently two or more ramp terminals in close proximity along the through lanes. In some interchange designs, ramps split into two separate ramps or combine into one ramp. Minimum and desirable distances between successive ramp terminals are shown in Figure 7-H.

7-06.4 Speed Change Lanes

The minimum length of speed change lanes on Freeways and Interstate highways are shown in Figures 7-C and 7-D. The lengths should be increased when the upgrade exceeds 3% on accelleration lanes and on decelleration lanes when the downgrade exceeds 3%. The 1965 AASHTO publication "A Policy and Geometric Design of Rural Highways" lists the ratio of length of speed-change lane on grade to length on level.

7-06.5 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.

7-07 ADDITIONAL LANES

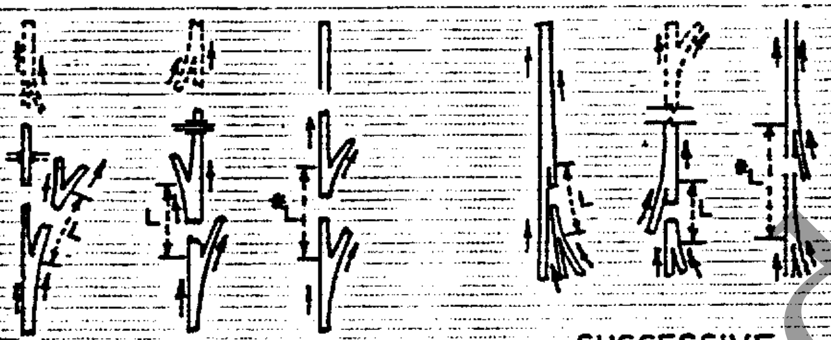
In order to ensure satisfactory operating conditions, additional lanes may be added to the basic width of traveled way.

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. This should be the general practice where the weaving distance is less than 2000 feet. Where interchanges are more widely spaced and ramp volumes are high, the need for an additional 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.

ARRANGEMENTS FOR SUCCESSIVE RAMP TERMINALS

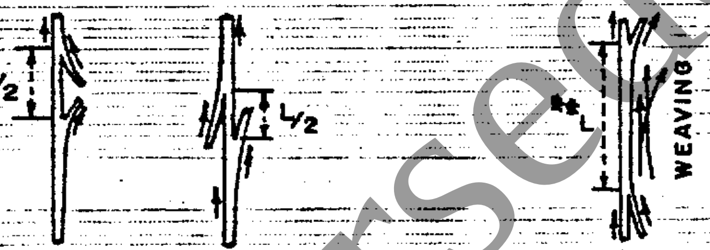
FIGURE 7-H

DATE: 10/83



SUCCESSIVE EXIT TERMINALS

SUCCESSIVE ENTRANCE TERMINALS



EXIT TERMINAL FOLLOWED BY ENTRANCE TERMINAL

ENTRANCE TERMINAL FOLLOWED BY EXIT TERMINAL

*L as in table but not less than length required for accel. or decel. lanes.

**L as in table but not less than length required for weaving.

DISTANCE BETWEEN SUCCESSIVE RAMP TERMINALS

Design speed, mph	30 or less	40 to 50	60 to 70	80
Av. running speed, mph	23 to 28	36 to 44	52 to 58	64
Distance L-Feet				
Minimum	200	400	500	900
Desirable	400	700	900	1200

Source: A Policy on Geometric Design of Rural Highways: AASHTO 1965

7-08 LANE REDUCTION

Lane reduction below the basic number of lanes is not permissible through an interchange. 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 entrance. It is desirable to locate lane drops on tangent alignment with a straight or sag profile so that there is maximum visibility to the pavement markings in the merge area.

7-09 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.

7-10 WEAVING SECTIONS

Weaving is created by vehicles entering and leaving the highway at common points, resulting in vehicle paths crossing each other. Weaving normally occurs within an interchange or between closely spaced interchanges.

Desirably on cloverleaf interchanges the distance between loop ramp terminals should not exceed 800-1000 ft. Where the weaving volumes require separations greater than the desirable, consideration should be given to providing a collector distributor road.

The "Highway Capacity Manual" should be consulted for further information on weaving.

7-11 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 desirably extend to the end of the ramp taper, or as a minimum 100 feet in urban areas and about 300 feet or more in rural areas beyond the end of the curb return or ramp radius, as the case may be.

Since site conditions are too varied, the distances suggested are to be used as general guides rather than specific maximum or minimum values.

SECTION 8

GUIDELINES FOR GUIDE RAIL DESIGN AND MEDIAN BARRIERS

8-01 INTRODUCTION

These guidelines are based on the 1977 AASHTO Guide for Selecting, Locating and Designing Traffic Barriers.

They are intended to serve as guidelines which will assist the designer in determining conditions which warrant the installation of guide rail as well as the dimensional characteristics of the installations.

It is important that application of these guidelines be made in conjunction with engineering judgement and thorough evaluation of site conditions so that a proper solution is arrived at.

It should be emphasized that guide rail should not be installed indiscriminately. Every effort should be made to eliminate the obstruction for which the guide rail is being considered.

In some cases,another type of traffic barrier, might be a better choice than guide rail. For example, obstructions in gores can often be more effectively shielded with a crash cushion. The designer should consider such alternatives and choose the most suitable solution based on safety requirements, economic limitations, maintenance, and aesthetic considerations.

8-02 GUIDE RAIL WARRANTS

8-02.1 General

Guide rail is considered a longitudinal barrier whose primary functions are to prevent penetration and to safely redirect an errant vehicle away from a roadside or median hazard.

8-02.2 How Warrants are Determined

The physical characteristics of the obstruction and its distance from the edge of the traveled way are the basic factors to be considered in determining if guide rail is warranted.

Although a wide range of roadside conditions are covered below, special cases will arise for which there is no clear choice as to whether or not guide rail is warranted. Such cases must be evaluated on an individual basis, and, in the final analysis must usually be solved by engineering judgement. In the absence of pertinent criteria, a cost effective analysis could be used to evaluate guide rail needs.

8-02.3 Definition of Warranting Obstruction

A warranting obstruction is defined as a fixed object or nontraversable hazard whose physical characteristics are such that injuries resulting from an impact with the obstruction would probably be more severe than injuries resulting from an impact with guide rail.

1. Fixed Objects

Examples of fixed objects which may warrant guide rail are: signs, traffic signals and luminaire supports of nonbreakaway design; concrete pedestals extending more than 4 inches above the ground; bridge piers, abutments and ends of parapets and railings; trees, 6 inches or more in diameter; wood poles or posts with a cross-sectional area greater than 50 square inches.

a. Trees

From a safety viewpoint, there should not be any trees within the clear zone. For discussion on clear zone see 8-02.4.

On freeways and interstate routes, it would be difficult to justify exceptions to such a policy. In some cases it may be appropriate to begin planting of replacement trees back from the highway so that the ultimate removal of trees in close proximity to the roadway may be accomplished without severe public criticism. However, on land service roads, it is likely that situations will be encountered where removal of trees within the clear zone cannot be accomplished. For instance, the aesthetic appeal of the trees may cause local opposition to their removal, the trees may not be within the ROW, or removal of the trees may not be environmentally acceptable.

Factors such as accident experience, traffic volume, speed, clearance to the traveled way and roadway geometry should be evaluated when determining whether it is appropriate to leave trees within the clear zone or install guide rail if the trees are to remain.

b. Utility poles

Although utility poles have a cross-sectional area greater than 50 square inches, they should not be handled the same as other warranting obstructions. It is questionable whether a safer roadside would result from installing guide rail for utility poles within the clear zone. The expected increase in severity must be carefully weighed including consideration of such factors as accident experience, traffic volume, operating speed, clearance to the traveled way, and roadway geometry. Ordinarily, an acceptable solution is to locate the poles as far from the traveled way as possible without guide rail. In no case, shall utility poles on new guide rail installations remain in front of the guide rail.

Where utility poles are placed behind guide rail, desirably they should be four (4) feet or greater from the face of rail. However as a minimum, the pole should be no closer than nine (9) inches from the face of rail.

2. Nontraversable Hazards

Examples of nontraversable hazards which may warrant guide rail are: rough rock cuts; large boulders; streams or permanent bodies of water more than 2 feet in depth; roadside ditches with slopes steeper than 1:1 and depths greater than 2 feet; and embankment slopes as described below.

a. Embankment (fill) Slopes	Maximum Height
1½:1	3'-0
2:1	6'-0
2½:1	9'-0
3:1 and flatter	Guide rail not required unless obstructions are within the clear zone. See Figure 8-A for clear distance.

Return maneuvers can not be accomplished on 3:1 slopes. A vehicle can be expected to travel to the bottom of the slope before steering recovery can be obtained. The designer should, therefore, evaluate each site before providing 3:1 slopes without guide rail.

When flattening existing slopes to remove guide rail, the proposed side slopes should be 4:1 or flatter. Rounding should be provided at slope breaks, See Figures 5-C, 5-H and 5-I.

b. Slopes in Cut Sections

Slopes in cut sections should not ordinarily be shielded with guide rail. However, their potential hazard should be recognized.

Slopes in cut section of 2:1 or flatter may be considered traversable and, as the cut slopes steepens, the chance of rollover increases. Where feasible, slopes steeper than 2:1 should be flattened. If there is a warranting obstruction on the cut slope, the following apply:

(1) Guide rail should be installed if the warranting obstruction is on a slope flatter than 0.7:1 and is within the clear zone width specified in section 8-02.4

(2) Guide rail should be installed if the warranting obstruction is on a slope of 0.7:1 or steeper and is less than 6 feet (measured along the slope) from the toe of slope and is within the clear zone width specified in section 8-02.4

(3) Guide rail is not required if the warranting obstruction is on a slope of 0.7:1 or steeper and is 6 feet or more (measured along the slope from the toe of slope.

See Figures 8-G and 8-H for an example of a guide rail installation in a cut section.

c. Ditches

Ditches should be designed to be traversable. Where feasible, existing ditches should be reconstructed so as to be traversable.

The 1977 AASHTO Guide for Selecting, Locating and Designing Traffic Barriers shows criteria for preferred ditch cross sections.

8-02.4 Clear Zone

Clear zone is defined as the roadside or border area, starting at the edge of the travelled way, available for safe use by errant vehicles.

The width of the clear zone (Lc) varies with the speed and roadside slope. Desirably the design speed should be used when determining the clear zone on either new or existing highways. As a minimum, the posted speed plus 5 mph may be used when determining the clear zone width on existing highways.

Figure 8-A indicates the clear zone widths for various speeds and roadside slopes. Figure 8-B illustrates the methods by which the clear zone should be adjusted for volumes less than 6000 ADT and/or for speeds other than those shown in figure 8-A.

Horizontal alignment does affect the clear zone width. Clear zone widths on the outside of horizontal curves can be determined as shown in figure 8-B.

8-03 DIMENSIONAL CHARACTERISTICS

8-03.1 Approach Length of Need

The length of "approach" guide rail should be determined in accordance with Figures 8-C, 8-D or 8-E. On a two-way, undivided highway or on a divided highway with a narrow traversable median, an "approach end" treatment may be required for both directions of traffic (See Figure 8-E). However the designers first consideration should be to remove or relocate the roadside obstacle so that a longitudinal barrier is unnecessary.

See figure 8-G for an example of determining length of need in a cut section.

8-03.2 End Treatments

1. Beam Guide Rail Anchorage

   On a one-way roadway or a divided roadway with a nontraversable median, trailing ends of guide rail installations should be anchored with a Beam Guide Rail Anchorage, Type I, Figure 8-I.

   In cut sections, the approach ends of guide rail installations should be anchored with a Beam Guide Rail In Line Anchorage, Type II and buried in the slope as shown on Figure 8-J.

   In special cases, where the end of a guide rail installation is located so that an end hit is unlikely, the end should be anchored with a Beam Guide Rail In Line Anchorage, Type II.

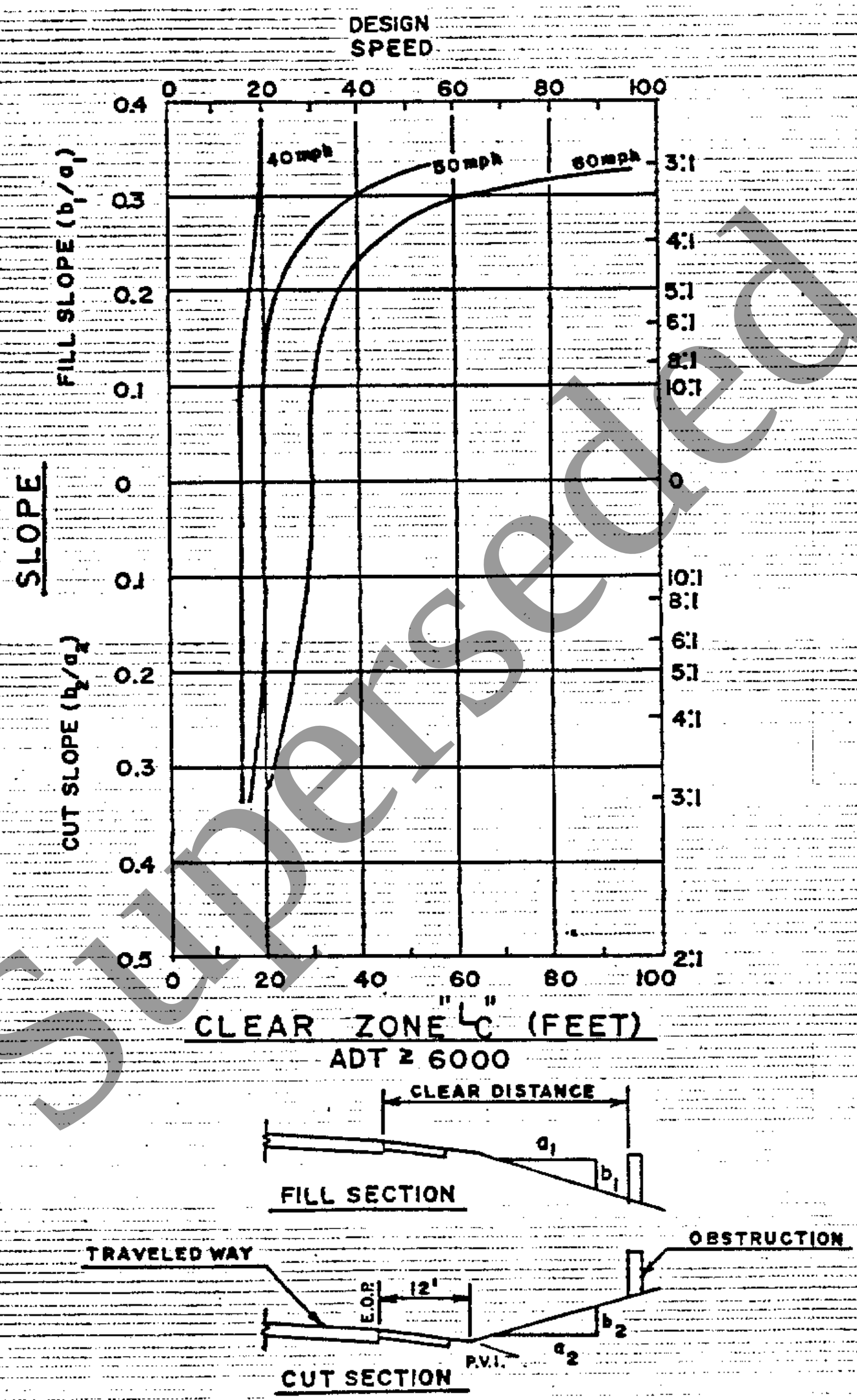
2. Breakaway Cable Terminal (BCT)

   Breakaway Cable Terminals should be used at ends of beam guide rail installations terminating within the clear zone, unless covered by conditions noted in 8-03.2.1 above. A BCT shall not be installed behind curb greater than 6 inches in height unless it is located beyond the clear distance.

CLEAR ZONE

FIGURE- 8-A

DATE: 11/83



Source: 1977 ASSHTO Guide For Selecting, Locating, and Designing Traffic Barriers.

CLEAR ZONE ADJUSTMENTS

FIGURE: 8-B

DATE: 11/83

1. CLEAR ZONE (L_c) FOR SPEEDS OTHER THAN 40, 50 or 60

The clear zone width for a speed of 70 mph can be determined as follows:

L_c (60 mph) = 30 ft. from Figure 8-A for a 10:1 or flatter roadside
L_r (60 mph) = 400 ft. from Table 1, Figure 8-C
L_r (70 mph) = 480 ft. from Table 1, Figure 8-C

$$L_c \text{ (70 mph)} = \frac{480}{400} \times 30 = 36 \text{ ft.}$$

2. CLEAR ZONE (L_c) FOR LOW VOLUMES

Table 1 of Figure 8-C for example, for an ADT of 500 and a speed of 40 mph, L_c can be determined as follows:

L_c (40 mph) = 15 ft. from Figure 8-A, for a 10:1 or flatter roadside
L_r (40 mph, ADT = 6000) = 240 ft. from Figure 8-C Table 1
L_r (40 mph, ADT = 500) = 180 ft. from Figure 8-C Table 1

$$L_c \text{ (40 mph, ADT of 500)} = \frac{180}{240} \times 15 = 11.25 \text{ Ft.}$$

3. CLEAR ZONE (L_c) FOR HORIZONTAL CURVES

The clear zone widths obtained from Figure 8-A should be increased on the outside of curves. The amount of increase can be determined by the following formula:

$$L_{ci} = R\left(1 - \cos \frac{57.3 L_r}{R}\right)$$

where L_{ci} = increase in clear zone width (10:1 or flatter roadside)
R = curve radius
L_r = See Table 1 of Figure 8-C

For example, the required clear zone width on the outside of a 5000 foot radius curve with a speed of 60 mph can be determined as follows:

$$5000 \left[1 - \cos\left(\frac{57.3 \times 400}{5000}\right)\right] = 16$$

Therefore L_c = 16 + 30 = 46 ft.

When L_c is required on the outside of a curve where the roadside is sloped, use the above formula but increase the value of L_c by the ratio of clear zone width for a sloped roadside divided by the clear zone width for a flat roadside.

