HIGHWAY DESIGN MANUAL

Chapter 6 - Interchanges

Revision 12

June 1979

CHAPTER 6.00

INTERCHANGES

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INTERCHANGES

6.01 DEFINITION

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.

6.02 WARRANTS FOR INTERCHANGES

6.02.01 FREEWAY DEVELOPMENT

Once it has been concluded to develop a route as a freeway, it must be determined whether each intersecting highway should be terminated, rerouted, or provided with a grade separation or interchange. For those crossroads that cannot be terminated, the individual warrant for a separation or interchange is absorbed in the decision to develop the freeway. The chief concern is the continuous flow on the major road. If traffic on the minor road is to cross the freeway, a grade separation or interchange is provided. Thus, an intersection that might warrant only traffic signal control, if considered as an isolated case, will warrant a grade separation or interchange when considered as part of a freeway.

6.02.02 ELIMINATION OF BOTTLENECKS OR SPOT CONGESTION

Insufficient capacity at the intersection of heavily travelled highways results in intolerable congestion on one or all approaches. Inability to provide the essential capacity with an at-grade facility provides the warrant for an interchange.

6.02.03 ELIMINATION OF HAZARD

Some at-grade intersections have a disproportionate share of serious accidents. Lacking inexpensive methods of eliminating hazards, a highway grade separation or interchange may be warranted. Accident prone intersections frequently are found at the junction of comparatively lightly travelled highways in sparsely settled rural areas where speeds are high. In such areas, structures usually can be constructed at little cost compared to urban areas, right of way is not expensive, and these lower cost developments can be justified by the elimination of only a few serious accidents. Serious accidents at heavily travelled intersections, of course, also provide a warrant for interchange facilities. In addition to greater safety, the interchange also expedites all movements.

6.02.04 ROAD USER BENEFITS

The road user costs due to delays at congested at-grade intersections are large. Cost of fuel, tires, oil, repairs, time, accidents, etc., at intersections that require speed changes, stops, and waiting, is well in excess of that for intersections permitting uninterrupted or continuous operation. In general, interchanges require somewhat more total travel distance than direct crossings at grade, but the added cost of the extra travel distance is less than the saving in cost affected by the reduction in stopping and delay costs. For any type of intersection, the relation of road user benefits to the cost of improvement is indicative of an economic warrant for that improvement. For convenience the relation is expressed as a ratio, the annual benefit divided by the annual capital cost of the improvement. Annual benefit is the difference in road user costs for the existing condition and that for the condition after improvement. Annual capital cost is the sum of interest and amortization for the cost of the improvement. The larger the ratio, the greater is the justification insofar as road user benefits are concerned. Comparison of these ratios for design alternates is an important factor in determining the type and extent of improvement to be made. If used not as a comparison of alternates index but for justifying a single project or design, a ratio in excess of one is necessary for minimum economic justification.

For details concerning computations and analyses of Road User Benefits, see 1960 A.A.S.H.O. pamphlet "Road User Benefit Analyses for Highway Improvements."

6.03 TYPES OF INTERCHANGES

6.03.01 THREE-LEG INTERCHANGES

An interchange at an intersection with three intersection legs consists of one or more highway grade separations and generally one-way roadways for all traffic movements. When two of the three intersection legs form a through road and the angle of intersection is not acute, the term T interchange applies. When all three intersection legs have a through character or the intersection angle with the third intersection leg is small, the interchange may be considered a Y type. A clear line of distinction between the T and Y types is not necessary or important. Regardless of the intersection angle, through road character, etc., any one basic interchange pattern may apply for widely variant conditions. See Figure 6-A for examples of three-leg interchanges.

6.03.02 FOUR-LEG INTERCHANGES

A. RAMPS IN ONE QUADRANT

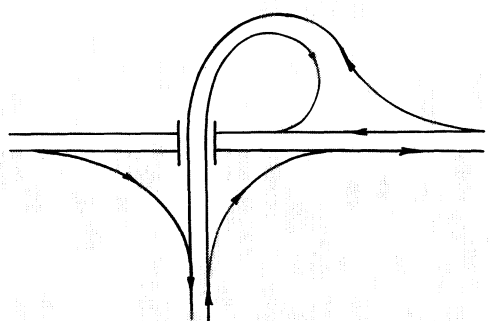
Interchanges with ramps in only one quadrant have application for intersections of roads with low volumes. Where a grade separation is provided at an intersection because of topography, even though volumes do not justify the structure, a single two-way ramp of near minimum design usually will suffice for all turning traffic. The ramp terminals may be plain T intersections. Locations where designs of this type are applicable are very limited.

At some interchanges, it may be necessary to limit ramp development to one quadrant due to topography, culture or other controls, even though the traffic volume justifies more extensive turning facilities. With ramps in one quadrant only, a high degree of channelization at the terminals and the median, together with left turn lanes on through facilities, normally are required to properly control turning movements, except when a through road is only two lanes wide. In other instances, a one-quadrant interchange may be constructed as the first step in a stage construction program. In this case, the initial ramps should be designed as a part of the ultimate development.

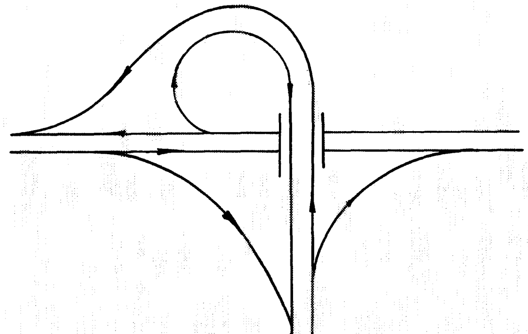
Figure 6-B shows an example of an interchange with ramps in one quadrant.

B. DIAMOND

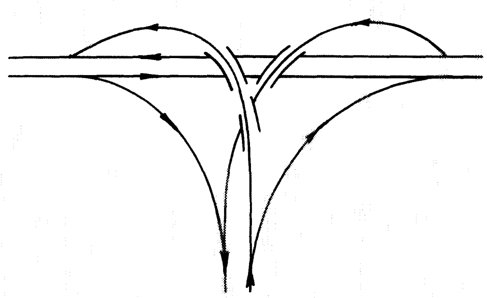
A full diamond interchange is formed when a one-way diagonal type ramp is provided in each quadrant. The ramps are alined for flat-angle terminals on one through highway and the left turns at grade are confined to the other (minor) highway. The diamond interchange has several advantages over a comparable partial cloverleaf. All traffic can enter and leave the major road at a relatively high



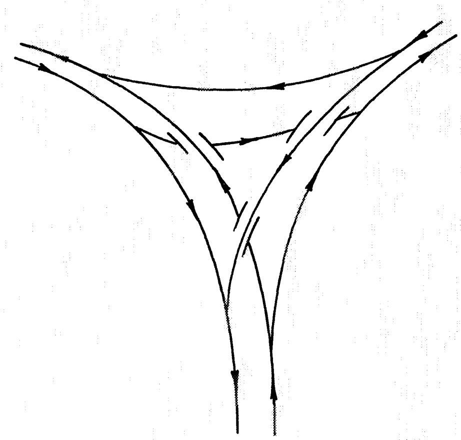
TRUMPET - A



TRUMPET - B

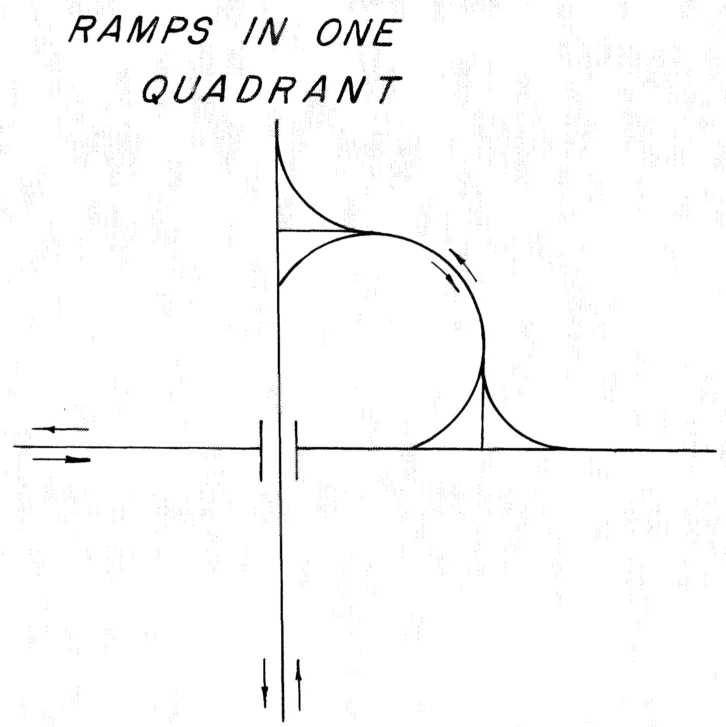


DIRECTIONAL -T



DIRECTIONAL -Y

NEW YORK STATE DEPT. OF TRANSPORTATION	THREE - LEG INTERCHANGES	FIG. 6 - A



NEW YORK STATE DEPT. OF TRANSPORTATION	FOUR-LEG INTERCHANGES	FIG. 6-B

speed. Left turning maneuvers entail little extra travel. A relatively narrow band of right of way is required, sometimes no more than that for the highways themselves.