For example, if in the previous example the roadside was not 10:1 but was a 4:1 slope, the clear zone width can be determined as follows:

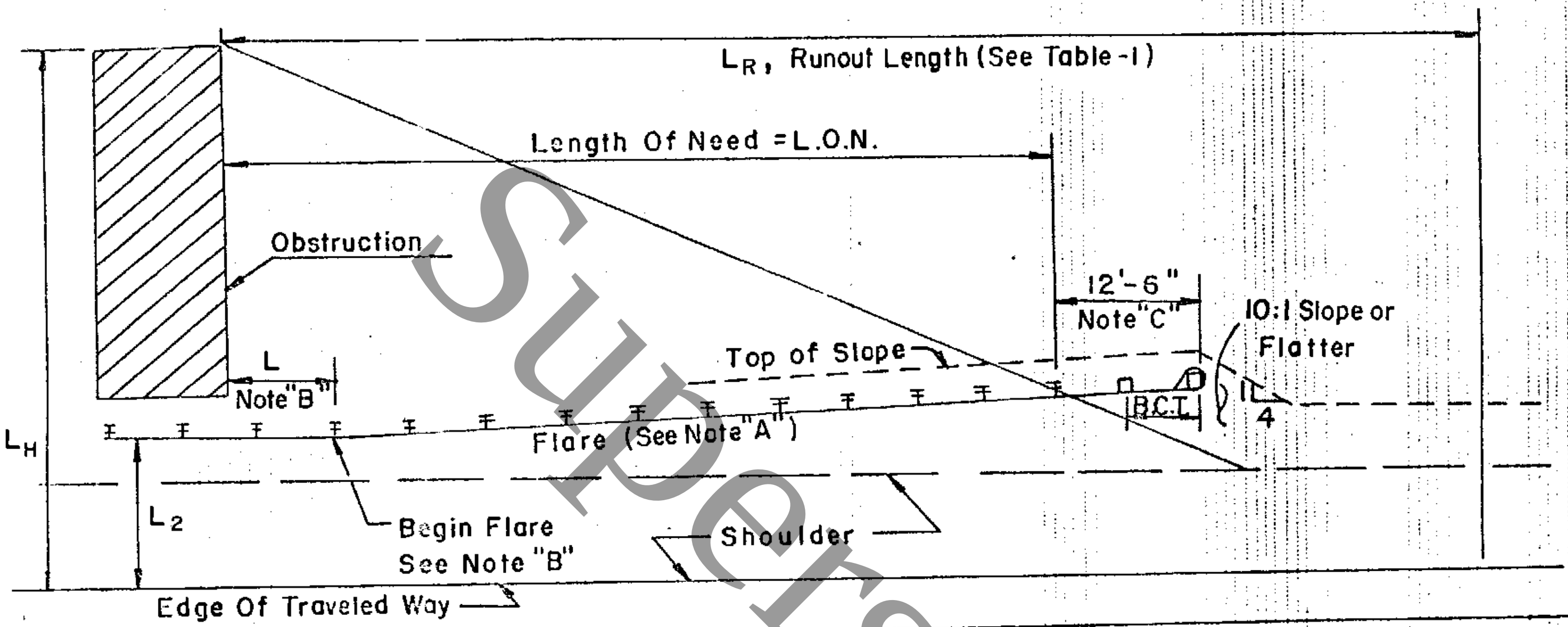
L_c (level area, straight roadway) = 30 ft. (from Figure 8-A)
L_c (4:1 slope, straight roadway) = 47 ft. (from Figure 8-A)

$\frac{47}{30} \times 16 = 25$ ft. Therefore L_c = 25 + 30 = 55 ft.

LENGTH OF NEED

FIGURE: 8-C

DATE: 11/83



NOTE "A"

Use Parabolic Flare on all Guiderail Installations Terminating within the clear zone. A Straight Flare May Be Used Where the Installation Terminates Beyond the clear zone. For Method of Determining L.O.N. in Cut Sections, See Figure 8-G.

NOTE "B"

Begin Flare at First Post that is 6'-3" Minimum Beyond Obstruction. In L.O.N. Formula For Straight Flare, Use $L_1 = 12.5'$.

NOTE "C"

See Figure 8-H For Guiderail Treatment in Cut Slopes.

TABLE - I

Design Speed (MPH)	Traffic Volume (A.D.T.): Over 6000, L_R	2000-6000, L_R	800-2000, L_R	Under 800, L_R	Shy Line Offset	Flare Rate
70	480	440	400	360	10.0	15:1
60	400	360	330	300	8.0	13:1
50	320	290	260	240	6.5	11:1
40	240	220	200	180	5.0	9:1
25	120	110	100	90	4.0	9:1

Note "D"

If Roadway is Curved, Draw Layout to Scale And Obtain L.O.N. Directly By Scaling From Drawing.

STRAIGHT FLARE

$$L.O.N. = \frac{L_H + \frac{b}{a}(L_1) - L_2}{\frac{b}{a} + \frac{L_H}{L_R}}$$

PARABOLIC FLARE

$$L.O.N. = \frac{L_R(L_H - L_2 - 1.78)}{L_H}$$

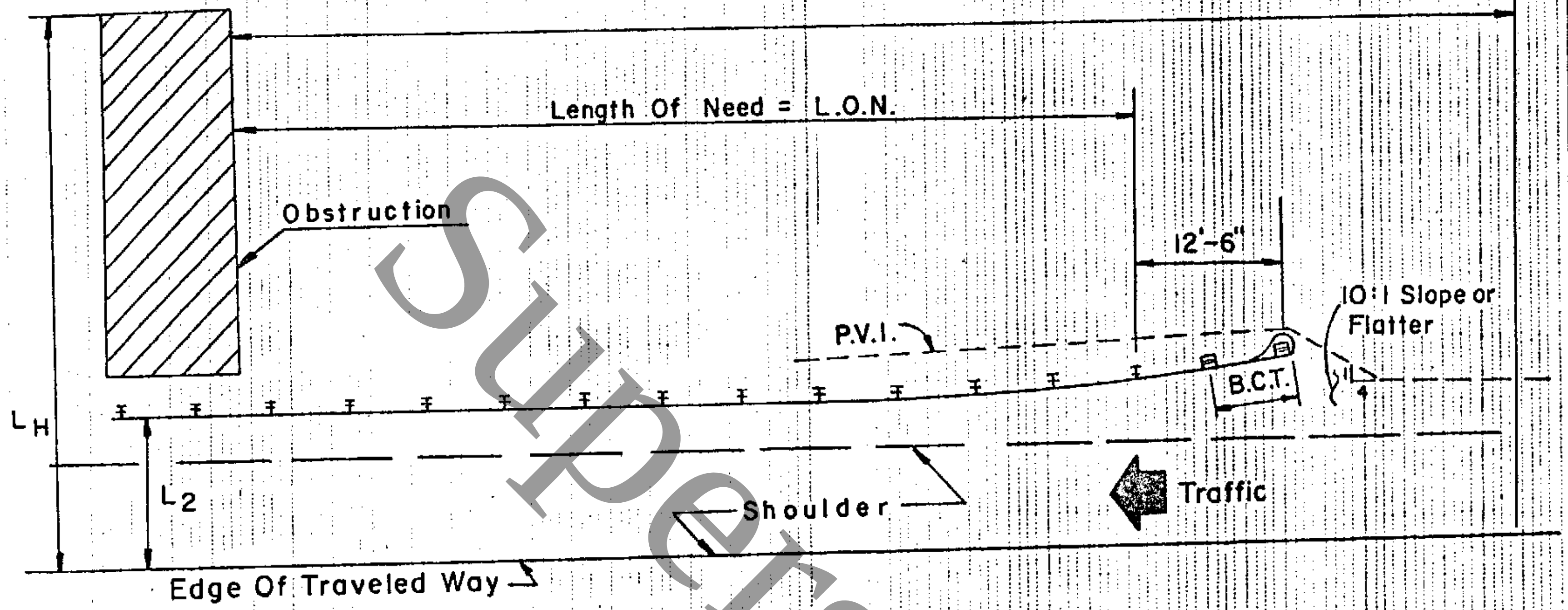
Superseded

8-8

APPROACH LENGTH OF NEED PARABOLIC FLARE

FIGURE: 8-D

DATE: 11/83



EXAMPLE:

A.D.T. = 7000

DESIGN SPEED = 70 mph

L_1 = Use 12.5'

L_2 = 16'

L_H = 22'

L_R = 480' (From Table-1, Fig. 8-D)

$$\text{L.O.N.} = \frac{L_R(L_H - L_2 - 1.78)}{L_H}$$

$$\text{L.O.N.} = \frac{480(22-16-1.78)}{22}$$

L.O.N. = 92.07'

Increase 92.07' To Nearest Multiple Of 12'-6",
Use L.O.N. = 100'

If $L_H > L_C$, Use L_C (L_C = Clear Zone Width)
(For Flare Offset of 4 ft. Only)

NOTE "A":

For Minimum Desirable Length of Need, See Figures 8-N, 8-O, & 8-P.

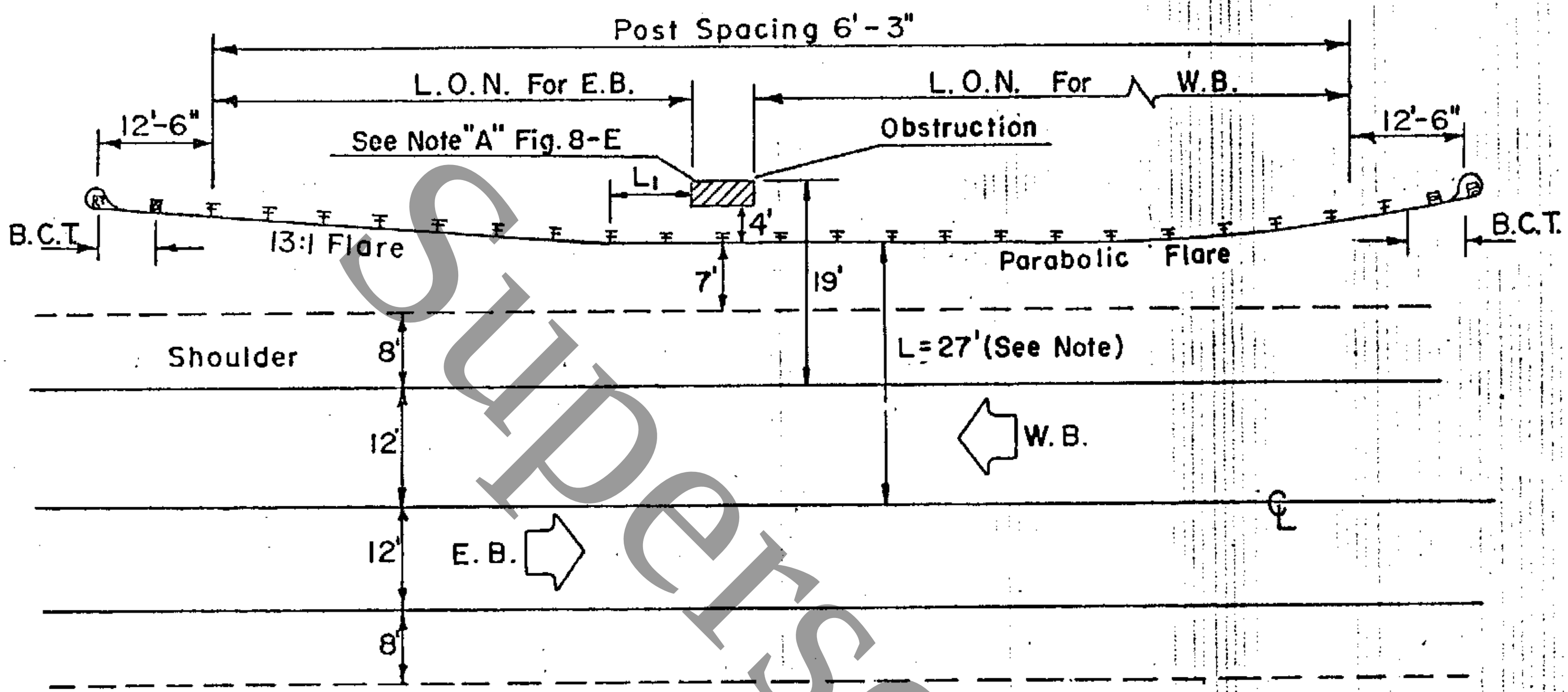
NOTE "B":

If Roadway Is Curved, Draw Layout To Scale And Obtain L.O.N. Directly By Scaling From Drawing.

L.O.N. FOR TWO WAY ROADWAY

FIGURE-8-E

DATE: 11/83



EXAMPLE

Design Speed = 60 M.P.H.
A.D.T. = 7000

L.O.N. For W.B.

$L_2 = 12'$ Use $L_1 = 12.5'$

$L_H = 19'$

L_R (From Table I) = 400

Use Parabolic Flare

L.O.N. = 400(19-15-1.78)19 = 46.73'

Use L.O.N. = 50' So That The Length including a B.C.T. Will Be 62'-6" Which is The Minimum Desirable Length.

L.O.N. For E.B.

$L_2 = 24'$ Use $L_1 = 12.5'$

$L_H = 31'$, Use $L_c = 30$

$L_R = 400$

Assume Straight Flare

$$\text{L.O.N.} = \frac{30 + \frac{1}{13}(12.5') - 24}{\frac{1}{13} + \frac{30}{400}}$$

Use L.O.N. = 50' So That The Length including a B.C.T. Will Be 62'-6" Which is The Minimum Desirable Length. Since The B.C.T. on The E.B. Approach is Beyond The Clear Zone A Straight Flare is Acceptable.

NOTE

"Approach" Length For E.B. Not Required If $L \geq L_c$. If E.B. & W.B. Are Separated By A Transversable Median, The Median Width Should Be Included When Determining L. (L_c = Clear Zone Width)

REQUIRED CLEAR ZONE IN CUT SECTIONS

FIGURE: 8-F

DATE: 11/83

The clear zone in cut sections is determined as follows:

(1) From figure 8-A select clear zone for the cut slope in question.

(2) If the total distance between the PVI and the edge of lane is greater than 12 ft.

(a) Subtract 12 ft. from the clear zone selected in Step 1.

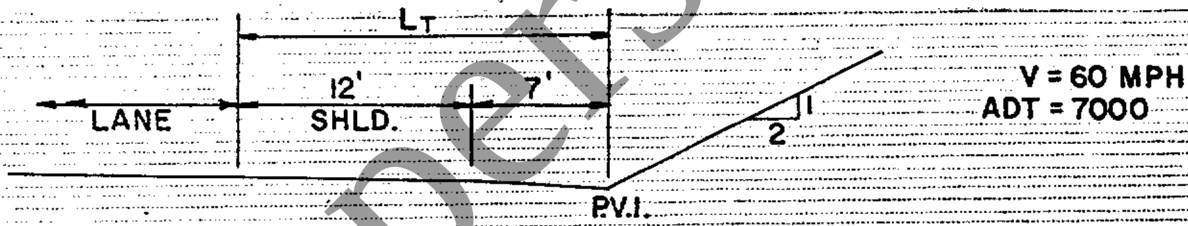
(b) Add the difference to the total distance between the PVI and edge of lane (L_T).

(c) Determine the clear zone required for a flat area.

(d) The smaller of the two distances in (b) or (c) will be the required clear zone.

(3) If the distance between the PVI and edge of lane is 12 ft. or less, the required clear zone will be determined directly from Figure 8-A.

<u>EXAMPLE</u>



(1) From Figure 8-A for a cut slope of 2:1 and a speed of 60 mph C_z = 20 ft.

(2a) 20-12 = 8 ft.

(2b) 8+(12+7) = 27 ft.

(2c) Clear zone for a flat area V = 60 is 30 ft.

(2d) Since (2b) is less than (2c), use 27 ft. Therefore Required Clear Zone is 27 ft.

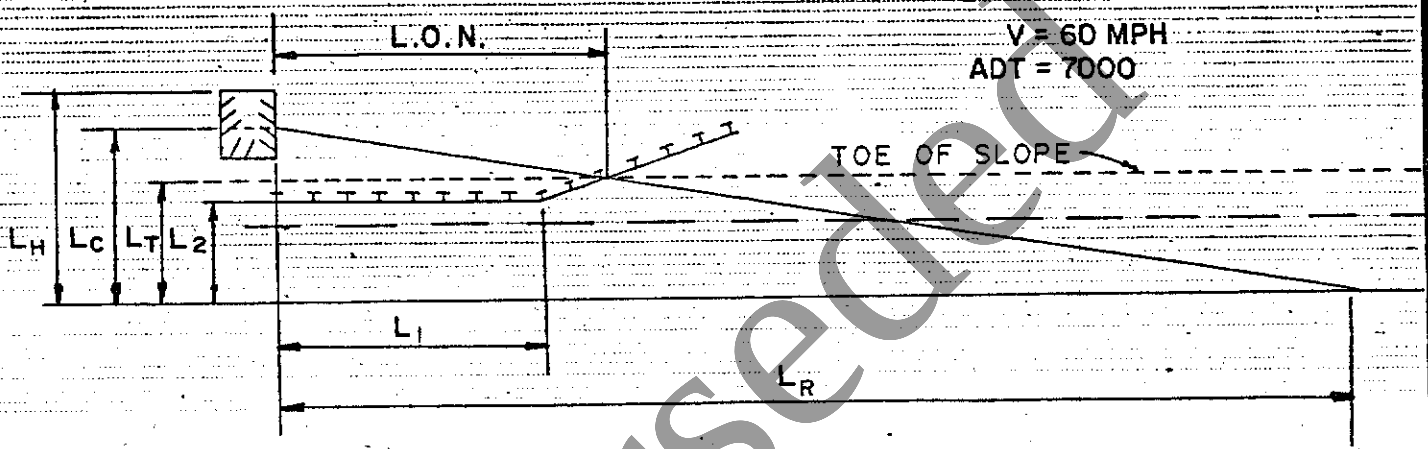
APPROACH LENGTH OF NEED IN CUT SECTIONS

FIGURE: 8-G

DATE: 11/83

Where an obstruction is encountered in a cut section and it is to be shielded with guide rail, it is desirable that the length of need (L.O.N.) end at the toe of slope Figure 8-H. In order to accomplish this, the length of guiderail (L_1) parallel to the toe of slope must be obtained. The following example shows how the L.O.N. is computed:

EXAMPLE



L_2 = 16 ft.

L_H = 30 ft.

L_R = 400 (From Figure 8-C Table 1)

L_T = 19 ft.

13:1 Flare (From Figure 8-C Table 1)

L_C = 27 ft. (From Example Figure 8-F)

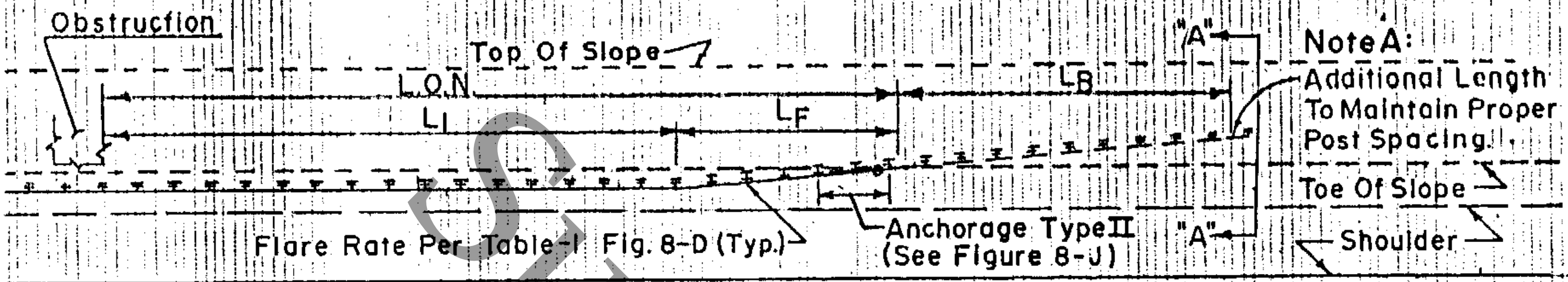
If $L_H > L_C$ use L_C

$L_1 = L_R - (L_T L_R / L_H) - a/b\ (L_T - L_2)$

$L_1 = 400 - (19 \times 400/27) - 13/1\ (19-16) = 79.5'$

Use 13 post @ 6.25' = 81.25 ft.
Therefore L_1 = 81.25'
Flare Length = $(L_T - L_2)\ a/b = (19-16)\ 13/1 = 39$ ft.
L.O.N. = 81.25 ft. + 39 ft. = 120.25 ft.

NOTE: L_1 shall not be less than 12' - 6".



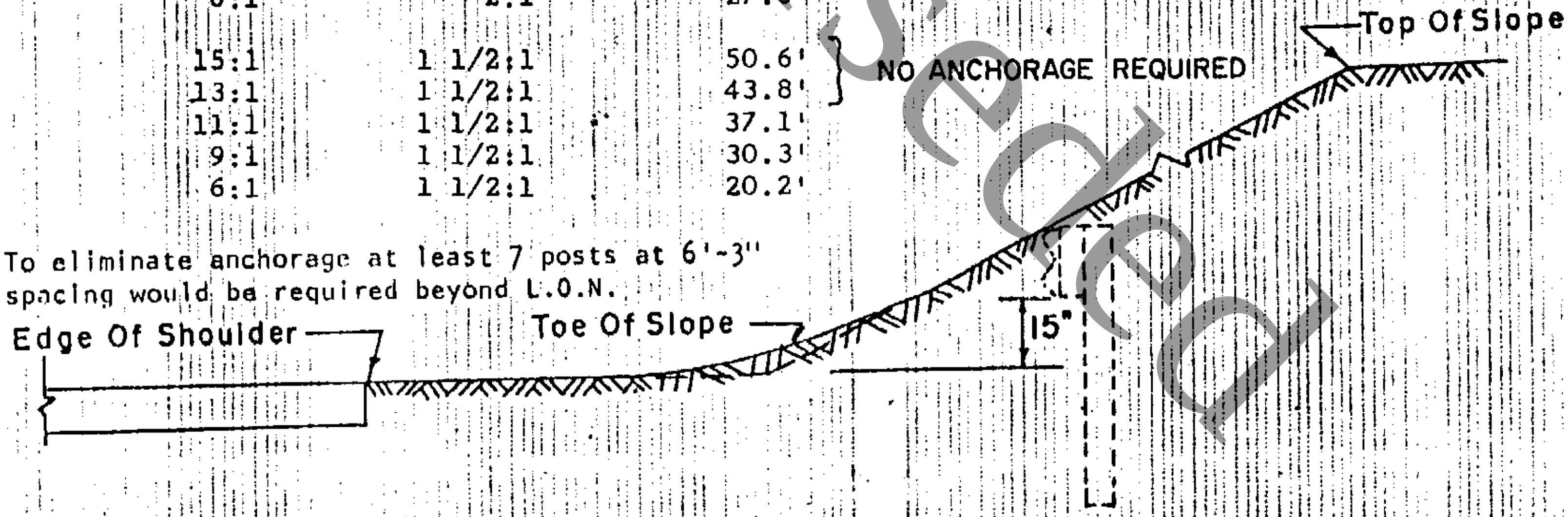
Traffic

TABLE II

L_B = Length needed beyond toe of slope to bury guide rail.

Flare	Slope	L_B	
15:1	2:1	67.5'	NO ANCHORAGE REQUIRED
13:1	2:1	58.5'	
11:1	2:1	49.5'	
9:1	2:1	40.5'	
6:1	2:1	27.0'	
15:1	1 1/2:1	50.6'	NO ANCHORAGE REQUIRED
13:1	1 1/2:1	43.8'	
11:1	1 1/2:1	37.1'	
9:1	1 1/2:1	30.3'	
6:1	1 1/2:1	20.2'	

To eliminate anchorage at least 7 posts at 6'-3" spacing would be required beyond L.O.N.



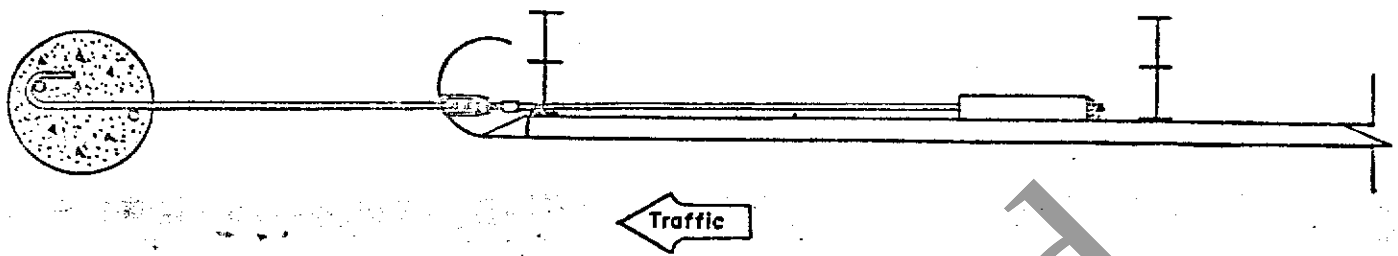
SECTION A-A

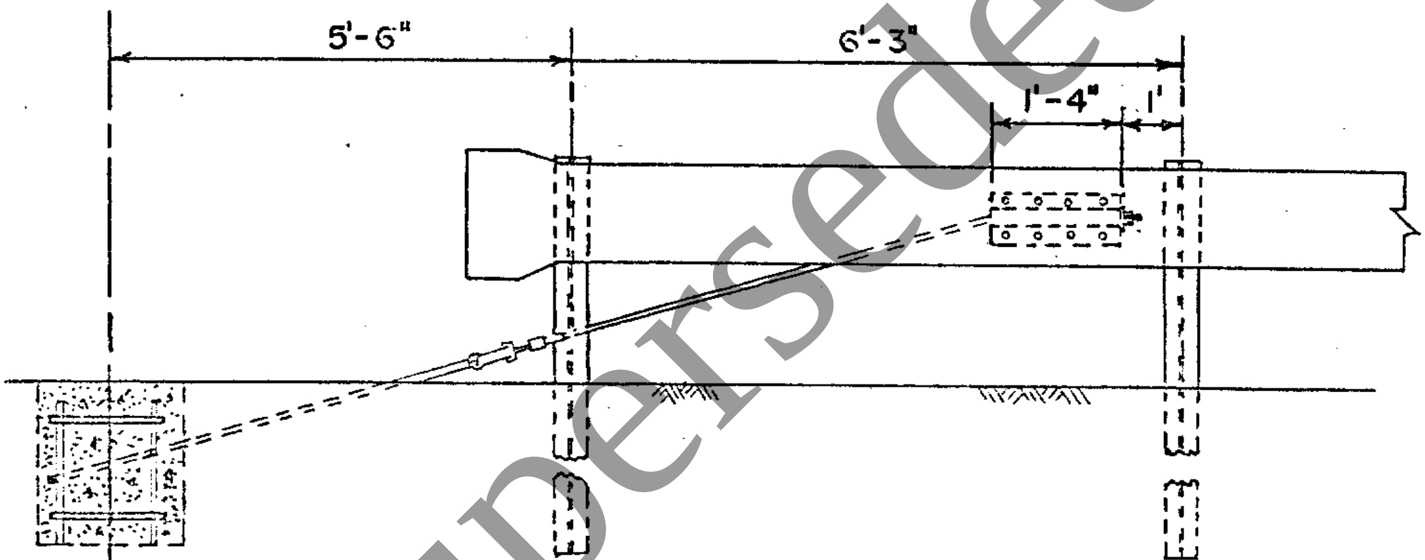
ANCHORAGE TYPE I

FIGURE : 8-I

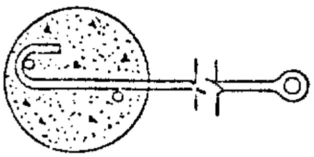
DATE: 9-26-83

D.B.

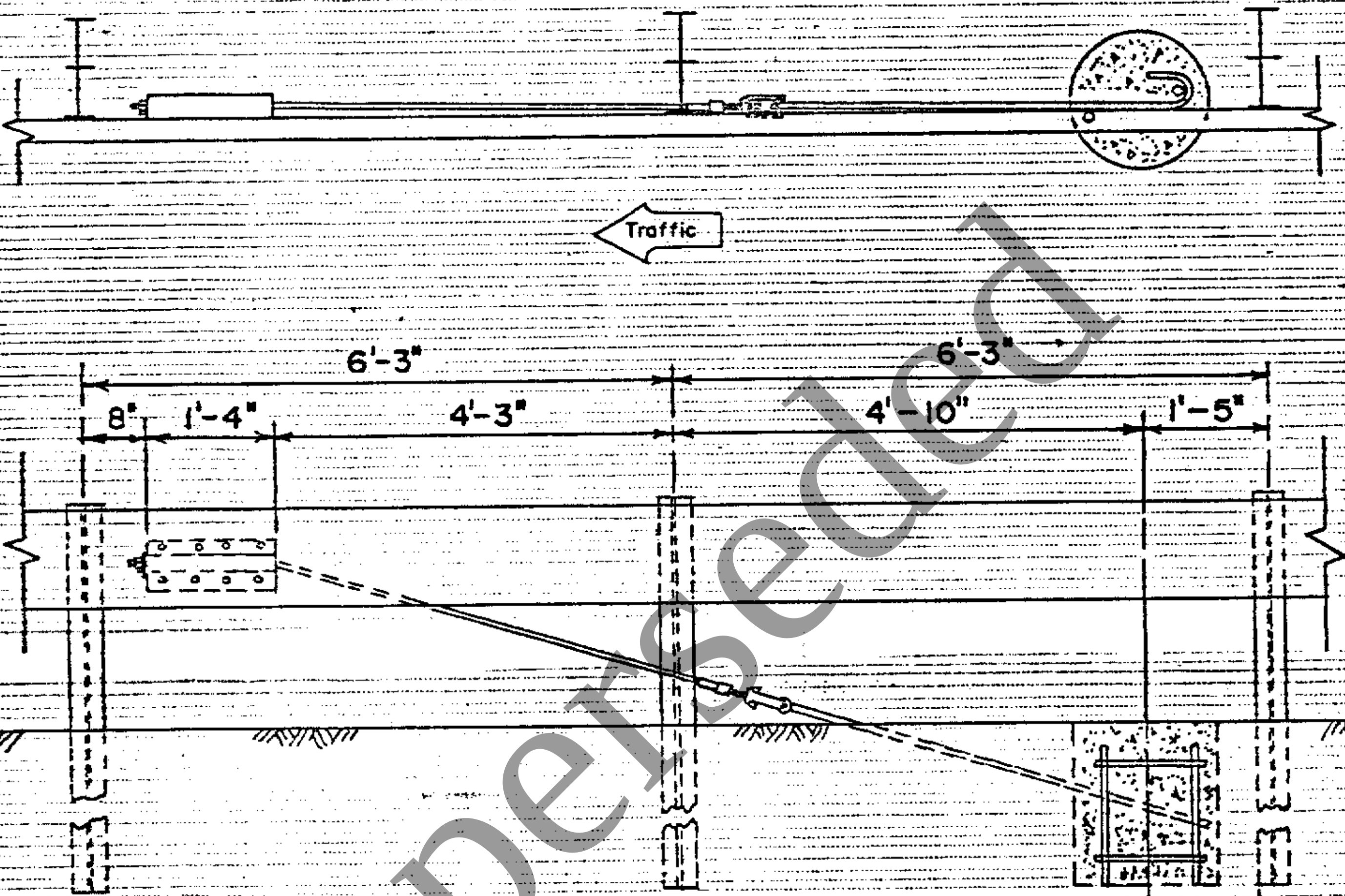


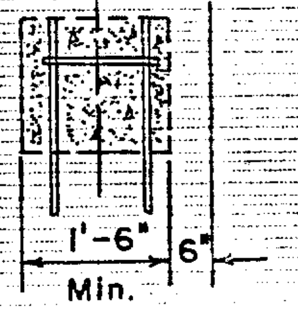


For dimensions or details not shown, see guide rail anchorage in the Standard Construction Details.



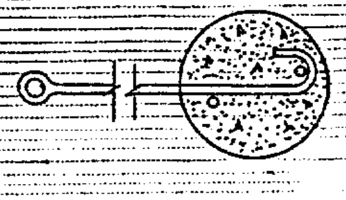
ANCHORAGE TYPE II	FIGURE: 8-J
	DATE: 11/83





Use when end of guide rail is buried in the slope.

For dimensions or details not shown, see guide rail anchorage in the Standard Construction Details.



Where BCT's must be constructed within the clear zone behind 8 inch vertical curb, fifty (50) feet of curb immediately in advance of the BCT shall be removed and replaced with 9" x 16" (4" face) white concrete vertical curb.

A clear area should be provided behind BCT installations when practical. The desired clear area is shown cross-hatched on figure 8-K.

In addition, rub rail, reduced post spacings, and double rail elements should not be used in the first 50 feet from an "approach" end using the BCT treatment.

3. <u>Median Breakaway Cable Terminal (MBCT)</u>

A Median Breakaway Cable Terminal should be used when terminating dual face beam guide rail within the clear distance of travel from the approach direction, Figure 8-L.

The MBCT should be installed on relatively flat surfaces (10:1 max. slope). Use on raised islands or behind curbs is not recommended.

When the offset to the end of guide rail shielding an obstruction from the approach direction, is equal to or greater than the clear distance, the dual BCT end treatment shown in Figure 8-M should be used.

8-03.3 <u>Clearance</u>

1. <u>From Traveled Way</u>

A highly desirable characteristic of any roadway is a uniform clearance from the traveled way to the guide rail. If possible, guide rail should be placed at a distance beyond which it will not be perceived as a threat by the driver, Figure 8-D, Table-I Shy Line Offset. In general, the following offsets should be used:

a. To the extent possible, guide rail should be located as fare as possible away from the traveled way.

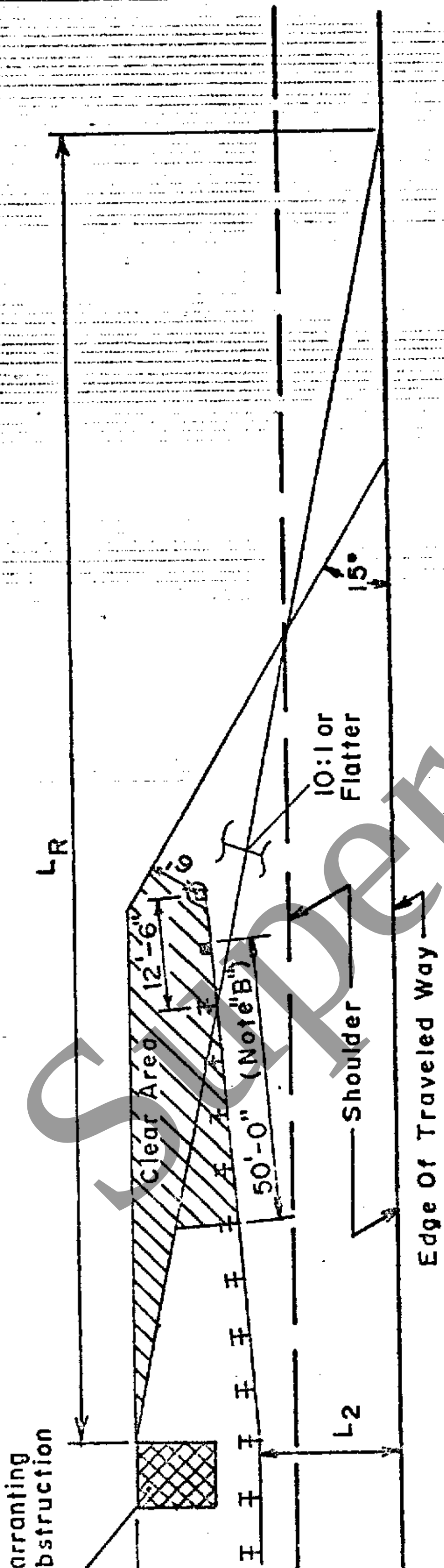
b. On interstate highways and freeways, the front face of the guide rail should be a minimum of 4 feet from the outside edge of shoulder except as provided in paragraph 8-03.7.

c. On land service highways the front face of the guide rail should be a minimum of 7 feet from the outside edge of shoulder.