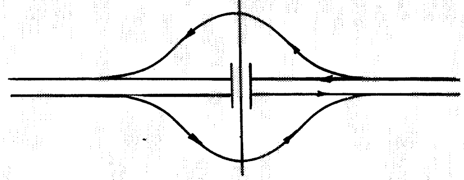
Diamond interchanges have application in both rural and urban areas. They are particularly adaptable to major-minor crossings, where left turns at grade on the minor road are fitting and can be handled without particular hazard or difficulty (See Section 5.06.07 for warrants for left turn slots). Where steep grades would cause a breaking problem for thru traffic on the crossroad and/or sight distance on the crossroad is less than that required for the crossroad design speed, provide left turn slots turns into ramps regardless of warrants. The intersections on the minor road formed by the terminals function like any other T intersections at grade. However, because these intersections have four legs, two of which are one way, they present a problem in traffic control to prevent wrong-way entry from the crossroad. Care should be taken in the design of diamond interchanges to avoid designs that may encourage wrong-way entry. In most cases additional signing to help prevent improper use of the ramps will have to be incorporated in the design of the interchange. Where traffic volumes at the minor road terminals are sufficient to require traffic signal control, pavement widening will be required on the ramps as well as on the crossroad through the interchange area. In most cases, a single-lane ramp serves traffic from the major highway; but it may have to be widened to two or three storage lanes at the minor road terminal in order to develop the necessary capacity for the at-grade condition. In rural areas, it is not practicable to handle more than about 800 vph on any one ramp of a diamond interchange. Examples of diamond interchanges are shown in Figure 6-C.

C. CLOVERLEAF

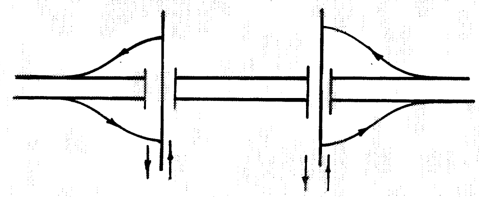
A full cloverleaf interchange is formed when a loop and an outer connection are provided in each quadrant. The cloverleaf is the only four-leg, single grade separation interchange pattern without terminal left turns at grade. Its use by drivers is well understood, and generally, it is utilized properly when both of the intersecting highways are divided through the interchange. Drivers desiring to turn left are required to travel beyond the point of through road intersection and turn right through about 270 degrees before gaining the desired direction. The principal disadvantages of the cloverleaf are the extra travel distance required for left turning traffic, the weaving maneuvers required, and the relatively large rights of way areas needed for it, particularly when designed to high standards. Cloverleaf interchanges with and without collector-distributor roads (C-D) are depicted in Figure 6-C.

D. PARTIAL CLOVERLEAF

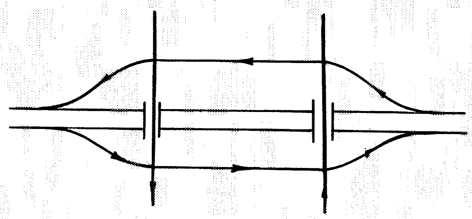
A partial cloverleaf (parclo) interchange is a cloverleaf layout where a full complement of ramps is not required or may not be attrainalbe due to site controls. Ramps should be so arranged that the entrance and exit turns impede the traffic flow on the major highway the least. To do this, the following controls or directions of turning on exits from and entrances to the highways should be considered in the arrangement of the ramps at partial cloverleafs.



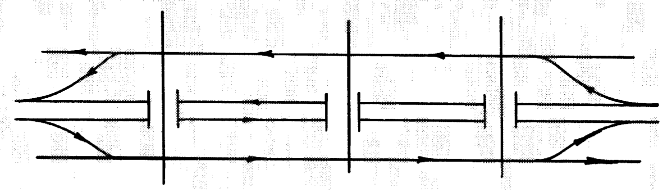
CONVENTIONAL DIAMOND



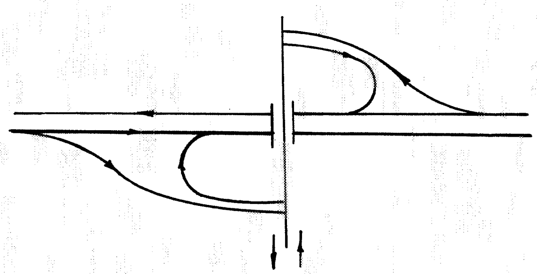
SPLIT DIAMOND



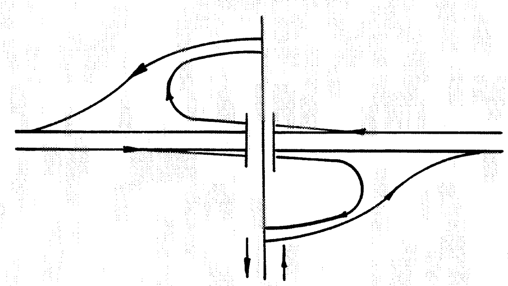
SPLIT DIAMOND WITH ONE-WAY CROSSROADS



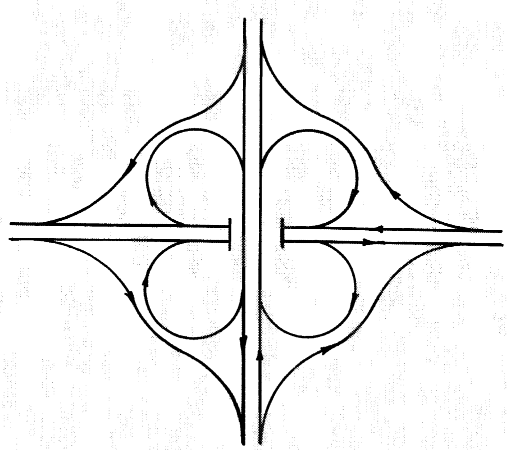
DIAMOND INTO ONE-WAY FRONTAGE ROADS



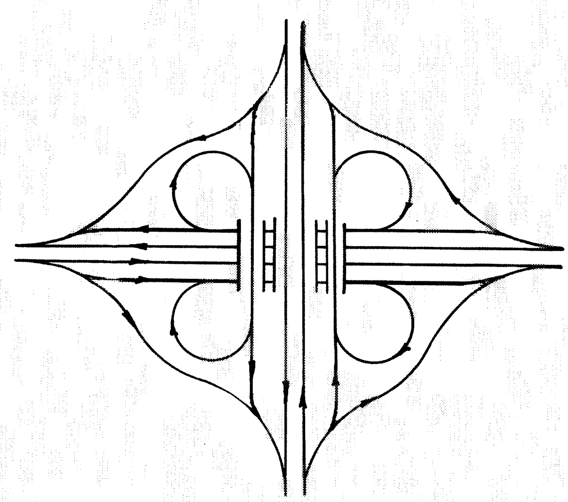
PARTIAL CLOVERLEAF



PARTIAL CLOVERLEAF



CLOVERLEAF



CLOVERLEAF WITH C-D ROADS

NEW YORK STATE DEPT. OF TRANSPORTATION	FOUR-LEG INTERCHANGES	FIG. 6-C

1. The ramp arrangement preferably should enable each turning movement to be made by right turn exits and entrances.

2. Where right turns are not feasible for both exits and entrances, and there is a choice between making either one a right turn, the exit should be chosen.

3. Where the traffic volume on a major highway is decidedly greater than that on the intersecting minor road, preference should be given to an arrangement of placing the right turns, either exit or entrance, on the major highway even though it results in a direct left turn off the minor road.

Figure 6-C shows examples of partial cloverleaf interchanges.

E. DIRECTIONAL

A directional interchange is one in which the ramp for one or more left turning movements is on directional alignment. These ramps are termed direct or semidirect connections in that they do not deviate much from the intended direction of travel.

A direct connection for a left turn movement is one in which the ramp exit and entrance terminals are located on the left side of the roadways (in the direction of travel). Both roadways must be divided in order to be able to provide direct left connections. On all types of interchanges the outer connections for right turns are also usually direct connections. A semidirect connection is one in which the ramp has the configuration of a jug handle. The driver is not required to travel 270 degrees to accomplish the left turn, as is the case in the inner loop ramp of the cloverleaf.