CLEAR AREA BEHIND B.C.T.

FIGURE: 8-K

DATE: 11/83



NOTE "A"

No Fixed Objects Should be within the Cross-Hatched Area.

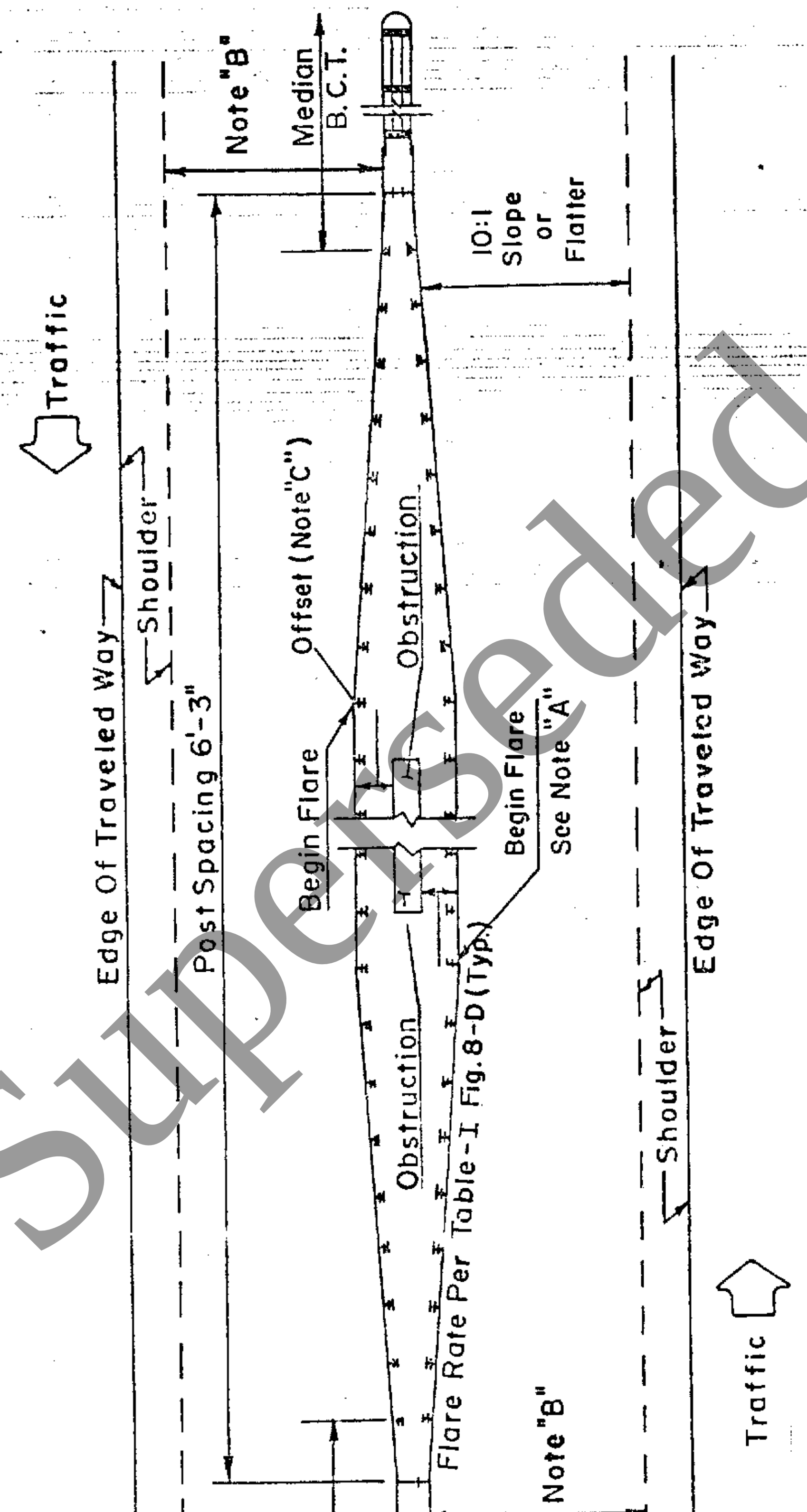
NOTE "B"

Rub Rail, Reduced Post Spacing And Double Rail Elements Should Not Be Used In The First 50'-0" From a B.C.T. Treatment.

OBSTRUCTION IN MEDIAN APPROACH END TREATMENT WITHIN THE CLEAR ZONE

FIGURE - 8-L

DATE: 11/83



NOTE "A"

Begin Flare At First Post That is 6'-3" Minimum From Obstruction.

NOTE "B"

Use MBCT When Offset is Less Than Clear Zone (Lc) Requirement.

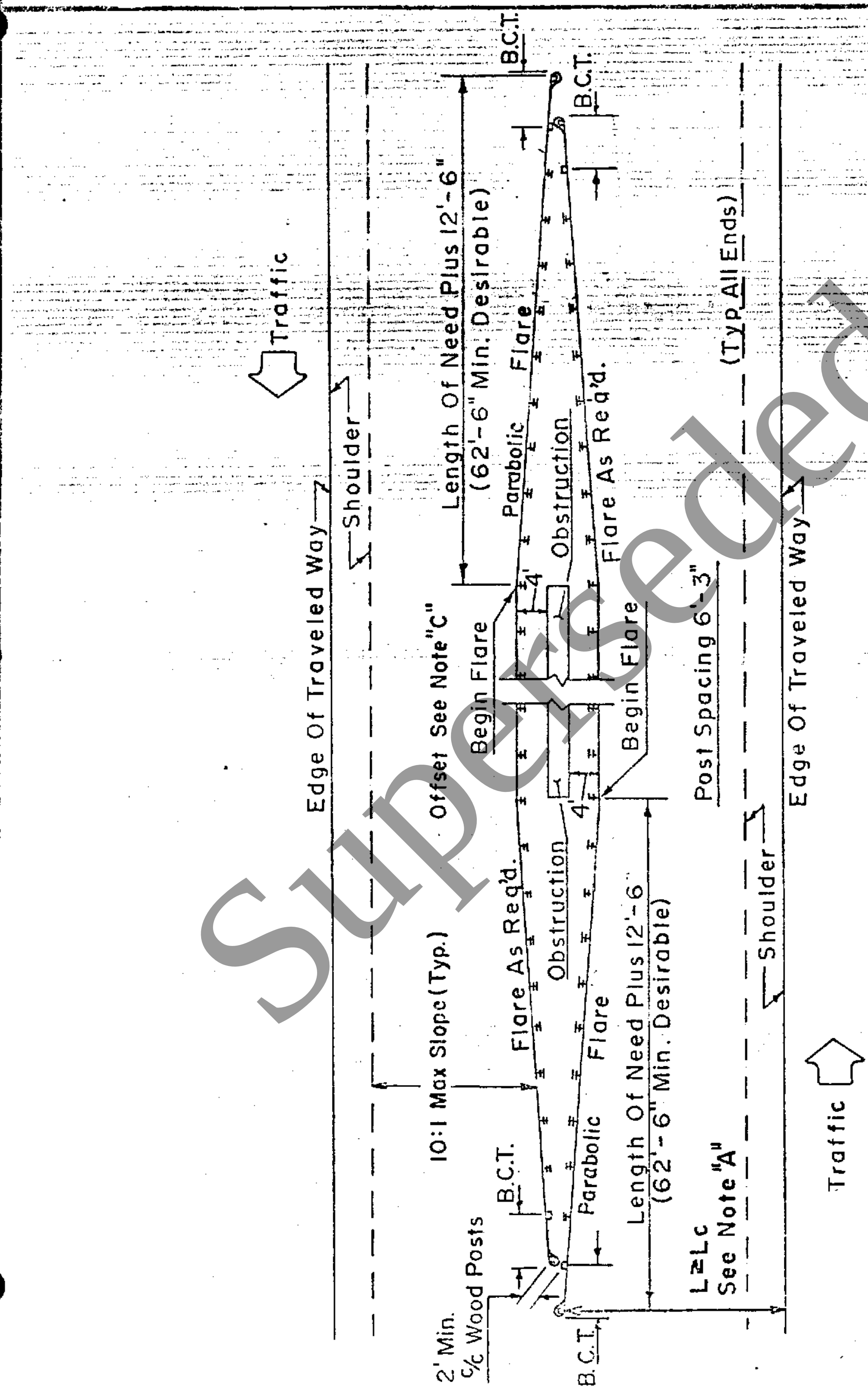
NOTE "C"

For Post Spacing at Obstruction See Figure 8-Q and 8-R.

OBSTRUCTION IN MEDIAN
APPROACH END TREATMENT BEYOND CLEAR ZONE

FIGURE-8-M

DATE: 11/83



NOTE "A"

Use Dual B.C.T.'S When Offset is equal to or greater than the Clear Zone (L_c) Requirement.

NOTE "B"

For Post Spacing at Obstruction See Figures 8-Q and 8-R.

2. <u>At Fixed Objects</u>

Where guide rail is used to shield an isolated obstruction it is more important that the guide rail be located as far from the traveled way as possible so as to minimize the probability of impact. The distance from the face of the rail element to the face of obstruction should desirably be a minimum of 4 feet, Figure 8-N. If less than 4 feet must be used, the guide rail system must be modified (See Figures 8-O thru 8-P.)

3. <u>On Bridges</u>

a. On existing freeway and interstate structures with safety walks equal to or greater than 1'-6" and not feasible to remove, and provide a concrete barrier shaped parapet, the guide rail should be carried through the structure.

b. When the roadway approaching a structure is curbed or bermed, the guiderail mounting height on the structure should be measured from the top of curb. On long structures, however, the guiderail mounting height may be measured from the gutterline provided the face of guide rail is flush with the curb face.

The guiderail mounting height should be measured from the gutterline on those structures where the approach roadway is an umbrella section and the face of guide rail is set flush with the curb face on the structure.

c. When there is a difference in the offset to the approach guide rail and the offset to the bridge parapet, a transition flare of 15:1 should be used.

Where guide rail is set flush with the curb face and the mounting height is measured from the gutter line, rub rail may be omitted.

d. Attachment of guiderail to bridges and structures shall be in accordance with the Departments's Standard Details.

8-03.4 <u>Flare Offset</u>

Although it is desirable to introduce the end of guiderail as far from the travelled way as possible, it is questionable whether a flared offset beyond the clear zone is beneficial. Providing flared offsets of this magnitude increases the length of the guiderail installation and may require additional earthwork and/or right-of-way. As a minimum, a flare offset of at least four (4) feet should be provided.

USE WHEN FACE OF RAIL ELEMENT IS 4 FEET FROM THE OBSTRUCTION	FIGURE - 8-N
	DATE: 11/83

NOTE "A"

For Straight Flare At First Post is 6'-3" Minimum From Obstruction.

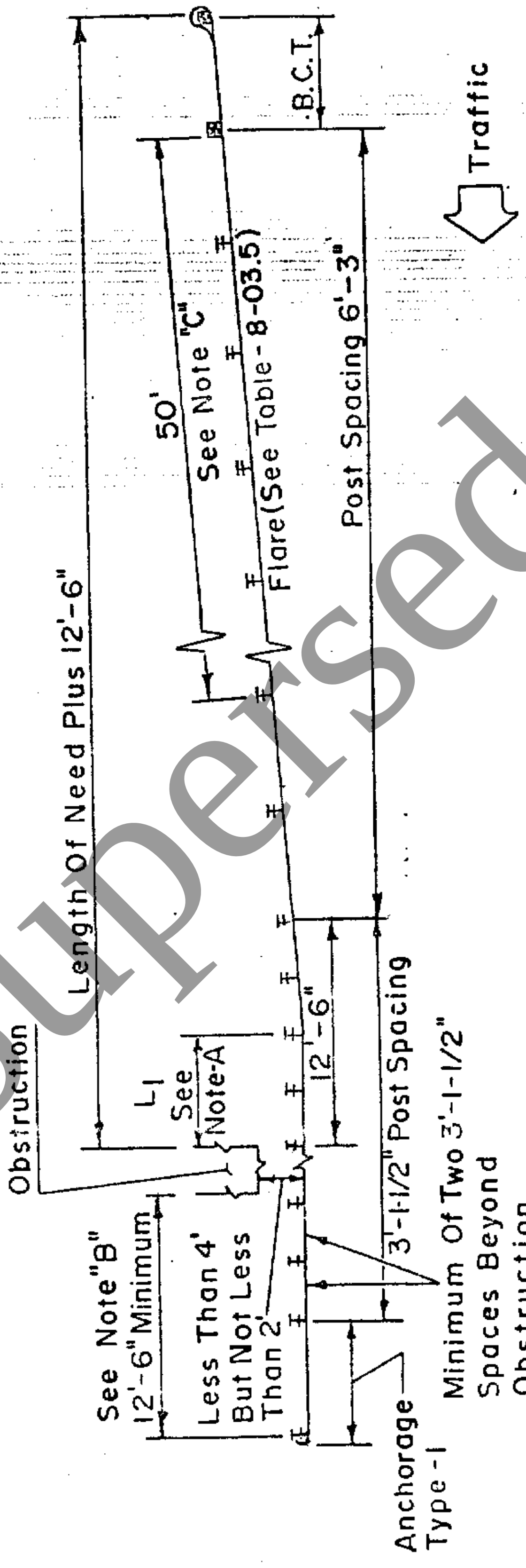
NOTE "B"

On A Two Way Undivided Roadway Or On A Divided Roadway With A Traversable Median, An "Approach" Treatment May Be Required (See Figure 8-E)

NOTE "C"

No Reduced Post Spacings, or Rub Rail.

USE WHEN FACE OF RAIL ELEMENT IS LESS THAN 4 FEET BUT NOT LESS THAN 2 FEET FROM OBSTRUCTION	FIGURE: 8-O DATE: 11/83



NOTE "A"

For Flare At First Post That is 6'-3" Minimum From Obstruction, Use L_1=12.5' For L.O.N. Calc.

NOTE "B"

On A Two Way Undivided Roadway Or On A Divided Roadway With A Traversable Median, An Approach Treatment May Be Required (See Figure-8-E)

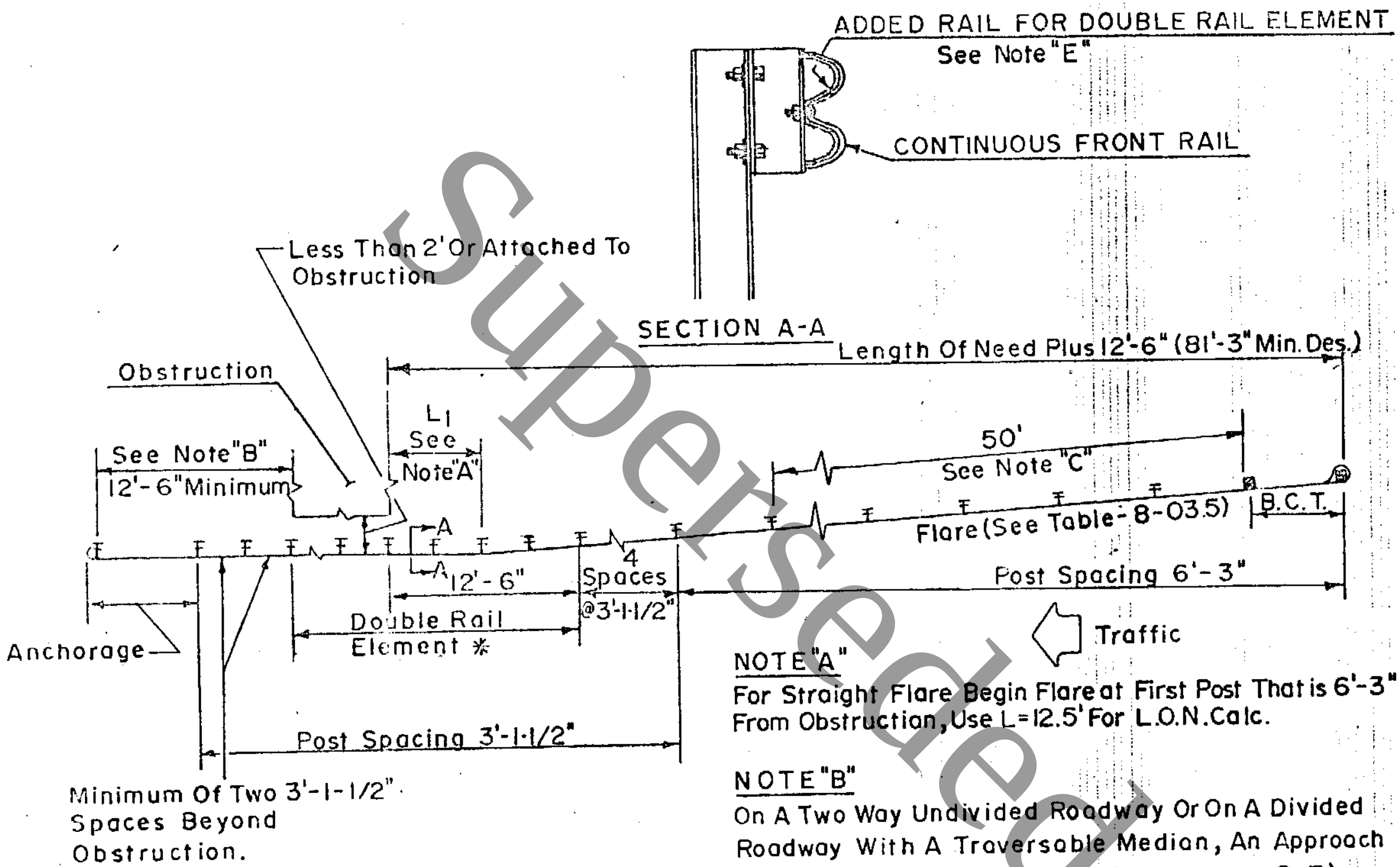
NOTE "C"

No Reduced Post Spacing or Rub Rail.