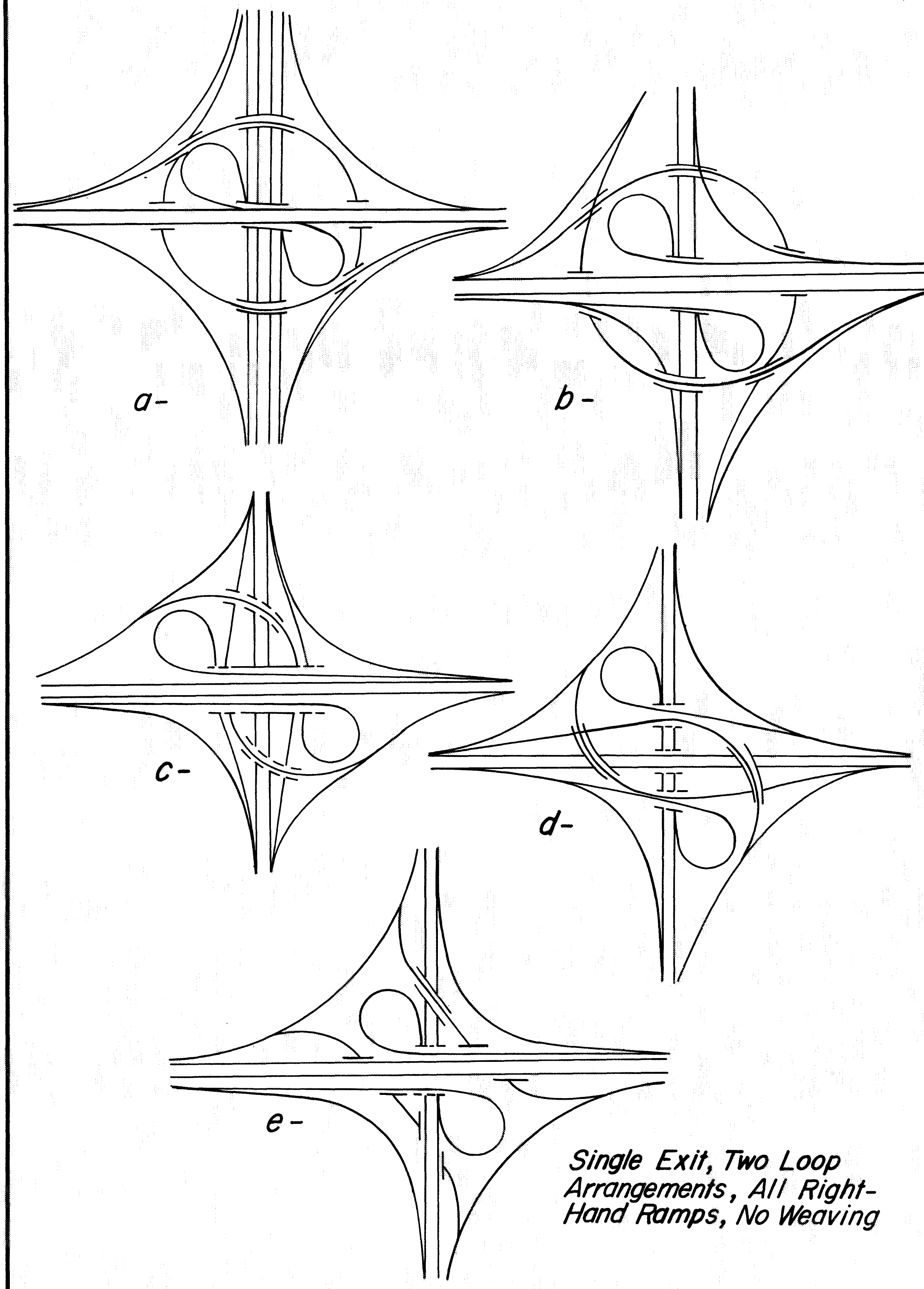
Direct or semidirect connections are used for important turning movements to reduce travel distance, increase speed and capacity, eliminate weaving, and avoid the loss of direction in driving a loop. Higher volumes can be accommodated on direct connections, and in some instances on semidirect ramps, rather than on loops because of relatively high speed and the likelihood of more adequate terminal design.

In rural areas there rarely is a volume justification for provision of direct connections in more than one or two quadrants. The remaining left turning movements usually are handled satisfactorily by loops or at-grade intersections. Directional interchanges when used at four-leg intersections always require more than one grade separation structure or a single structure with more than the conventional two levels. Many different forms of directional four-leg interchanges can be designed depending on topography, availability of right of way, relative traffic volumes and many other factors. They sometimes include left-hand exits, left-hand entrances, loops for minor moves and occasionally some weaves either on the interchange roadways or even on the main line. In view of the statements contained elsewhere in this chapter concerning the undesirability of left-hand exits and entrances and of weaves and the desirability of single-exit and, to a lesser extent, single-entrance designs, Figures 6-D, 6-E, 6-F and 6-G illustrate some examples of directional interchanges incorporating single-exit, all right-hand ramps, no weaving designs both with (semi-directional) and without (all-directional) loops.

6.03.03 ROTARY INTERCHANGES

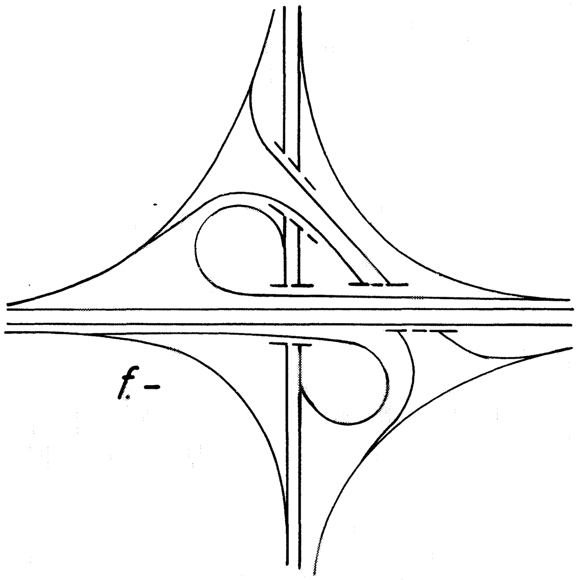
A rotary interchange is fitting where there are five or more intersection legs and all movements, other than through traffic on the principal highways, can be handled properly on the weaving sections. Two types of rotary interchanges are shown in Figure 6-H. A rotary intersection may be overpassed or underpassed by one of the intersecting highways. With two structures and four diagonal ramps, a complete interchange is provided. The weaving sections are a critical design feature of rotary interchanges. See Section 6.05.01 A.

Traffic leaving and entering the principal route does so directly as on a diamond interchange. Design features, operation, and capacity of the rotary are basically the same as that of an at-grade rotary. Rotary interchanges are not suitable where realtively high speeds are to be maintained on the crossroads. Normally, a rotary interchange need not occupy any more space than a cloverleaf.



Single Exit, Two Loop Arrangements, All Right-Hand Ramps, No Weaving

NEW YORK STATE DEPT. OF TRANSPORTATION	DIRECTIONAL INTERCHANGES	FIG. 6-D



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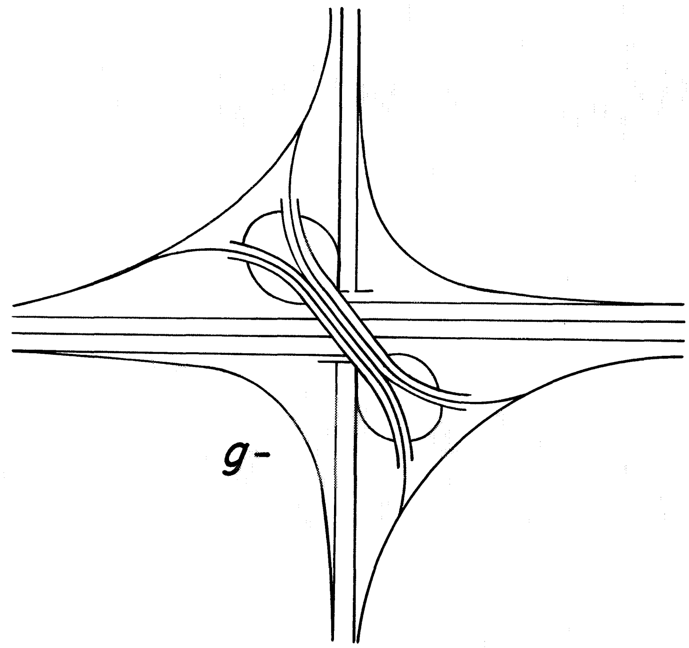

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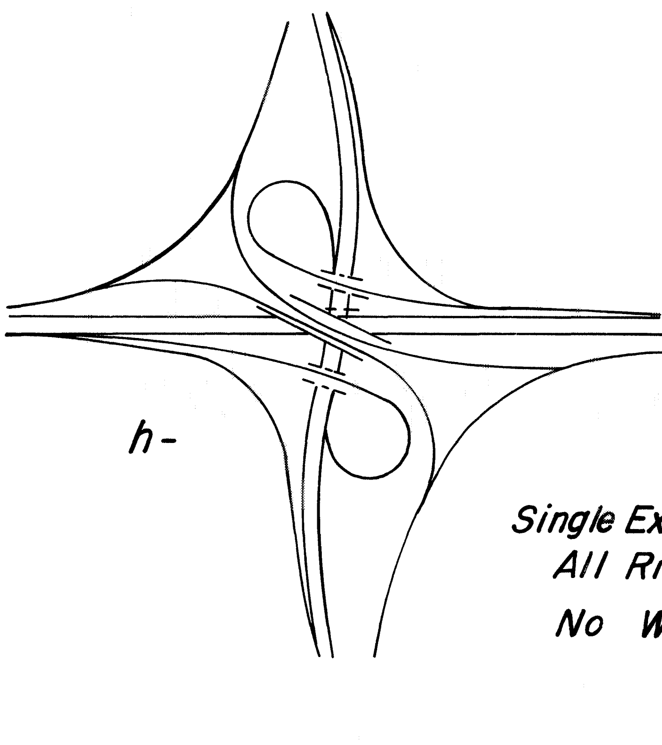

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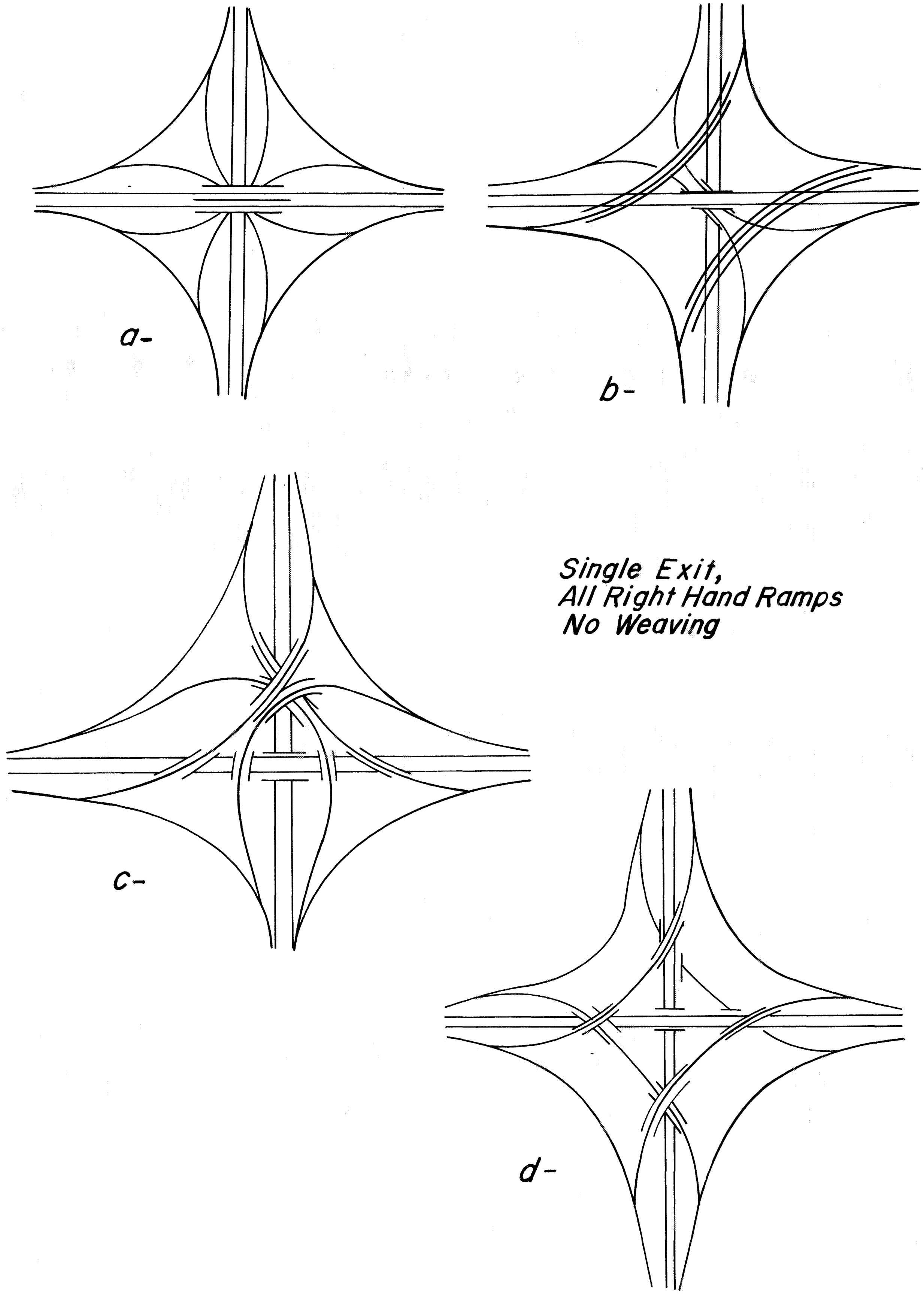
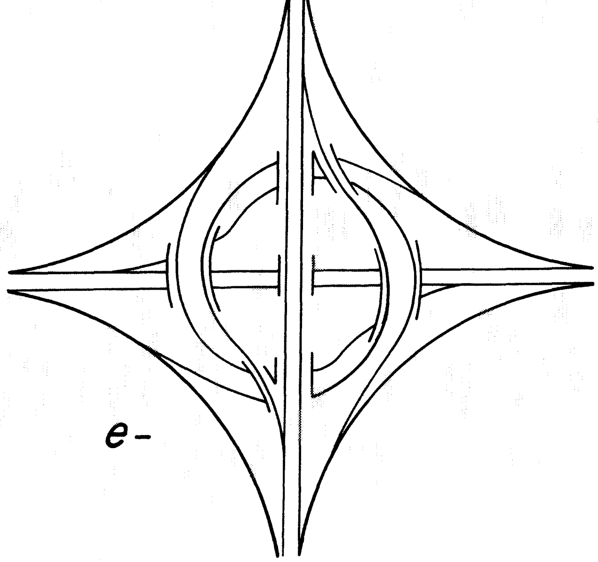
Single Exit, Two-Loop Arrangements
All Right Hand Ramps
No Weaving

NEW YORK STATE DEPT. OF TRANSPORTATION	DIRECTIONAL INTERCHANGES	FIG. 6-E



NEW YORK STATE DEPT. OF TRANSPORTATION	ALL DIRECTIONAL INTERCHANGES	FIG. 6-F



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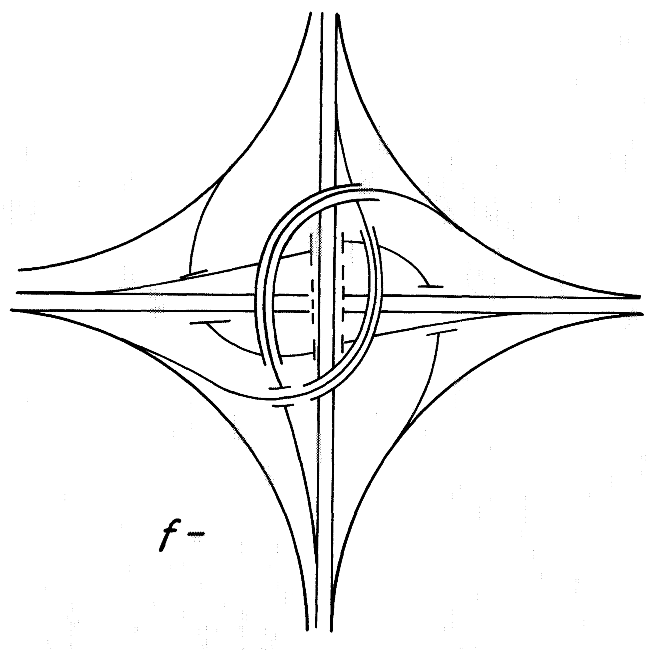

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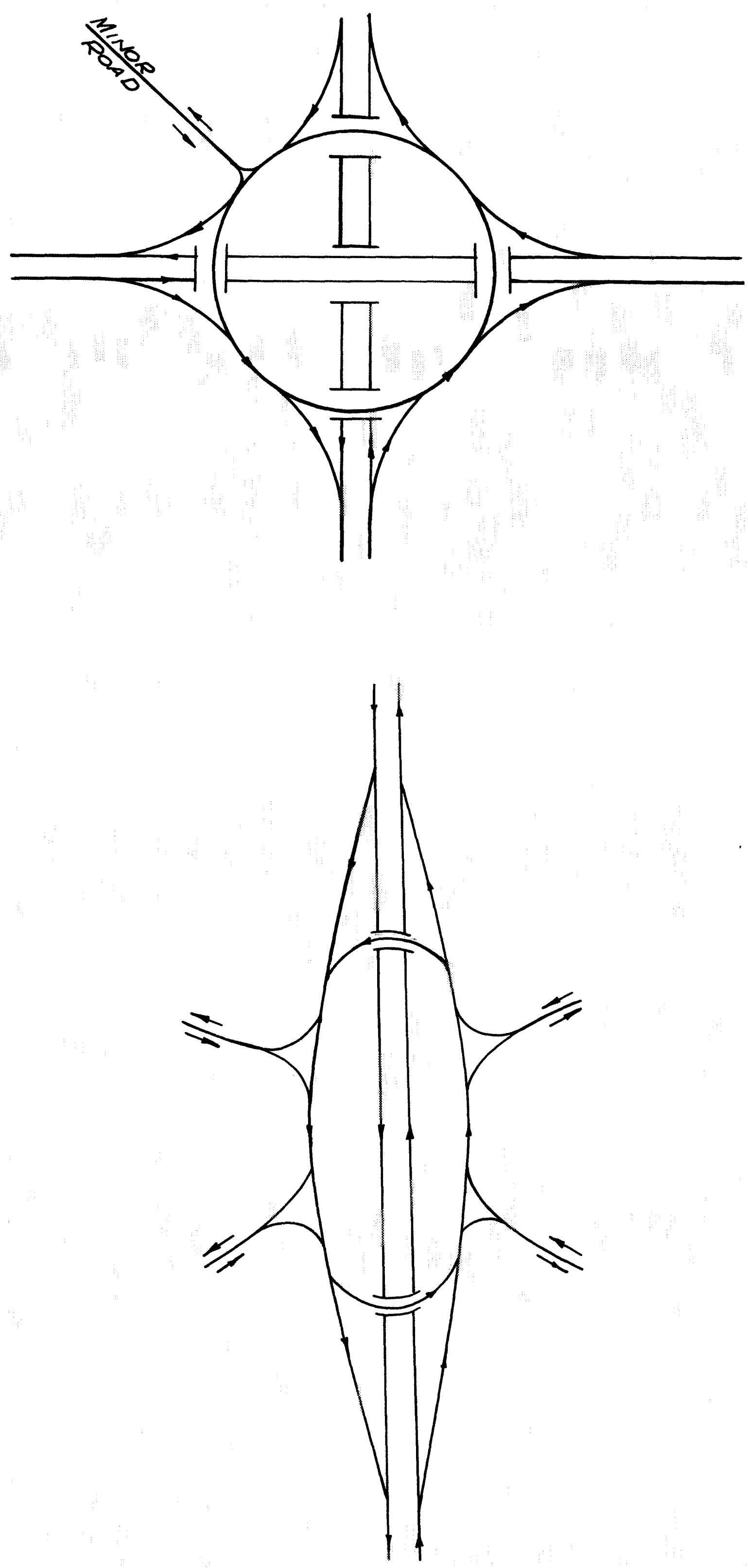
Single-Exit, All Right-Hand Ramps
No Weaving

NEW YORK STATE DEPT. OF TRANSPORTATION	ALL DIRECTIONAL INTERCHANGES	FIG. 6-G



NEW YORK STATE DEPT. OF TRANSPORTATION	ROTARY INTERCHANGES	FIG. 6-H

6.04 RAMPS

6.04.01 TYPES

Each ramp of an interchange is generally a one-way roadway, although two-way ramps sometimes are employed. Figure 6-I illustrates several types of ramps and their characteristic shapes.