USE WHEN FACE OF RAIL ELEMENT IS LESS THAN 2 FEET FROM OBSTRUCTION OR ATTACHED TO OBSTRUCTION

FIGURE - 8-P

DATE: 11/83



NOTE "A"

For Straight Flare Begin Flare at First Post That is 6'-3" From Obstruction, Use L=12.5' For L.O.N. Calc.

NOTE "B"

On A Two Way Undivided Roadway Or On A Divided Roadway With A Traversable Median, An Approach Treatment May Be Required (See Figure 8-E)

NOTE "C"

No Reduced Post Spacing Or Rub Rail.

NOTE "D"

See "Standard Construction Details" For Attachment Details When Guide Rail is Attachmend To Obstruction.

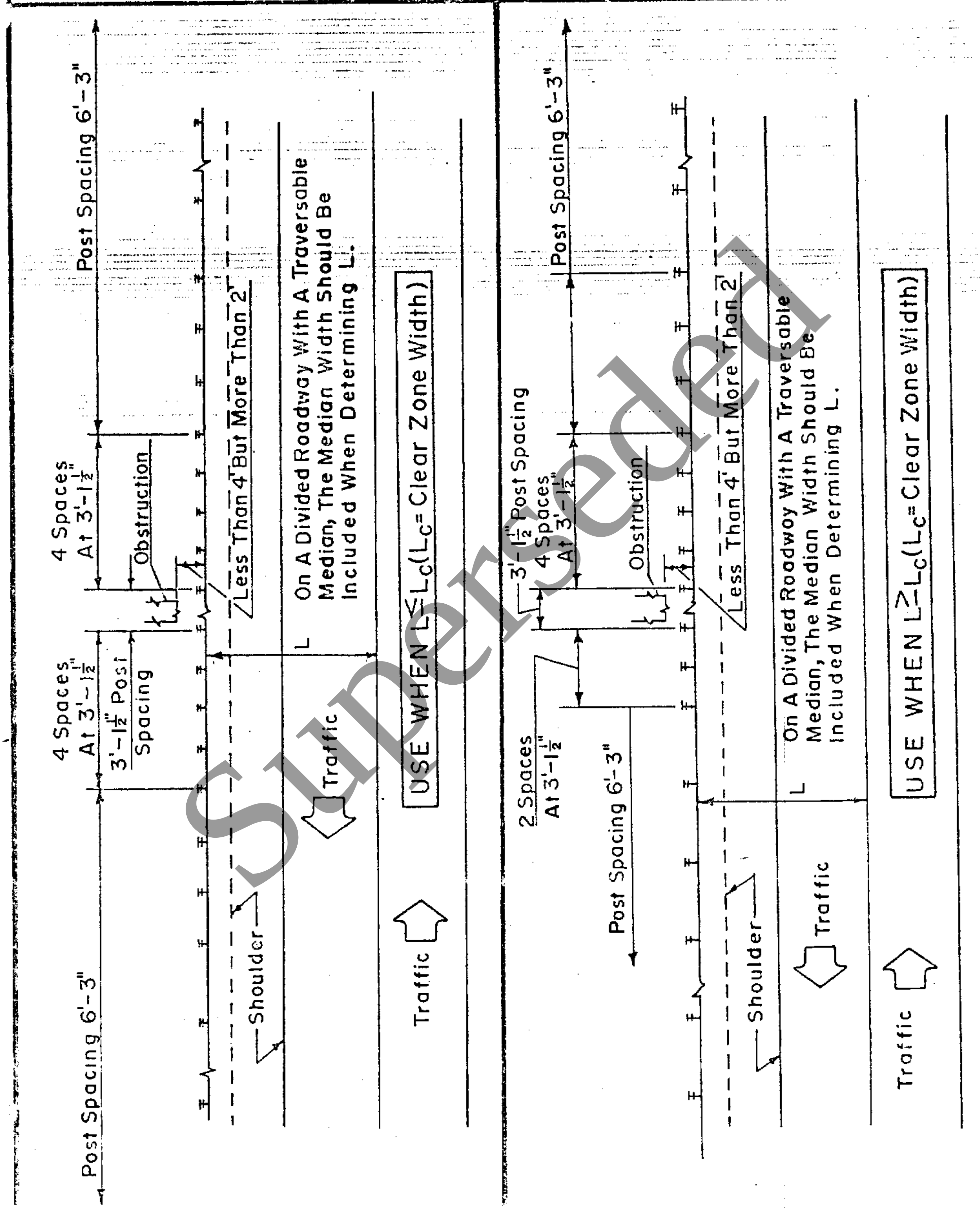
NOTE "E"

Where Double Rail Element is Required The Additional Rail Should Be Added Behind The Continuous Front Rail.

CONTINUOUS GUIDE RAIL AT OBSTRUCTION
Distance From Guide Rail To Obstruction Is Less Than 4' But More Than 2'

FIGURE-8-Q

DATE: 11/83

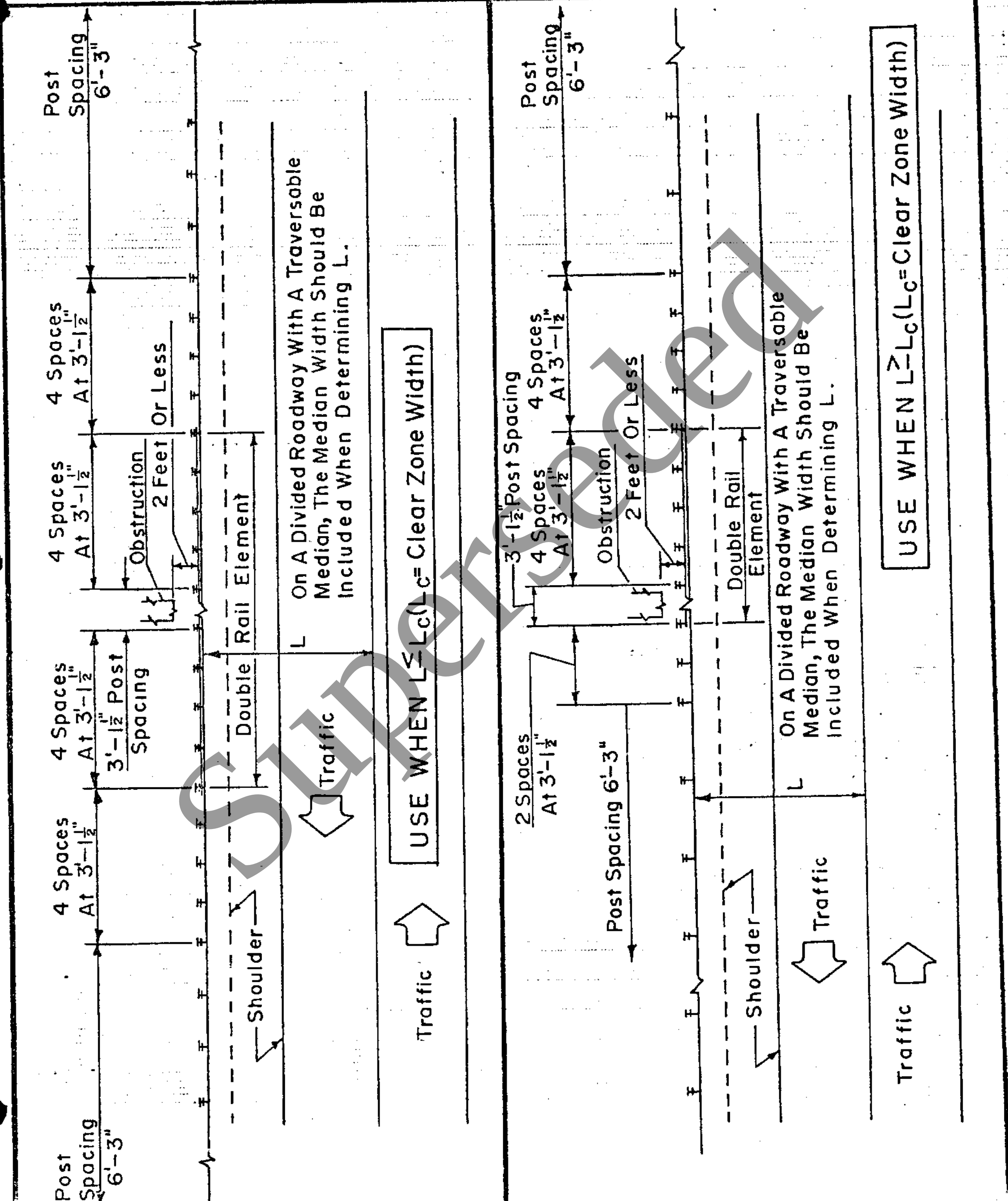


CONTINUOUS GUIDE RAIL AT OBSTRUCTION

Distance From Guide Rail To Obstruction Is 2' Or Less

FIGURE-8-R

DATE: 11/83



8-03.5 Flare

A 37'-6" parabolic flare shall be used with all BCT end treatments terminating within the clear zone. A straight flare should be used on all guide rail installations when the guide rail is buried in a side slope or when the BCT is beyond the clear zone. Table-1 of Figure 8-D shows the straight flare rate allowable for various speeds.

8-03.6 Guide rail on Embankment (fill) Slopes

1. Guide rail may be placed on slopes 10:1 or flatter provided the rollover between the shoulder slope and the embankment slope is not greater than 10%.

2. On slopes steeper than 10:1 (10%) but flatter than or equal to 6:1 (16.7%), guide rail may be placed on the slope but should be located 12 feet or more from the top of the slope.

3. Guide rail should not be placed on slopes steeper than 6:1.

4. Where guide rail is located at the top of an embankment slope, the posts should be a minimum of 2 feet from the PVI to the center of the post. When less than 2 feet is provided, the following post lengths should be used:

Slopes	Additional Post Length (feet)
Flatter than 6:1	No Change
6:1 to 4:1	1'
3:1 to 2:1	2'
Steeper than 2:1	4'

5. Figure 8-S illustrates the guiderail treatment for embankment slopes.

8-03.7 Curb or Raised Berm in Front of Guide Rail

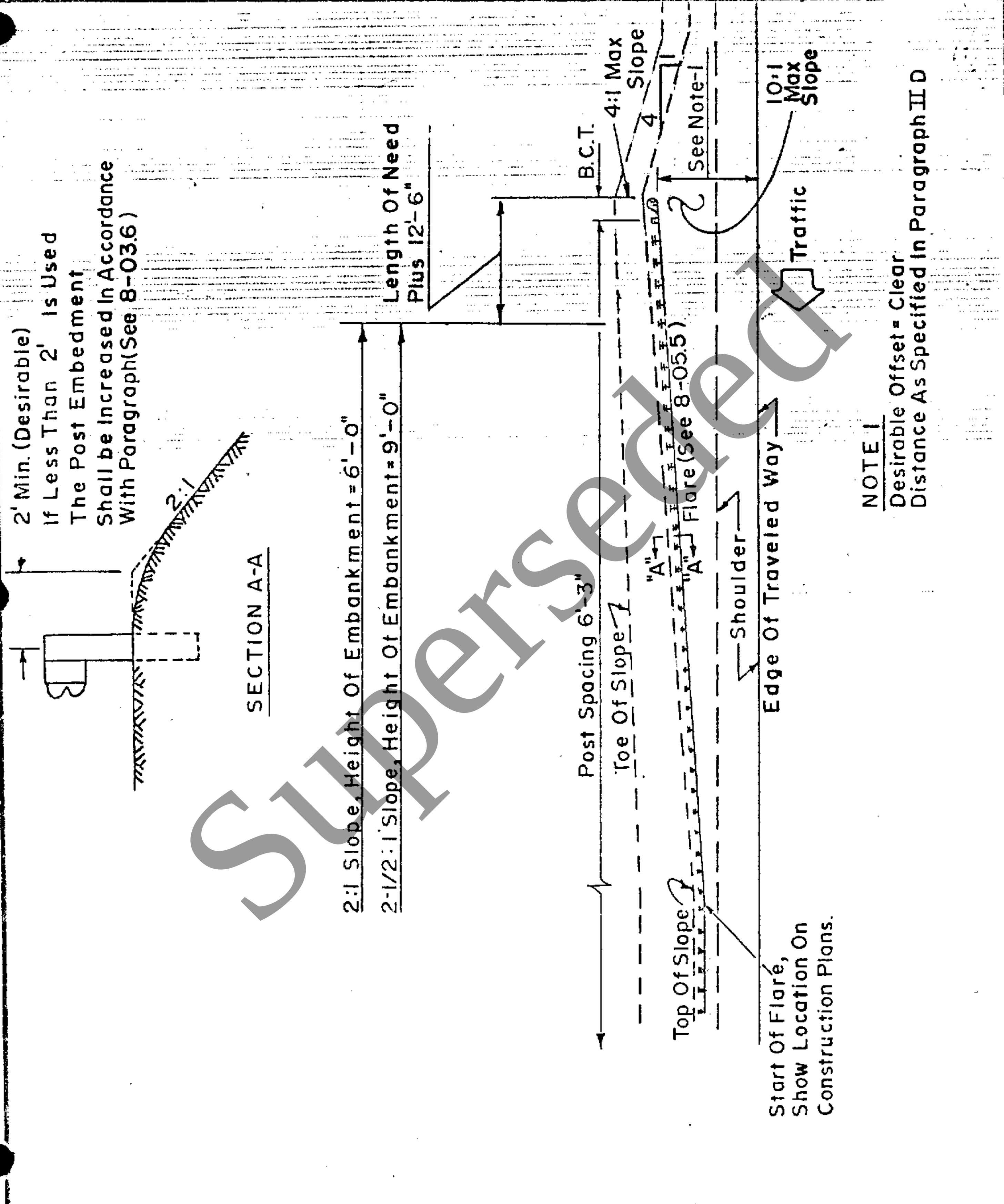
Curb or a raised berm in front of guide rail should be avoided.

On new construction, a design without curb or raised berm in front of guide rail should be provided where possible.

GUIDE RAIL TREATMENT FOR EMBANKMENT

FIGURE- 8-S

DATE: 12/83



On projects which involve upgrading existing roadways, where there is curb or a raised berm in front of guide rail, removal or modification of the curb or raised berm should be the first consideration.

If curb is present which cannot be removed, the following apply:

1. <u>Highways With a Posted Speed of More than 40 MPH</u>:

   On freeways and interstate highways where sufficient roadside width is available, guide rail should be placed 10 feet or more behind the gutter line. A minimum offset of 7 feet should be provided on landservice roadways where there is a sidewalk or sidewalk area used by pedestrains.

   Where the above offsets are not possible and there is a vertical curb or berm 6" or greater, the face of guide rail should be set flush with the gutter line.

2. <u>On Roadways With a Posted Speed of 40 MPH or Less</u>:

   Guide rail may be placed any distance behind the gutter line, but usually an offset of 4 or 7 feet should be used.

8-03.8 <u>Rub Rail</u>

When the posted speed is more than 40 mph and guide rail is constructed less than three (3) feet from a curb or raised berm 4" or greater in height, a rub rail is required.

On all projects involving new guide rail or the upgrading of existing guide rail, every effort should be given to the elimination or reduction in the use of rub rail.

Acceptable methods for reducing or eliminating the need for rub rail include; providing sufficient offsets, removing or revising earth berms, providing designs without curb, and eliminating existing curb where economically feasible.

8-03.9 <u>Guide Rail Details</u>

The dimensions and other characteristics of beam guide rail posts, rail elements, fasteners, etc. are shown in the Standard Construction Details.

8-03.10 <u>General Comments</u>

1. <u>Guide rail should not restrict sight distance.</u> Sight distances should be checked when guide rail is to be installed at intersections, ramp terminals, driveways, along sharply curving roadways, etc. If the sight distance is determined to be inadequate, the guide rail placement shall be adjusted.

2. Gaps of 200 feet or less between individual guide rail installations should be avoided where possible.

3. Guide rail should not be installed beyond the right-of-way unless easements or necessary rights-of-way are acquired.

4. Figures 8-T and 8-U show the guide rail treatment at driveways and adjacent bridges respectively.

5. Fire Hydrants

   Since fire hydrants do not meet the current AASHTO definition for breakway design, they fall into the category of fixed objects that may warrant guide rail. The same reasoning applies here as was applicable to utility poles. The acceptable solution is to locate the hydrants as far from the traveled way as possible. In no case, shall fire hydrants be located in front of the guide rail. However, the hydrants must be located so as to be readily accessible at all times. Where guide rail is required for some other reason and will be in front of a hydrant, the preferred treatment is to raise the hydrant so as to permit connection to be made over the guide rail. Usually, the connection may be a maximum of 36" above grade. It is the responsibility of the designer to confirm with the local Fire Department that such a treatment is acceptable. A less desirable treatment is to provide a short opening in the guide rail at the hydrant. Where an opening is provided, BCT's or an Anchorage must be provided in accordance with section 8-03.2.

6. Thrie Beam

   Thrie beam should not be substituted for W-beam with rub rail unless there are extenuating circumstances and then only with the approval of the Chief Engineer, Design.

7. All B.C.T.'s shall be located on the construction plans by station and offset. The applicable flare rate shall also be indicated.

8-04 MEDIAN BARRIER

A median barrier is a longitudinal system used to prevent an errant vehicle from crossing that portion of a divided highway separating travelled ways for traffic in opposite directions.