Right-turning ramps are normally of direct connection variety, i.e., a ramp on which traffic turns right to go right. Such ramps are either curvilinear, diagonal, or reverse-curve. In all cases, operationally such ramps produce a right exit-right entrance situation. Diagonal ramps usually have both a left and right-turning movement at the terminal on the minor intersecting road. Left-turning ramps separate into distinctive configurations geometrically and operationally. The loop always assumes a right exit-right entrance situation. The semidirect connection or jughandle can be arranged to produce three different operational configurations: right exit-right entrance, right exit-left entrance, and left exit-right entrance. The direct connection always assumes a left exit-left entrance pattern.

A two-way ramp is illustrated in Figure 6-J.

6.04.02 CAPACITY

The capacity of a ramp may be limited by any one of its component parts: ramp proper (central or uniform width section); entrance terminal (ramp to highway); and exit terminal (highway to ramp). Also, the capacity may be governed by sections where ramp traffic is required to weave with other traffic.

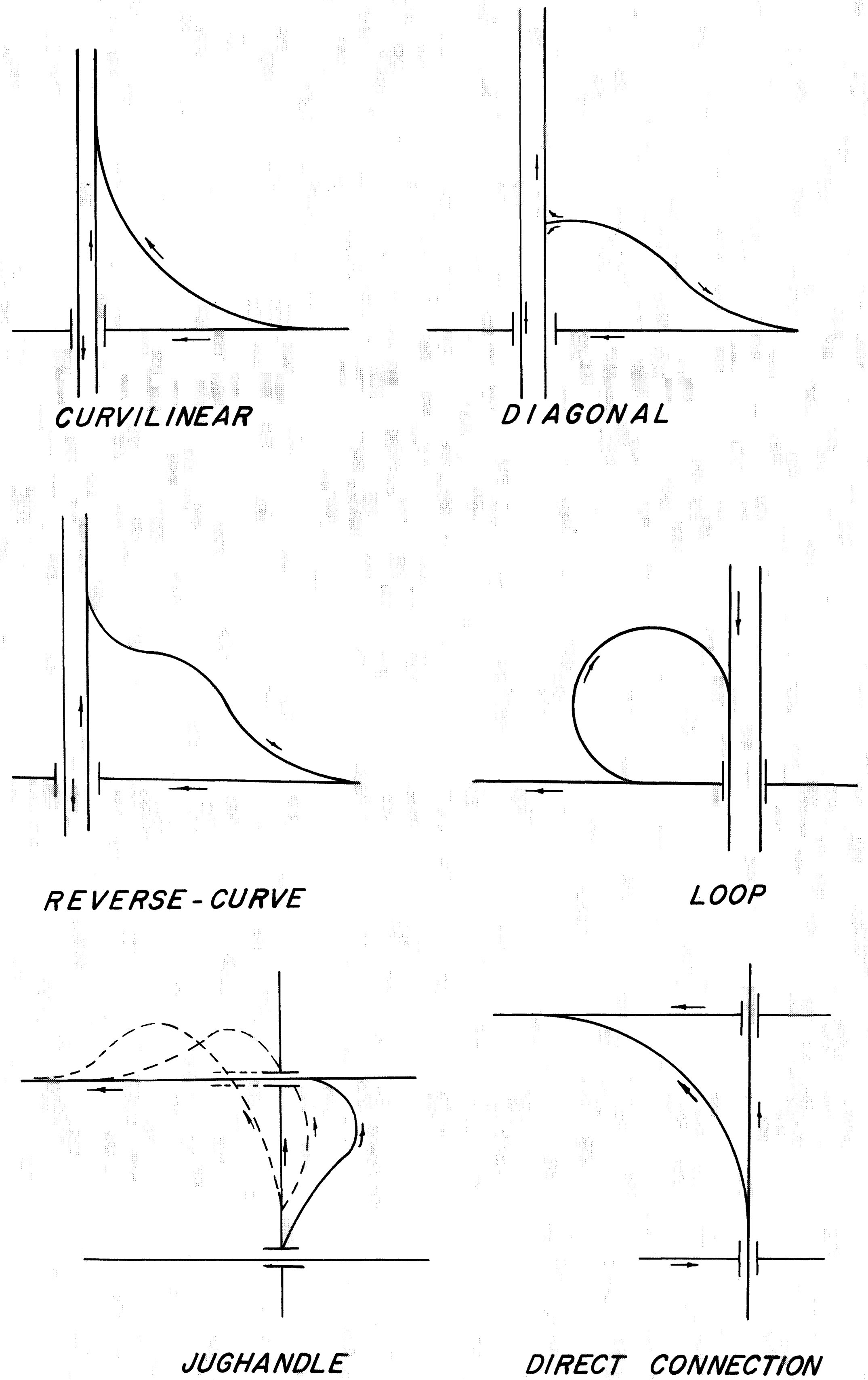
A. RAMP PROPER

The design capacity of a ramp proper is affected by curvature, gradient, and the percentage of trucks. Design capacities with single-lane operation on ramps proper as affected by these variables are indicated in Table 6-1.

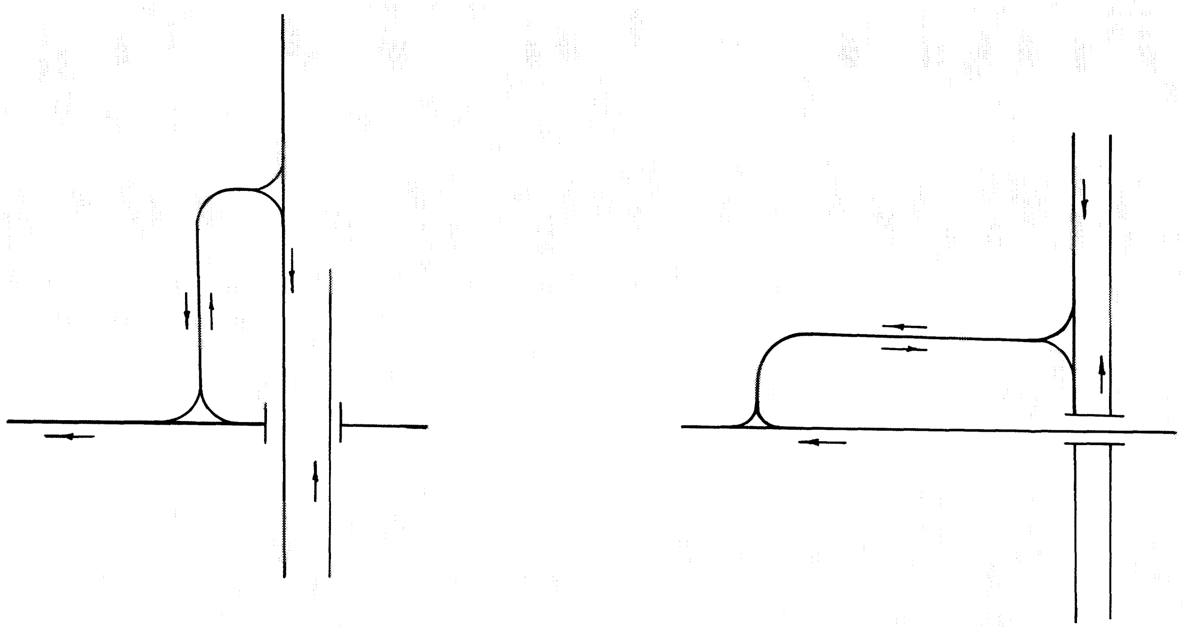
B. ENTRANCE TERMINAL

At ramp entrance terminals, capacity is governed by the design of the terminal and the type of traffic control utilized.