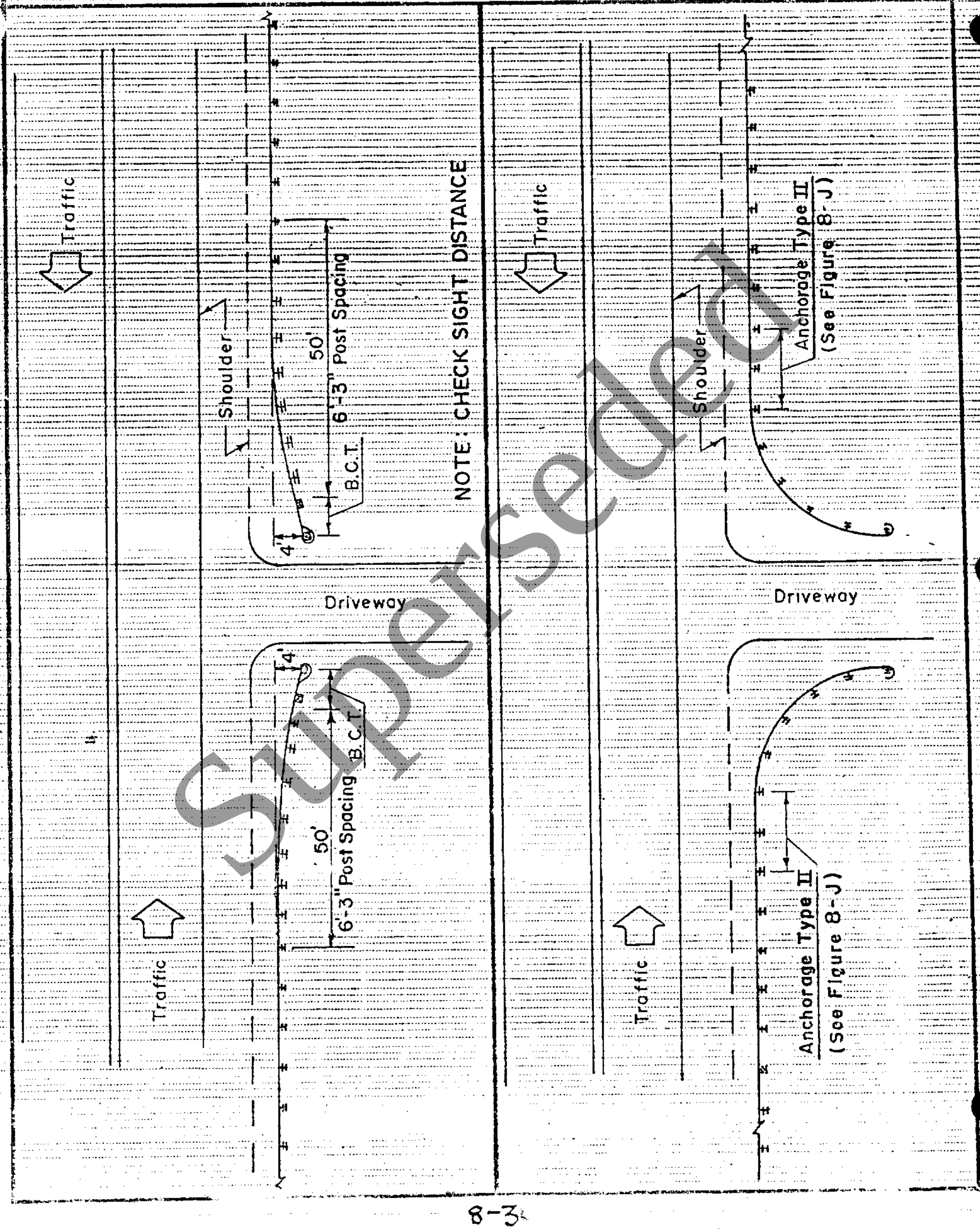
8-04.1 Warrants for Median Barriers

Figure 8-V presents the warrants for median barriers on high speed, highways with traversable slopes 10:1 or flatter.

GUIDE RAIL TREATMENT AT DRIVEWAYS

FIGURE - 8-T

DATE: 12/83

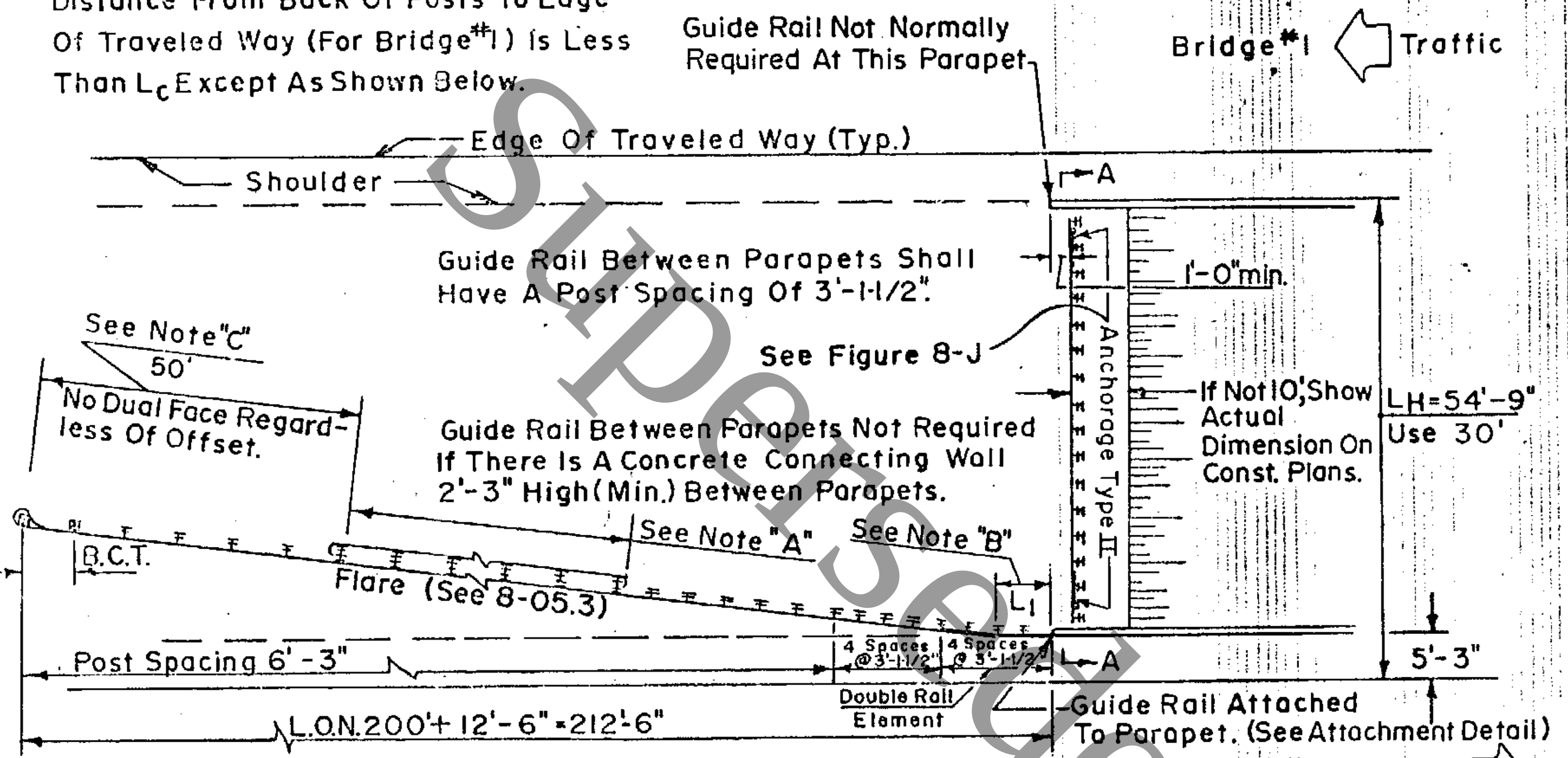


MEDIAN GUIDE RAIL TREATMENT AT ADJACENT BRIDGES

FIGURE-8-U

DATE: 12/83

NOTE "A"
Dual Face Guide Rail Required Where Distance From Back Of Posts To Edge Of Traveled Way (For Bridge #1) Is Less Than L_c Except As Shown Below.



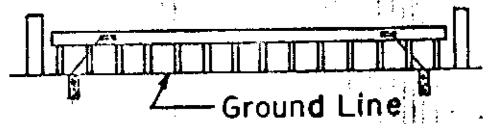
$L_H = 54'\text{-}9"$ Use 30' $L_2 = 5'\text{-}3"$ $L_R = 480$ Use $L_1 = 12.5'$

$$\text{L.O.N.} = \frac{30 + 15(12.5') - 5.25'}{\frac{1}{15} + \frac{30}{480}} = 198.03'$$

Use L.O.N = 200' + 12'-6" = 212'-6"

NOTE "B"
Begin Flare At First Post That is 6'-3" Minimum From Obstruction, Use $L_1 = 12.5'$ For L.O.N. Calc.

NOTE "C"
No Rub Rail Or Reduced Post Spacing.



SECTION A-A

At low ADT's the probability of a vehicle crossing the median is relatively small. Thus, for ADT's less than 20,000 and median widths less than 20 feet, the medians barrier is optional. Likewise, for relatively wide meadians, the probility of a vehicle crossings the median is also relatively small. Thus, for median widths greater than 30 feet with ADT's below the warrant curve, and for widths greater than 50 feet, regardless of the ADT, the use of median barriers is optional, unless there is a history of across-the-median accidents.

Figure 8-V also indicate the suggested type of median barrier as related to median width.

When the median barrier terminates within the clear zone area, a crashworthy end treatment is warranted. Acceptable methods of developing a crashworthy end treatment would be to use crash cushions or a Median Breakaway Cable Terminal with guide rail.

8-05 CONSTRUCTION PROCEDURES

In an effort to provide a safe roadside area during construction, the following procedures are a guide to installing guide rail on existing highways. Consideration should be given to placing the following procedures in the supplemental specifications of each individual project:

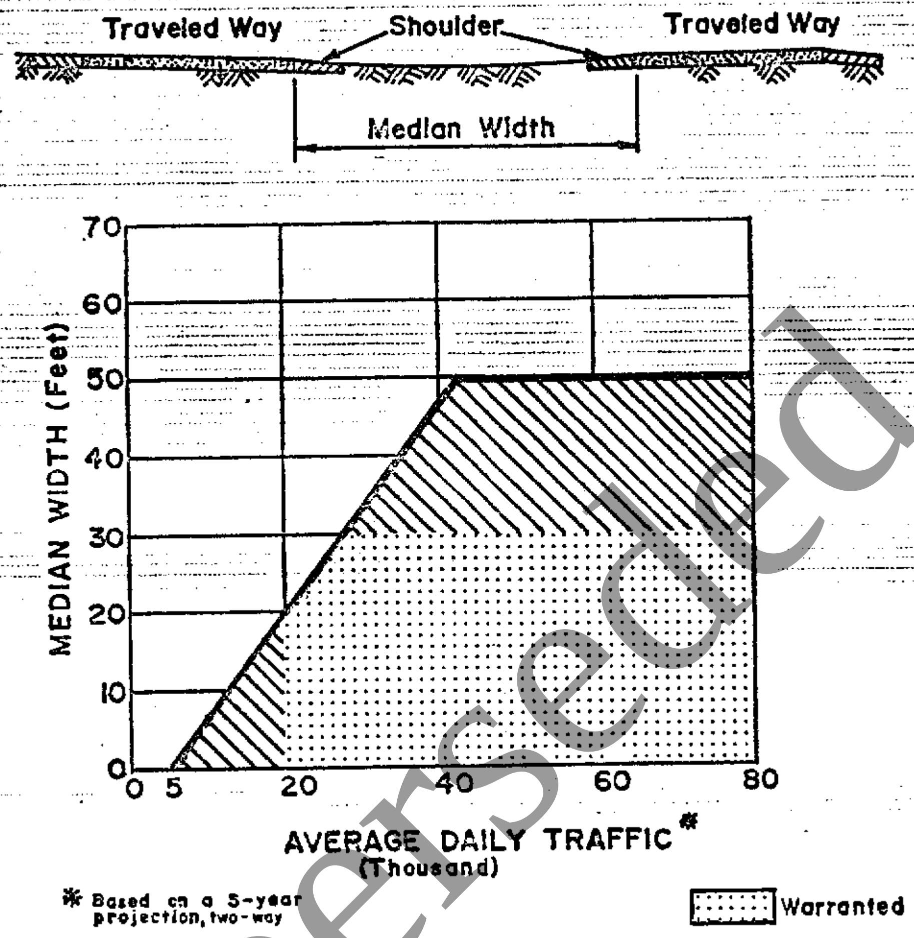
A. During the installation of guide rail, the approach terminal end shall be the first item of work.

B. Posts and rails shall be constructed in the direction of traffic.

C. At the end of the work shift, all posts that have been installed shall have the rail elements attached.

D. New guiderail should be installed prior to the removal of the existing system to the maximum extent possible to provide some degree of protection for the warranting obstruction at all times.

Warrants For Median Barrier	FIGURE: 8-V
	DATE: 11/83



Median Barrier Warrants

Source 1977 ASSHTO Guide For Selecting, Locating, and Designing Traffic Barriers.

MEDIAN BARRIER TYPE, WHEN WARRANTED, AS RELATED TO MEDIAN WIDTH

Median Width	Median Barrier Type
UP TO 12 FEET	CONCRETE BARRIER
13 FEET TO 26 FEET	CONCRETE BARRIER (PREFERRED TREATMENT) DUAL FACE BEAM GUIDE RAIL
ABOVE 26 FEET	DUAL FACE BEAM GUIDE RAIL

SECTION 9

GUIDELINES for the SELECTION and DESIGN of CRASH CUSHIONS

9-01 INTRODUCTION

Fixed objects within the clear distance should be removed, relocated or modified so as to be breakaway. When this is not practical, the obstruction should be shielded so as to prevent an impact of the obstruction by an errant vehicle.

A detailed discussion on warranting obstructions and clear distance can be found in Section 8 - Guidelines for Guide Rail Design and Median Barriers.

A crash cushion is a type of traffic barrier that can be used to shield warranting obstructions such as overhead sign supports, bridge piers, bridge abutments, and ends of retaining walls, bridge parapets and bridge railings, etc.

The most common use of a crash cushion is to shield a warranting obstruction in a gore. However, warranting obstructions in the median and along the roadside can also be shielded with a crash cushion. (See Figure 9-A).

9-02 SELECTION GUIDELINES

Once it has been determined that a crash cushion is to be used to shield a warranting obstruction, a choice must be made as to which crash cushion is best for the particular location under consideration. The crash cushions presently approved for use are:

1. Fitch Inertial Barrier

2. Energite Inertial Barrier

3. Hi-Dro Cell Sandwich

4. Hi-Dro Cell Cluster

5. Hi-Dri Cell Sandwich

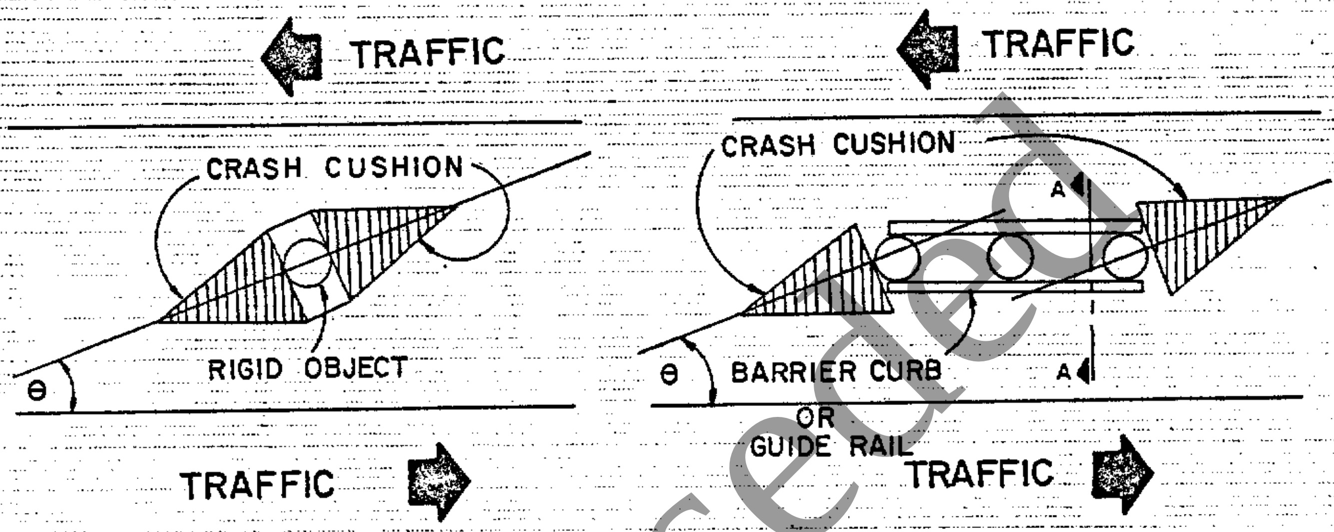
6. Guard Rail Energy Absorbing Terminal (G.R.E.A.T.)

CRASH CUSHION

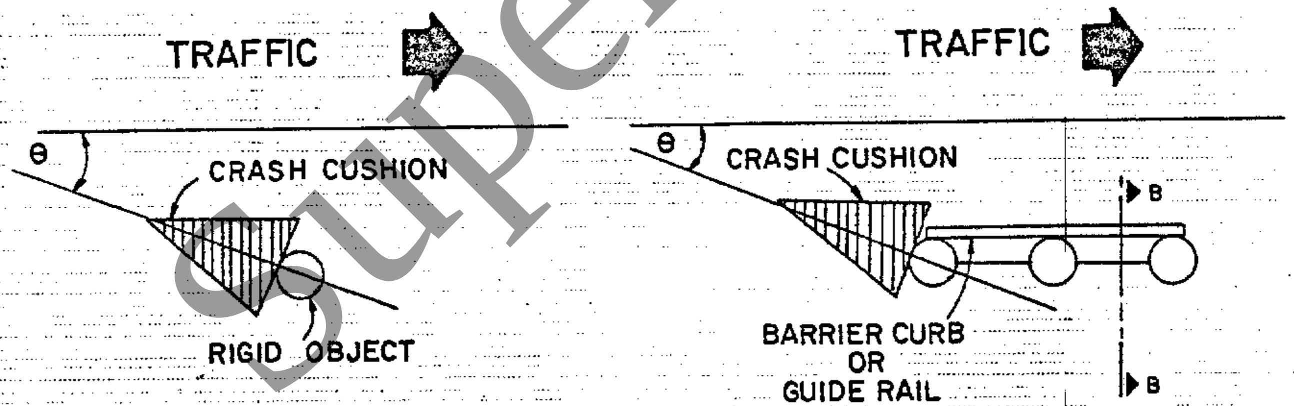
FIGURE: 9-A

DATE: 11/83

FLAT* MEDIANS



FLAT* ROADSIDE AREA



θ = 10 DEGREES MAX

* - SLOPES 5% or LESS

Several factors must be evaluated when determining which of the approved crash cuons should be used. The number and type of the factors to be evaluated precludes the development of a simple, systematic selection procedure.