For ramps entering free flowing roadways, as on freeways, about 1500 passenger cars per hour can merge in one lane without reducing the speed of traffic on the highway to less than about 35 mph, provided there is a long taper or acceleration lane and that sight distance is good. A value of 1200 passenger cars



NEW YORK STATE DEPT. OF TRANSPORTATION	RAMP TYPES	FIG. 6-I



TWO - WAY RAMPS

NEW YORK STATE DEPT. OF TRANSPORTATION	RAMP TYPES	FIG. 6-J

Table 6-1

DESIGN CAPACITY OF RAMP PROPER

SINGLE-LANE OPERATION

Upgrade in percent	Ramp Design Capacity in vph when V = design speed and T = percentage of trucks during peak hour							
	V = 20 mph or less				V = 30 mph or more			
	T = 0	T = 10	T = 20	T = 30	T = 0	T = 10	T = 20	T = 30
0-2 (or downgrade)	1000	900	830	770	1500	1350	1250	1150
3-4	1000	830	710	620	1500	1250	1100	950
5 and over	1000	770	620	530	1500	1150	950	800

Notes:
(1) For loops having design speeds lower than 40 mph, design capacity of ramp proper should be reduced by 20 percent.
(2) Possible capacity approximately 1.25 times above values.
(3) For 2-lane operation, increase the tabular values up to nearly double.

merging may be taken as a basis for determining the design capacity of a ramp entrance (with acceleration lane) in rural areas. In urban areas where speeds are lower, the corresponding value is 1500 passenger cars per hour. These values include both the vehicles entering from the ramp and those on the outer lane of the through roadway. To determine the number of entering vehicles per hour that can merge with traffic on the highway, it is necessary to deduct from this hourly capacity value the through vehicles per hour likely to occupy the lane adjacent to the ramp, and to adjust further for the proportions of trucks involved.

The volume of through traffic occupying the lane adjacent to the ramp depends upon a number of variable roadway and traffic conditions; such as the proximity of nearby on and off ramps and the volume of traffic using these ramps, the total traffic volume on the freeway and the number of freeway lanes. The Highway Capacity Manual contains equations and nomograph charts for estimating the volume of through traffic using the lane adjacent to the ramp. Having made this determination the procedure for evaluating the capacity of a ramp entrance for single lane and 2-lane operation on the ramp is given in Table 6-2. For many situations it is not unusual, in actual practice, to dispense with capacity calculations as given in this table and determine capacity by the capacity of the main roadway beyond the ramp terminal on the theory that traffic in the several lanes will adjust itself to such volumes and their distribution among the lanes. This is shown in the note under Table 6-2. These apply to ramps joining through traffic on the left as well as on the right.

Computer Program 85004, Ramp Capacity, is also available to determine capacity. At some interchanges, such as a diamond or partial cloverleaf, the entrance terminal is designed as a T intersection at grade. The capacity of such ramp entrances are evaluated as for any intersection at grade. Usually these ramps are designed for single-lane operation at the exit from a highway and on the ramp proper (one-way), but at the entrance to a highway the ramp often is widened to two or three lanes for necessary storage and capacity.

C. EXIT TERMINAL

At ramp exit terminals, the number of vehicles that can leave the highway is affected by the volume of through traffic using the lane adjacent to the ramp. The practical or design capacity on rural roads of a single-lane exit, with adequate deceleration lane is about 1300 passenger cars per hour, less the number of through vehicles occupying the lane adjacent to the ramp during the same hour. In urban areas, where speeds are lower and acceptable densities are high, the corresponding value is 1600 passenger cars per hour. The number of through vehicles occupying the lane adjacent to the ramp varies with the distances to adjacent on and off ramps and the volumes of traffic using these ramps; with the total traffic volume using the highway in relation to the number

Table 6-2

Capacity of Ramp Entrance to Highway (Merging)

A. SINGLE-LANE ENTRANCE - with acceleration lane Rural: $C = \dfrac{1200 - V_1 (1 + t_1)}{1 + t}$ Urban: $C = \dfrac{1500 - V_1 (1 + t_1)}{1 + t}$ Where C = design capacity, vph from the ramp V_1 = volume of through traffic in lane adjacent to ramp, vph t_1 = trucks as a percentage of total through volume, $\div$ 100 t = trucks as a percentage of entering volume, $\div$ 100 See Highway Capacity Manual for procedure to estimate V_1 for various roadway and traffic conditions.
B. SINGLE-LANE ENTRANCE - with right lane from ramp continued onto highway beyond ramp as an added lane on highway. Capacity is that of single lane on ramp proper.
C. TWO-LANE ENTRANCE - major fork design with right lane from ramp continued onto highway beyond ramp as an added lane on highway, and ramp of adequate width to discharge two lanes of vehicles. C = sum of capacities of the two lanes - Right lane, use single-lane capacity on ramp proper. Left-lane use expression A above.

NOTE: Capacity of ramp entrance may also be governed by the capacity of the highway beyond the ramp; i.e., the combined volume of entering and through traffic should not exceed the capacity of the highway section ahead. This type of check should always be made.

of lanes; the number of drivers desiring to use the ramps under consideration; and other variables. Equations and charts for estimating the total volume of traffic (exiting traffic plus through traffic) in the lane adjacent to the ramp and immediately upstream from the terminal are provided in the Highway Capacity Manual. This volume should not exceed 1,600 equivalent passenger cars for urban freeways and 1,300 for rural freeways. The difference between these values and the volume of through traffic in the lane adjacent to the ramp is the volume of traffic exiting from the freeway, see Section A in Table 6-3. Where the number of through lanes is reduced beyond the exit to the ramp, it is reasonable to assume that a single-lane ramp will have a capacity equal to a single lane on a ramp proper, Table 6-1, even though a few vehicles are likely to use the outer lane, see Section B in Table 6-3.

Where the number of through lanes is reduced beyond a 2-lane exit ramp, it is also reasonable to assume that the right lane of the ramp will have a capacity equal to a single lane on a ramp proper, Table 6-1, but the capacity of the second lane is subject to the restrictions of the through lane second from the right edge being occupied with through vehicles. Its capacity, therefore, is deemed to be about the same as that of a single-lane ramp leaving a through highway which is not reduced in number of lanes. This may be calculated as the value "C" in Section A of Table 6-3. Two-lane exit ramps are not recommended unless the number of through lanes is reduced beyond the exit terminal. A reduction in number of through lanes beyond the terminal may be achieved without basically changing the number of through lanes by adding a comparatively short section of lane of full width in advance of and in addition to the usual speed-change lane or taper.

6.04.03 DESIGN SPEED

Guide values for ramp design speed in terms of highway design speed are shown in Table 6-4. Ramp designs should be based on the desirable design speed where feasible. When design speeds of intersecting highways are different, the ramp design speed preferably should be related to the intersection leg of higher design speed. As an alternative, each terminal may be designed in relation to the intersection leg with which it connects, and the intervening ramp section designed for a suitable intermediate speed.

6.04.04 SIGHT DISTANCE

The minimum stopping sight distance values for ramps are shown in Table 6-5. Longer sight distances should be considered wherever feasible.

Table 6-3

Capacity of a Ramp Exit from Highway (Diverging)

A. SINGLE-LANE EXIT - with deceleration lane

Rural: $$C = \frac{1300 - V_1 (1 + t_1)}{1 + t}$$

Urban: $$C = \frac{1600 - V_1 (1 + t_1)}{1 + t}$$

Where C = design capacity, vph to the ramp
V_1 = Volume of through traffic in lane adjacent to ramp, vph
t_1 = trucks as a percentage of through volume, ÷ 100
t = trucks as a percentage of ramp volume, ÷ 100

See Highway Capacity Manual for procedure to estimate V_1 for various roadway and traffic conditions.

B. SINGLE-LANE EXIT - with outer lane of highway continued onto ramp and number of lanes on highway reduced beyond ramp. Capacity is that of single lane on ramp proper.

C. TWO-LANE EXIT - major fork design with outer lane of highway continued onto ramp and the ramp of adequate width to receive two lanes of vehicles. Capacity of right lane as in B above. Capacity of left lane as in A above.

Table 6-4

Guide Values for Ramp Design Speed as Related to Highway Design Speed

Highway design speed, mph	30	40	50	60	65	70	75	80
Ramp design speed, mph								
Desirable	25	35	45	50	55	60	60	65
Minimum	15	20	25	30	30	30	35	40
Corresponding minimum radius, ft.								
Desirable	150	300	550	690	840	1040	1040	1260
Minimum	50	90	150	230	230	230	300	430

Table 6-5

Minimum Stopping Sight Distance for Ramps

Design speed, mph	15	20	25	30	35	40	45	50	55	60	65
Minimum stopping sight distance, ft.*	80	120	160	200	240	275	325	350	425	475	550

*Measured from driver's eye 3.75 feet above road surface to top of object 6 inches high on road surface.

6.04.05 GRADES

The profile of a typical ramp usually consists of a central portion of an appreciable grade, coupled with terminal vertical curves and connections to the profiles of the intersection legs. The following references to ramp gradient pertain largely to the central portion of the ramp profile. Profiles at the terminals are determined mostly by the through road profiles and seldom are tangent grades. At least one of the intersecting highways at an interchange usually has high design standards, and to be consistent, ramp gradients should be limited to 3 to 5 percent. It may be necessary to use ramp gradients of up to 8 percent in some instances but these should be considered as special cases, warranted only by limited site conditions or by a light turning movement. In general, adequate sight distance is more important than a specific gradient control and should be favored in design. Usually these two controls are compatible.

On one-way ramps a distinction can and should be made between up and down gradients. For high speed ramp designs the values cited above apply. However, with proper ramp terminal facilities, short upgrades of 7 to 8 percent permit safe operation without unduly slowing down passenger cars. Short upgrades of as much as 5 percent do not unduly interfere with truck and bus operation. On one-way down ramps, gradients up to 8 percent do not cause hazard due to excessive acceleration.

Ramp gradients are not directly related to design speed; however, design speed is a general indication of the standards being used, and gradients for a ramp with a high design speed should be flatter than for one with a low design speed. As general criteria, it is desirable that gradients on up ramps with a design speed of 35 mph and greater be limited to 3 to 5 percent; 25-30 mph speed to 4 to 6 percent; and a 15-20 mph speed to 6 to 8 percent. One-way gradients on down ramps should be held to the same general maximums, but in special cases may be 2 percent greater.

6.04.06 CROSS SECTION

A. PAVEMENT

The widths of pavement for ramps shall be as shown in Chapter 3.00, Typical Sections. The normal pavement slope for a one-way, one-lane ramp shall be $\frac{1}{4}$ inch per foot, with the left edge of pavement (in the direction of travel) as the high side. The maximum allowable superelevation rate shall be 0.06 feet per foot. Superelevation rates for various design speeds and degrees of curve are shown in Chapter 3.00, Typical Sections. The ramp pavement material shall be asphalt concrete. Due to the varying width required for ramps, the use of cement concrete has proven to be considerably more expensive than for the mainline. Where the mainline is cement concrete, the ramp should also be cement concrete up to the next joint beyond the gore (i.e., the intersection of the right edge of the mainline pavement and the left edge of the ramp pavement). This eliminates the undesirable effect created by a longitudinal joint between disimilar pavement types. There will be some situations (e.g., at direct-type interchanges between freeways) where this policy may not be applicable because of other more significant factors.

B. SHOULDER

The shoulder width and slope for ramps shall be as shown in Chapter 3.00, Typical Sections.

On the inside of ramp circular curves, the stabilized shoulder from a point 20' before the P.C. to a point 10' past the P.T. shall consist of asphalt concrete of equivalent strength to that of the adjacent ramp pavement.

C. CURB

Curbs at the edge of travel lanes will be used only for special conditions such as matching existing conditions or on some parkways.

Curbs may be used at the outer edge of shoulder where required for drainage or other purposes.

D. GENERAL

For further details on ramp sections, see Chapter 3.00, Typical Sections.

6.04.07 RAMP TERMINAL DESIGN

The terminal of a ramp is that portion adjacent to the through travelled way, including speed-change lanes, tapers, noses, merging ends, and islands. Ramp terminals may be of the at-grade intersection type, as at diamond or partial cloverleaf interchanges, or of the directional type where ramp traffic merges with or diverges from through traffic at flat angles.

A. AT-GRADE INTERSECTION TYPE

Where ramps intersect the minor crossroad at grade, the ramp terminals are normal intersections and should be designed as such. The length of highway open to view from the ramp terminal must be greater than the product of the speed of a vehicle on the crossroad and the time necessary for the vehicle entering the crossroad from a stopped position on the ramp to start and complete a left turn into the crossroad.

The distances travelled by vehicles entering the crossroad by turning left from the ramp before clearing the lane used by a vehicle approaching from the left are about 60 feet for passenger cars, 90 feet for SU vehicles, and 120 feet for WB-50 vehicles. These distances are based on the assumption that the turning vehicle accelerates from a stopped position on the ramp with the front of the stopped vehicle 10 feet from the edge of through pavement, the turning vehicles follow the minimum turning paths for the respective design vehicles, and the vehicles enter a 2-lane, two-way highway.

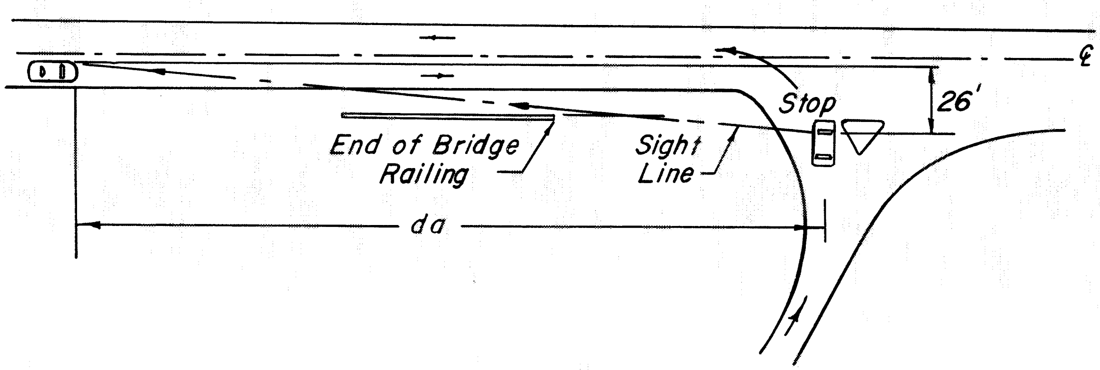
The sight distance requirements for various design speeds and for the three classes of vehicles are shown in Table 6-6. The assumed design speed on the crossroad through the interchange is shown in this table. This should be a speed somewhat faster than that of vehicles expected on the section of highway in the interchange area and is greater than the average running speed. Both the horizontal sight triangle and the vertical curvature as shown in Figure 6-K should be checked to insure that the sight distance shown in Table 6-6 is provided. The check is best done graphically. Figure 6-K shows the check made at a typical diamond interchange in a rural area to insure that there is sufficient sight distance past the bridge parapet. A similar check should be made where the crossroad crosses underneath the freeway but in this case the abutment or columns instead of the end of a parapet would have to be cleared. In some cases, vertical curves longer than those necessary to provide safe stopping distance for the crossroad may have to be provided. In other cases, the ramp terminals may have to be relocated farther from the structure.

An inherent problem of diamond interchanges is the possibility of a driver entering one of the exit terminals from the crossroad and proceeding along the freeway in the wrong direction in spite of signing. Attention to several details of design at the intersection can discourage this hazardous maneuver. As shown in Figures 6-L and 6-M, a "sharp" or angular intersection is provided at the junction of the left edge of the ramp entering the crossroad and the right edge of the through pavement. The control radius should be tangent to the crossroad centerline, not edge. This type of design does not invite drivers to make the improper right turn into the one-way ramp.

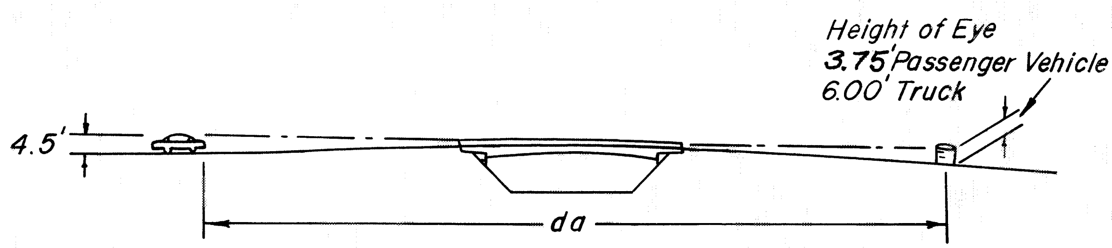
As shown in Figures 6-L and 6-M(b), islands can be used in the terminal area where ramps intersect the crossroads. The islands provide a means of channelizing the traffic into proper paths and can be effectively used for sign placement.

Where economical, provision of a median as a deterrent to wrong-way movement as illustrated in Figure 6-M(b) is generally a very effective treatment. The median makes the left turn movement onto the exit ramp terminal very difficult and a very short radius curve or angular break is provided at the intersection of the left edge of the exit ramp and the crossroad to discourage wrong-way right turns from the crossroad.