The factors which normally should be considered are briefly discussed below. In many cases, evaluation of the first few items will establish the type of crash cushion to be used. Figure 9-B should be used as a check list to assist in the selection process. It is recommended that this check list be used for each location where a crash cushion is required.

9-02.1 DIMENSIONS OF THE OBSTRUCTION

Inertial barriers can be designed to shield obstructions of practically any width. The Hi-Dro Cell Sandwich and Hi-Dri Cell Sandwich are limited to a maximum width of obstruction of 7.5 feet. Hi-Dro Cell Clusters are not usually used to shield obstructions wider than 3 to 4 feet. The maximum width of an obstruction that may be shielded by the G.R.E.A.T. system is approximately 3 feet.

Crash cushions are not ordinarily used along the length of an obstruction. Usually other types of traffic barriers are used. Figure 9-A shows typical installations where a crash cushion is used in conjunction with a barrier curb.

9-02.2 SPACE REQUIREMENTS

Area occupied by the crash cushion:

The Hi-Dro Cell Cluster occupies the least amount of space. The G.R.E.A.T., Hi-Dro Cell Sandwich and Hi-Dri Cell Sandwich will usually require about 25 percent more length than an inertial barrier. To meet the requirements of Figure 9-C, inertial barriers will have a minimum width of 6 feet. The Hi-Dro Cell sandwich and Hi-Dri Cell Sandwich have a minimum width of about 4 feet and vary to a maximum of about 8.5 feet. The G.R.E.A.T. occupies a minimum width of 2 feet. Figures 9-D, 9-E, and 9-F indicate the lengths of the Hi-Dro Cell, Hi-Dri Cell and G.R.E.A.T. system required to satisfy the allowable deceleration forces noted in 9-02.7.

Clearance from traveled way to crash cushion:

Figure 9-G shows showed dimensions for an impact attenuator reserve area. Where possible, the clearances recommended therein should be adhered to. However, it is recognized that often it will not be possible to provide the recommended clearances, particularly on existing roadways, in which case, the crash cushion should be designed so as not to encroach into the shoulder. In extreme cases, where the crash cushion must encroach into the

SELECTION PROCESS

FIGURE: 9-B

DATE: 11/83

FACTORS TO BE EVALUATED	CRASH CUSHION SYSTEM			
	INTERIAL BARRIERS	HI-DRO * CELL SANDWICH	HI-DRO CELL CLUSTER	HI-DRI * CELL SANDWICH
A. Dimensions of the Obstruction				
B. Space Requirements				
C. Geometrics of Site				
D. Physical Conditions of Site				
E. Redirection Characteristics				
F. Maximum Impact Speed				
G. Allowable Deceleration Force				
H. Back-up Structure Requirements				
I. Anchorage Requirements				
J. Flying Debris Characteristics				
K. Initial Cost				
L. Maintenance Cost				

Scoring Notations
- Acceptable - OK
- Not Acceptable - OUT
- Marginal - ?

*When there is no basis on which to make a choice between a Hi-Dro Cell Sandwich and a Hi-Dri Cell Sandwich, a Hi-Dro Cell Sandwich shall be used.

shoulder, a reusable crash cushion should be given serious consideration since a higher than normal frequency of impacts could reasonably be expected when the crash cushion is so close to the traveled way.

9-02.3 GEOMETRICS OF THE SITE

The vertical and horizontal alignment, especially curvature of the road and sight distance, are important factors to be considered. Adverse geometrics could contribute to a higher than normal frequency of impacts.

9-02.4 PHYSICAL CONDITIONS OF THE SITE

The presence of a curb can seriously reduce the effectiveness of a crash cushion. While new curbs should not be built where crash cushions are to be installed, it is not essential to remove existing curbs less than 4 inches in height. Curbs from 4 to 6 inches in height should be removed unless consideration of the curb shape, site geometry, imminency of overlays that would reduce the curb height, and cost of removal indicates the appropriateness of allowing the curb to remain. Curbs over 6 inches high should be removed before installing a crash cushion. When a curb is terminated behind a crash cushion, the terminal should be gently flared and/or ramped. Flares of 15 to 1 and ramps of 20 to 1 are appropriate on high speed facilities.

All crash cushions should be placed on a concrete or asphalt surface. The concrete should have a minimum thickness of 4 inches. The asphalt should be capable of supporting 5 psi.

Longitudinal and transverse slopes in excess of 5 percent could adversely affect the performance of a crash cushion and should be avoided.

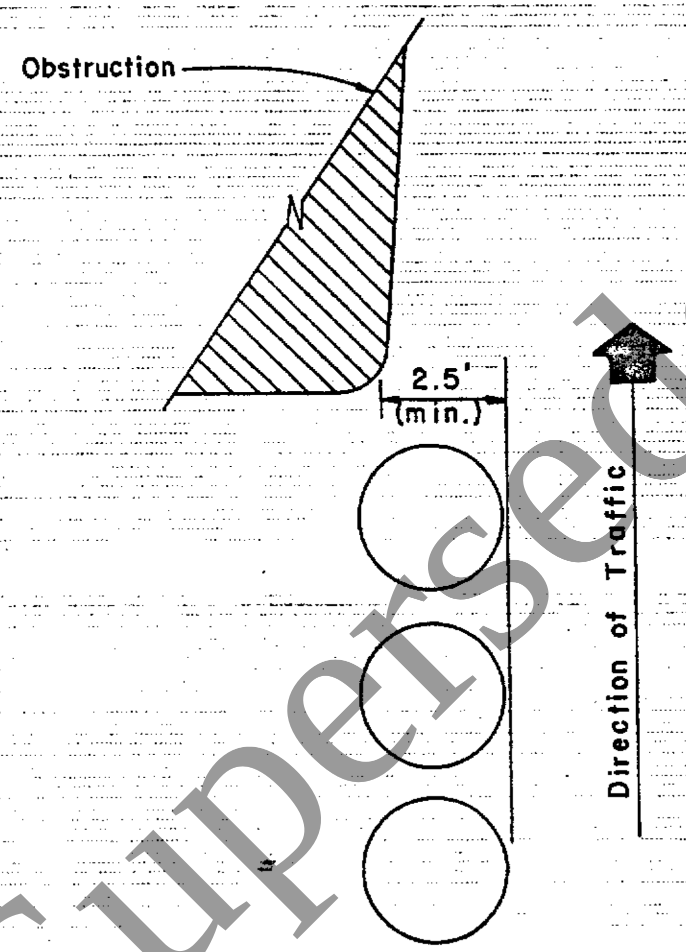
Joints, especially expansion joints, in the crash cushion area may require special design accommodations for those crash cushions that require anchorage.

9-02.5 REDIRECTION CHARACTERISTICS

The G.R.E.A.T., Hi-Dro Cell Sandwich and the Hi-Dri Cell Sandwich have redirection capabilities.

Since sandfilled plastic barrels and Hi-Dro Cell Clusters have no redirection capabilities, it is important that the recommended placement details shown in Figure 9-C be adhered to so as to minimize the danger of a vehicle penetrating the barrier from the side and hitting the obstructing.

SUGGESTED LAYOFF FOR LAST THREE EXTERIOR MODULES IN AN INERTIAL BARRIER	FIGURE: 9-C
	DATE: 11/83



9-02.6 MAXIMUM IMPACT SPEED

The Hi-Dro Cell Cluster should not be used where speeds will exceed 45 mph. The other approved crash cushions can be designed for any reasonable speed.

9-02.7 ALLOWABLE DECELERATION FORCE

Where practical, crash cushions should be designed for an average deceleration force of 6 g's. Where space is limited, a crash cushion may be designed for a maximum of 12 g's.

9-02.8 BACK-UP STRUCTURE REQUIREMENTS

Hi-Dro

The G.R.E.A.T. Cell Sandwich, Hi-Dro Cell Cluster and Hi-Dri Cell Sandwich require a back-up structure that is capable of withstanding the forces of an impact. Figures 9-D, 9-E and 9-F show the load on the back-up structure for various models of the Hi-Dro Cell Sandwich and Hi-Dri Cell Sandwich. If the obstruction is not capable of withstanding these forces, it must be modified or a separate back-up structure added.

9-02.9 ANCHORAGE REQUIREMENTS

The G.R.E.A.T., Hi-Dro Cell Sandwich and Hi-Dri Cell Sandwich require an anchorage which is capable of restraining the crash cushion during an impact. The manufacturers' standard designs of these crash cushions include the necessary anchorage.

9-02.10 FLYING DEBRIS CHARACTERISTICS

Impact with an inertial barrier will produce some flying debris. However, this is not considered a serious drawback.

9-02.11 INITIAL COST

The inertial barriers have the lowest initial cost. Compared to inertial barriers, Hi-Dro Cell Clusters have a moderate initial cost, the G.R.E.A.T. and Hi-Dro Cell Sandwich and Hi-Dri Cell Sandwich have a high initial cost. Assuming that about the same site preparations are required, the initial cost of a Hi-Dro Cell Sandwich or a Hi-Dri Cell Sandwich will usually be 5 to 6 times as an inertial barrier.

9-02.12 MAINTENANCE COST

Very little routine maintenance is required for any of the approved crash cushions. Restitution costs for damage from minor impacts and vandalism varies. Inertial barriers are particularly susceptable to damage from minor impacts. The G.R.E.A.T. and the Hi-Dri Cell Sandwich are the least susceptible to vandalism. Restitution cost for major impacts will usually be substantially

DESIGN DATA TABLE
HI-DRO CUSHION

FIGURE: 9-D

DATE: 11/83

No. of Bays	Velocity	40	45	50	55	60	65	70
4	Av. G Load	7.15	9.06	11.18	13.53	16.10	18.89	21.92
*9'-11½"	Kips (Peak)	50	65	80	95	115	135	155
5	G's	5.83	7.39	9.12	11.04	13.13	15.41	17.85
12'- 2½"	Kips	40	55	65	80	95	110	130
6	G's	4.93	6.24	7.70	9.32	11.08	13.01	15.07
14'- 5½"	Kips	35	45	55	65	70	80	110
7	G's	4.26	5.40	6.66	8.06	9.60	11.26	13.05
16'- 8½"	Kips	30	40	50	60	70	80	95
8	G's	3.76	4.76	5.87	7.11	8.46	9.92	11.50
18'-11½"	Kips	25	35	40	50	60	70	85
9	G's	3.35	4.25	5.28	6.35	7.56	8.87	10.28
21'- 2½"	Kips	25	30	40	45	55	65	75
10	G's	3.04	3.84	4.74	5.74	6.83	8.02	9.29
23'- 5½"	Kips	20	25	35	40	50	60	70
11	G's	2.77	3.50	4.33	5.24	6.24	7.31	8.49
25'- 8½"	Kips	20	25	30	35	45	55	60
12	G's		3.22	3.98	4.82	5.73	6.73	7.81
27'-11½"	Kips		25	30	25	45	50	55

*Total Length of unit as measured from front face of backup to forward edge of front cells.
Values shown in table above are based on 75% efficiency. Actual efficiency is in excess of 83%.

Source: Energy Absorption Systems, Inc., Design Data

DESIGN DATA TABLE
HI-DRI CUSHION

FIGURE: 9-E

DATE: 11/83

No. of Bays	Velocity	40	45	50	55	60	65	70
4	Av. G Load	6.43	8.15	10.06	12.18	14.43	17.00	19.73
*10'-5½"	Kips (Peak)	45	55	70	85	100	115	135
5	G's	5.25	6.65	8.21	9.94	11.82	13.87	16.06
12'-8½"	Kips	35	45	55	65	80	95	110
6	G's	4.44	5.62	6.93	8.38	9.97	11.71	13.56
14'-11½"	Kips	30	40	45	55	65	80	90
7	G's	3.83	4.86	6.00	7.25	8.64	10.13	11.75
17'-2½"	Kips	25	35	40	50	60	70	80
8	G's	3.38	4.28	5.28	6.40	7.61	8.93	10.35
19'-5½"	Kips	25	30	35	45	50	60	70
9	G's	3.02	3.83	4.75	5.72	6.80	7.98	9.25
21'-8½"	Kips	20	25	30	40	45	55	65
10	G's	2.74	3.46	4.26	5.17	6.15	7.22	8.36
23'-11½"	Kips	20	25	30	35	40	50	55
11	G's	2.49	3.15	3.90	4.72	5.62	6.58	7.64
26'-2½"	Kips	20	20	25	30	40	45	50
12	G's		2.89	3.58	4.33	5.16	6.06	7.03
28'-5½"	Kips		20	25	30	35	40	45

*Total length of unit as measured from front face of backup to forward edge of front cartridge.
Values shown in table above are based on 85% efficiency.

Source: Energy Absorption Systems, Inc., Design Data

DESIGN DATA TABLE
G.R.E.A.T. SYSTEM

FIGURE: 9-F

DATE: 11/83

No. of Bays	Velocity (MPH)	15	20	25	30	35	40	45
1	Average G-Load	1.63	2.90	4.54	6.54	8.90	11.62	14.70
5'9"	KIPS (peak)	11.1	19.7	30.9	44.5	60.5	79.0	100.0
2	G's	1.07	1.91	2.98	4.29	5.85	7.63	9.66
8'9"	KIPS	7.3	13.0	20.3	29.2	39.8	51.7	65.7
3	G's	.80	1.42	2.22	3.20	4.35	5.69	7.20
11'9"	KIPS	5.4	9.7	15.1	21.8	29.6	38.7	49.0

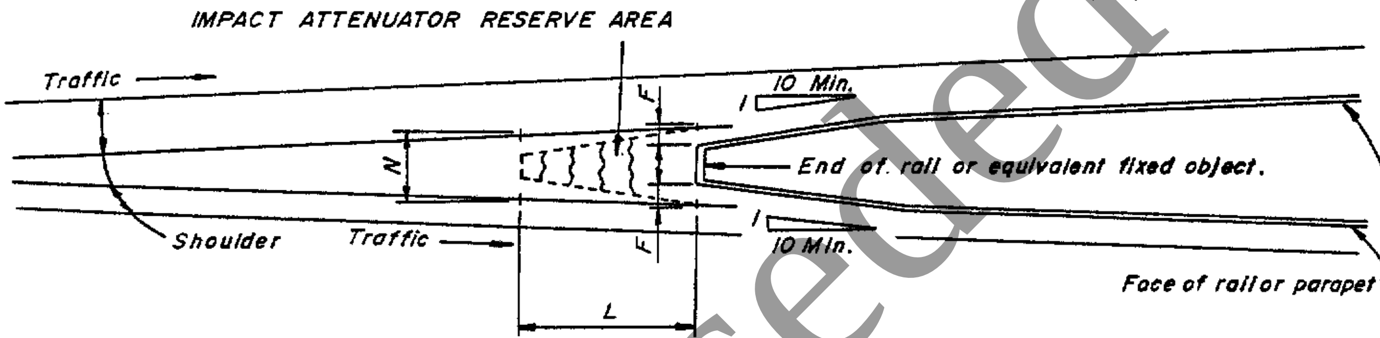
No. of Bays	Velocity	45	50	55	60	65	70
3	G's	7.20	8.88	10.75	12.79	--	--
11'9"	KIPS	49.0	60.4	73.1	87.0	--	--
4	G's	5.73	7.08	8.57	10.19	11.97	13.88
14'9"	KIPS	39.0	48.1	57.6	69.3	81.4	94.4
5	G's	4.76	5.88	7.12	8.47	9.94	11.53
17'9"	KIPS	32.4	40.0	48.4	57.6	67.6	78.4
6	G's	4.07	5.03	6.09	7.24	8.51	9.86
20'9"	KIPS	27.7	34.2	41.4	49.2	57.9	67.0
7	G's	3.56	4.40	5.32	6.33	7.43	8.62
23'9"	KIPS	24.2	29.9	36.2	43.0	50.5	58.6
8	G's	3.16	3.90	4.72	5.62	6.60	7.65
26'9"	KIPS	21.5	26.5	32.1	38.2	44.9	52.0
9	G's	2.84	3.51	4.25	5.05	5.93	6.88
29'9"	KIPS	19.3	23.9	28.9	34.3	40.3	46.8
10	G's	2.58	3.18	3.80	4.59	5.39	6.25
32'9"	KIPS	17.5	21.6	26.2	31.2	36.7	42.5

Source: Energy Absorption Systems, Inc., Design Data

IMPACT ATTENUATOR RESERVE AREA DETAILS

FIGURE: 9-G

DATE: 11/83



DESIGN SPEED ON MAINLINE (M.P.H.)	DIMENSIONS FOR IMPACT ATTENUATOR RESERVE AREA ON NEW CONSTRUCTION (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

higher for inertial barriers. Energite and Fitch inertial barrels are interchangeable in any array.

9-03 DESIGN PROCEDURE

9-03.1 FITCH INERTIAL BARRIER AND ENERGITE INERTIAL BARRIER

The design of an inertial barrier is based on the law of conservation of momentum. It can be shown that:

$$V = V_o \left(\frac{W}{W + W_s} \right) \qquad \text{Equation 1}$$

V = velocity of vehicle after impact with Ws in fps
Vo = velocity of vehicle prior to impact with Ws in fps
W = weight of vehicle in lbs.
Ws = weight of sand actually impacted by a 6 ft. wide vehicle in lbs.

This equation is used to calculate the velocity of a vehicle as it penetrates the inertial barrier. When a vehicle has been slowed to 10 mph + 5 mph (14.7 fps + 7.3 fps) per Equation 1, it will actually have been stopped because of deceleration forces that have been neglected in Equation 1.

Slowing of the vehicle must take place gradually so that the average deceleration force is 6g desirable, 12g maximum. The average deceleration force is calculated using Equation 2. Note: Velocity is in f.p.s.

$$G\ \text{average} = (V_s^2 - V_f^2)/64.4D \qquad \text{Equation 2}$$

G average = the average deceleration force in g's

Vs = design speed (fps)

Vf = final speed (fps)

D = distance traveled in decelerating from Vs to Vf

Example of inertial barrier design:

Step 1. Find minimum stopping distance:

Rearranging Equation 2 gives

$$D = (V_s^2 - V_f^2)/64.4G$$

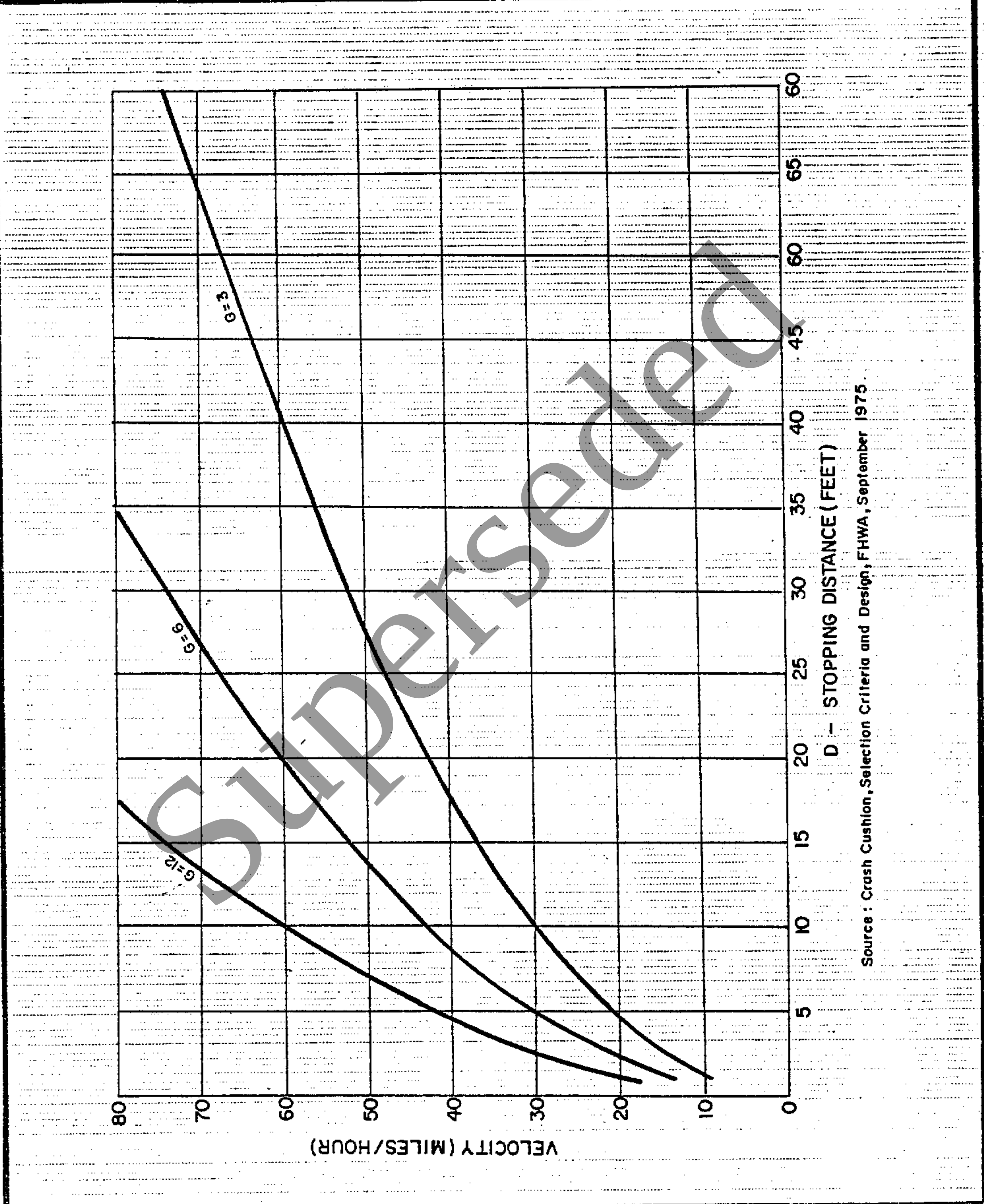
for Vs = 88 fps (60 mph), Vf = 0, G = 6, then D = 20 feet.

D can also be obtained directly from Figure 9-H.

MINIMUM STOPPING DISTANCE

FIGURE: 9-H

DATE : 11/83



Source : Crash Cushion, Selection Criteria and Design, FHWA, September 1975

Step 2. Assume a configuration:

See Figure 9-I & 9-J for typical configurations

A minimum of 2 modules are required in the last 3 rows to meet the 2'-6" criteria shown in Figure 9-C. The last row should be 1400 lb. modules to reduce the chances of a vehicle penetrating to the obstruction. When a wide obstruction is being shielded, the modules may be spaced up to 3 feet apart. However, this spacing must be accounted for in the design. Ws in Equation 1 is the weight of sand impacted by a 6 ft. wide vehicle. Therefore, if the above 1400 lb. modules were spaced 2 ft. apart, Ws would equal 1867 lbs.

Step 3. Check configuration for 1900 lb. vehicle:

Assume design speed of 60 mph for this illustration.

ROW	Ws	Vo	V*	G*
1	200	88	79.6	7.3
2	200	79.6	72.0	6.0
3	400	72.0	59.5	8.5
4	700	59.5	43.5	8.5
5	1400	43.5	25.0	6.6
6	1400	25.0	14.4	2.2
7	2800	14.4	5.82	0.9

V and G are calculated using Equations 1 & 2.V ~~&~~ can be obtained directly from Figures 9-K.

Figure 9-F shows V and G for various configurations and design speeds.

**It is desirable to limit G for each row to a maximum of 6. However, since 200 pounds is the lightest module recommended for use, the 7.6 cannot be decreased.

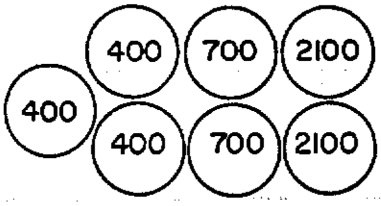
Step 4. Check configuration for 4500 lb. vehicle:

ROW	Ws	Vo	V	G
1	200	88.0	84.3	3.3
2	200	84.3	80.7	3.1
3	400	80.7	74.1	5.3
4	700	74.1	64.1	7.2
5	1400	64.1	48.9	8.9
6	1400	48.9	37.3	5.2
7	2800	37.3	23.0	4.5
8	2800	23.0	14.2	1.7

TYPICAL SAND BARREL CONFIGURATION

FIGURE: 9-I

DATE: 11/83

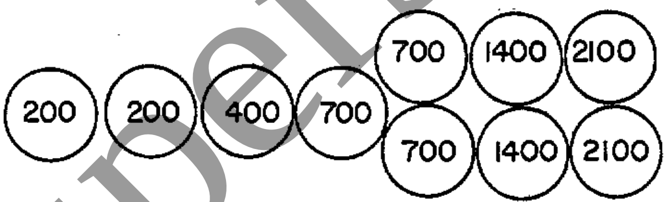


40 MPH DESIGN - 1800 LB. VEHICLE

ROW	Weight (lb)	Vs	V	G
1	400	58.7	48.0	5.9
2	800	48.0	33.2	6.2
3	1400	33.2	18.7	3.9
4	4200	18.7	5.6	1.6

40 MPH DESIGN - 4500 LB. VEHICLE

ROW	Weight	Vs	V	G
1	400	58.7	53.8	2.8
2	800	53.8	45.8	4.2
3	1400	45.8	34.9	4.5
4	4200	34.9	18.9	4.4



50 MPH DESIGN - 1800 LB. VEHICLE

ROW	Weight (lbs.)	Vs	V	G
1	200	73.3	66.0	5.3
2	200	66.0	59.4	4.2
3	400	59.4	48.6	6.0
4	700	48.6	35.0	5.9
5	1400	35.0	19.7	4.3
6	2800	19.7	7.7	1.7
7	4200	---	---	---

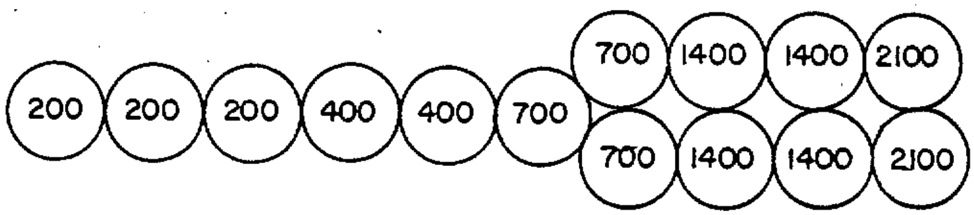
50 MPH DESIGN - 4500 LB. VEHICLE

ROW	Weight	Vs	V	G
1	200	73.3	70.2	2.3
2	200	70.2	67.2	2.1
3	400	67.2	61.7	3.7
4	700	61.7	53.4	4.9
5	1400	53.4	40.7	6.2
6	2800	40.7	25.1	5.3
7	4200	25.1	13.0	2.4

TYPICAL SAND BARREL CONFIGURATION

FIGURE: 9-J

DATE: 11/83



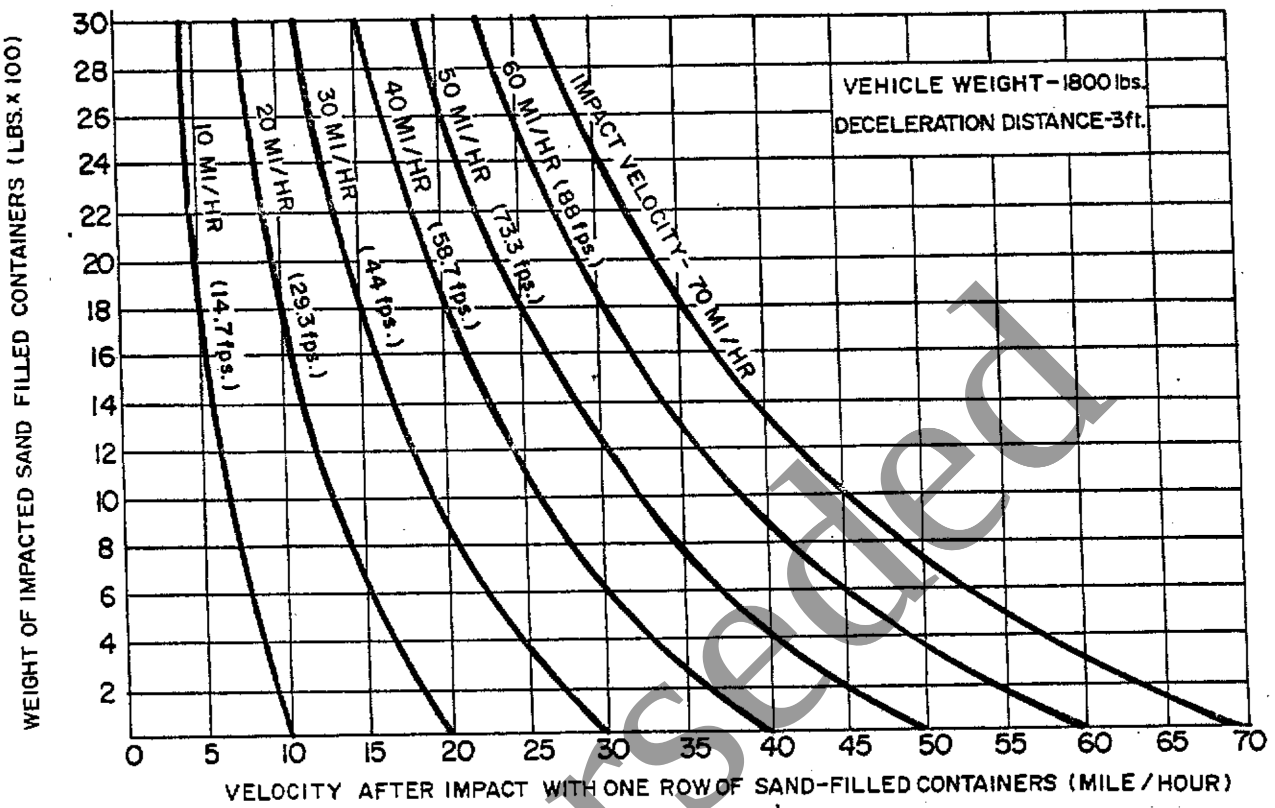
60 MPH DESIGN 1800 LB. VEHICLE

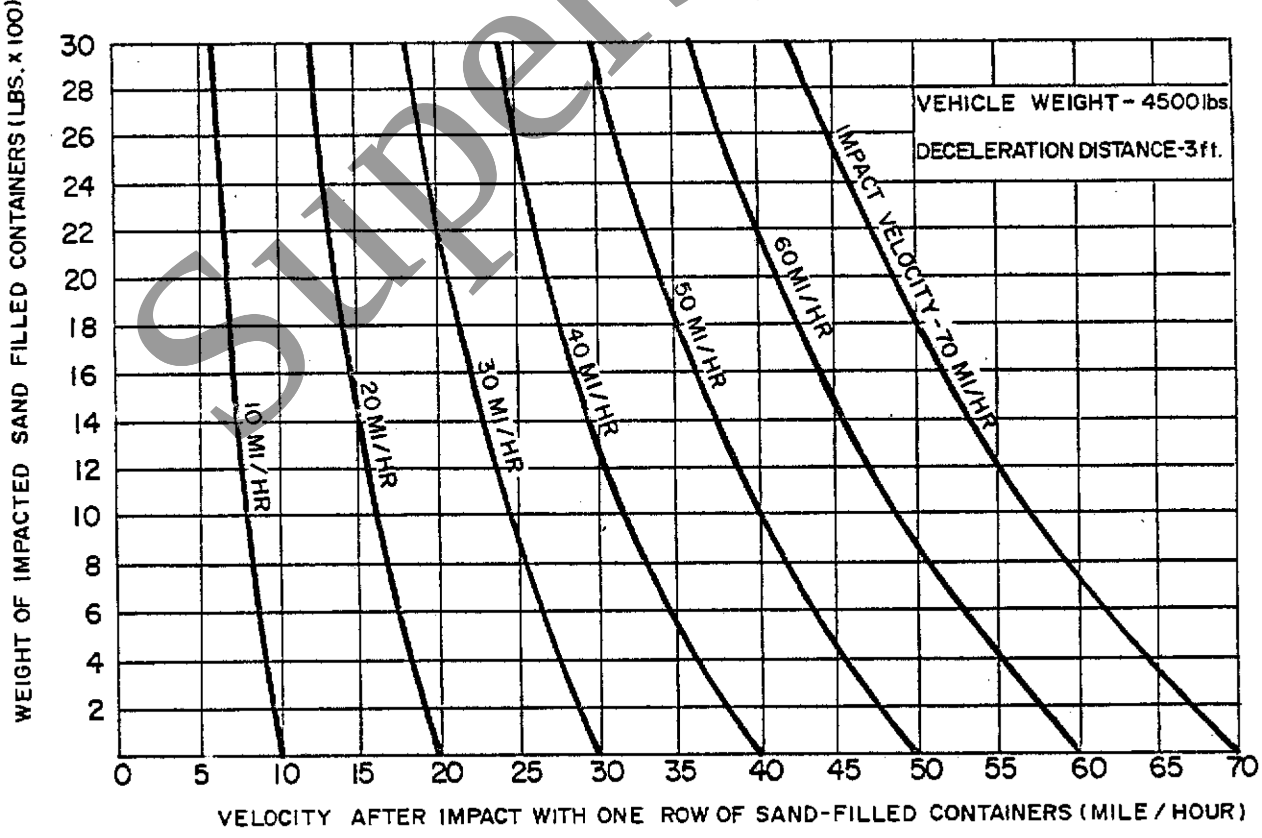
ROW	Ws	Vo	V	G
1	200	88.0	79.2	7.6
2	200	79.2	71.3	6.2
3	200	71.3	64.2	5.0
4	400	64.2	52.5	7.1
5	400	52.5	47.3	2.7
6	700	47.3	34.0	5.5
7	1400	34.0	19.1	4.1
8	1400	19.1	10.7	1.3
9	2800	---	---	---
10	4200	---	---	---

60 MPH DESIGN 4500 LB. VEHICLE

ROW	Ws	Vo	V	G
1	200	88.0	84.3	3.3
2	200	84.3	80.7	3.1
3	200	80.7	77.2	2.9
4	400	77.2	70.9	4.8
5	400	70.9	65.1	4.1
6	700	65.1	56.3	5.5
7	1400	56.3	43.0	6.8
8	1400	43.0	32.8	4.0
9	2800	32.8	20.2	3.5
10	4200	20.2	10.5	1.5

CHANGE IN VELOCITY FOR SAND-FILLED CONTAINERS	FIGURE: 9-K
	DATE: 11/83





Source: Crash Cushions, Selection Criteria and Design, FHWA September 1975.

Step 5. Since the assumed configuration meets all the requirements specified, no changes are necessary.

Manufacturers of inertial barriers have developed designs for various obstructions. Most of these designs are based on a maximum average deceleration force of 6g's. However, the space required for a 6g design will not always be available, especially in gore areas, in which case, a design for higher deceleration forces (12g's maximum) may be used.

9-03.2 HI-DRO CELL SANDWICH, HI DRI CELL SANDWICH AND THE G.R.E.A.T.

Because of the complex reaction of these crash cushion to an impact, a simple design procedure is not possible. The manufacturer has developed several standard arrangements. Figures 9-D, 9-E and 9-F show the dimensions and operational characteristics of the standard models. Custom models can be designed but the costs thereof are very high. Standard designs for back-up structures are available from the manufacturer.

9-03.3 HI-DRO CELL CLUSTER

The Hi-Dro Cell Cluster occupies the least amount of space of any of the approved crash cushions. However, the Hi-Dro Cell Cluster is only approved for vehicle speeds up to 45 mph. The Cluster may be arranged to suit the object being shielded. Standard designs for back-up structures are available from the manufacturer.

Example of Hi-Dro Cell Cluster design:

design speed = 30 mph (44 fps)

minimum stopping distance $\frac{V^2}{64.4(G)} = \frac{44^2}{64.4(G)} = 5.01$

Each cell is 6", therefore, use 10 rows. Usually a minimum of 5 cells should be used in each row, however, the number of cells per row will vary depending on the dimensions of the obstruction.