Revised May 1976

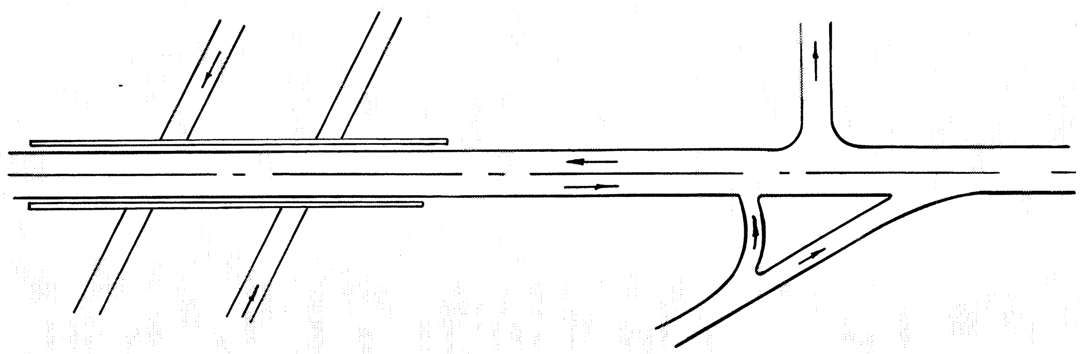


- A -



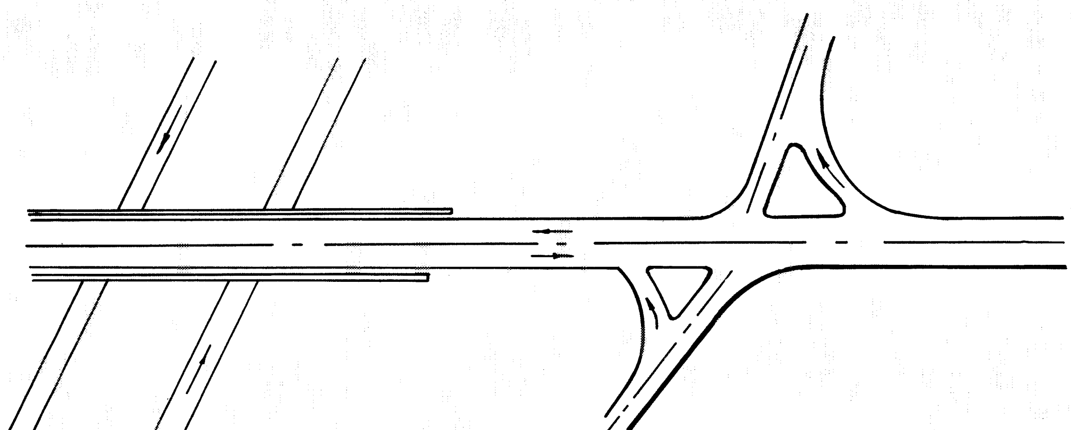
- B -

NEW YORK STATE DEPT. OF TRANSPORTATION	MEASUREMENT OF SIGHT DISTANCE AT DIAMOND RAMP TERMINALS	FIG. 6-K



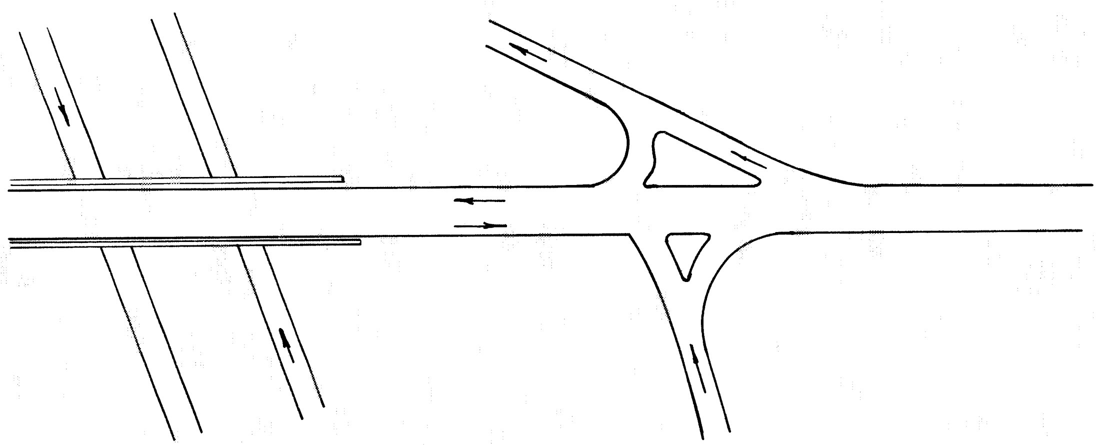
30 DEGREE ANGLE OF INTERSECTION

- A -



50 DEGREE ANGLE OF INTERSECTION

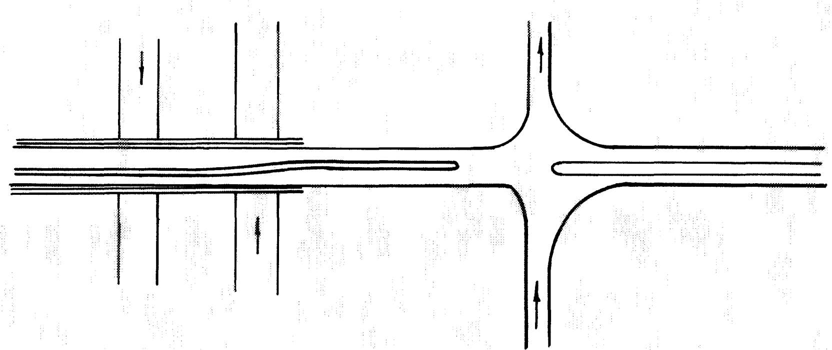
- B -



125 DEGREE ANGLE OF INTERSECTION

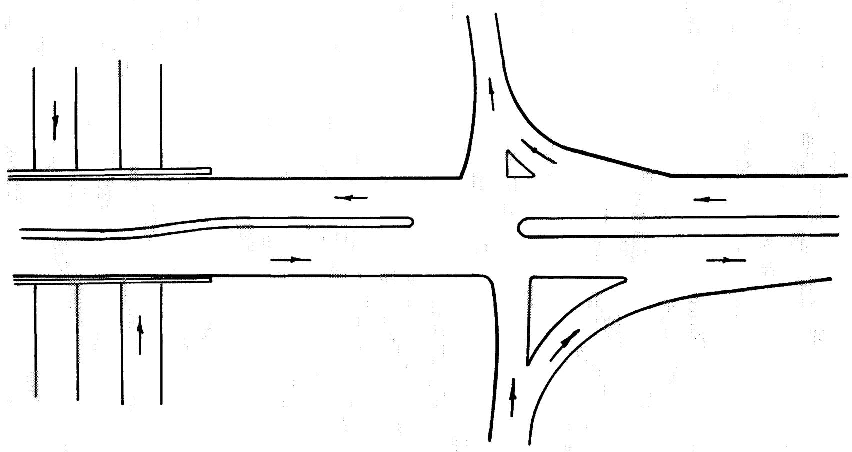
- C -

NEW YORK STATE DEPT. OF TRANSPORTATION	INTERSECTION OF SKEWED RAMP WITH CROSSROAD	FIG. 6 - L



2 LANE CROSSROAD

-a-



4 LANE CROSSROAD

-b-

NEW YORK STATE DEPT. OF TRANSPORTATION	INTERSECTIONS AT DIAMOND RAMP TERMINALS, 90° ANGLES	FIG. 6-M

Table 6-6

Required Sight Distance Along the Crossroad at Terminals of Ramps at Diamond Interchange

Assumed design speed on crossroad through the interchange	Sight distance required to permit design vehicle to turn left from ramp to crossroad, feet*			Sight distance available to entering vehicle when vertical curve on crossroad is designed for minimum stopping sight distance**	
	Design vehicle assumed at ramp terminal.				
	P	SU	WB-50	P	SU or WB-50
70	740	1060	1430	920	1040
60	630	910	1230	730	820
50	530	760	1030	540	600
40	420	610	820	420	480
30	320	460	620	310	350

*Sight distance measured from height of eye 3.75 feet for P design vehicles and 6 feet for SU and WB-50 design vehicles to an object 4.5 feet high.

**Minimum available stopping sight distance based on the assumption that there is no horizontal sight obstruction and that $S<L$.

B. DIRECTIONAL TYPE

1. General

The question of the shape of speed change lanes at ramp terminals, that is, parallel auxiliary lanes versus direct tapers, has generated considerable controversy among Highway Engineers for many years. One result of this controversy has been that both designs have been used on the freeway system. This lack of standardization has caused confusion among drivers to the point where many people have never learned how to properly utilize this added pavement width.

2. Comparison of Alternate Designs

The most important desired characteristic of ramp terminals is that smooth, efficient merging and diverging maneuvers be provided for and encouraged. This requires that the moves be made in a direct and positive manner, that the exiting driver not be forced to begin braking until he is clear of the mainline and that the entering driver enter the through lanes at a speed closely approximating the speed of the mainline traffic flow.

Studies were made of driver behavior on acceleration and deceleration lanes in Oregon, California, Texas and Indiana. These studies generally agree on many factors among which are:

(a) Of the drivers who use the speed change lanes properly, the majority follow a gradually tapered path regardless of the physical design.

(b) Many drivers do not know how to use speed change lanes.

(c) Direct taper designs tend to encourage a larger percentage of drivers to make proper use of speed change lanes.

The main argument in favor of parallel acceleration lanes has been that it provides a somewhat greater distance for the entering driver to find an acceptable gap in the mainline traffic stream. While this argument is theoretically correct, several other factors tend to negate it:

(a) As stated earlier, most drivers tend to follow a gradually tapered path.

(b) Psychologically, the fact that entering vehicles, on a direct taper acceleration lane, are on a constantly converging course, tends to encourage the drivers in the right hand through lane to yield to the entering traffic.

(c) With the standardization of flush paved shoulders, the shoulder area is now available for emergency maneuvering if the gap should fail to appear or should unexpectedly be closed.

Parallel deceleration lanes are said to provide better advance delineation of the exit and to prevent accidental use of the ramp by drivers who want to stay on the through highway. Current thinking is that the direct taper operates similarly due to its angular nature which is accentuated in the foreshortened view seen by approaching drivers.

Direct taper exit terminals have a distinct advantage in that they encourage drivers to maintain a reasonable speed until they are clear of the through lanes. Additionally, as noted earlier, the direct taper conforms more closely to the desire of most drivers.

In conclusion, direct taper designs shall be used for all exit and entrance ramp terminals on freeways and other major highways in New York State except as noted below.

3. Warrants for Use of Parallel Speed Change Lanes

A parallel deceleration lane may be used when a right hand exit is unavoidably located on a mainline curve to the left and it is feared that the use of the standard direct taper design might result in inadvertent use of the ramp by through traffic. This configuration, that is a right hand exit on a mainline curve to the left, is considered extremely undesirable and should be avoided if at all possible. However, it is recognized that under some conditions in urban areas it might be unavoidable and in that case a special ramp terminal design is required possibly including a parallel deceleration lane. The designer is cautioned to take into account roll-over rate at the joint between the mainline and the ramp. See Table VII-14 in 1965 AASHO Policy on Geometric Design of Rural Highways.

A parallel deceleration lane may also be required where an exit is unavoidably located immediately beyond a crest vertical curve.

A parallel acceleration lane may be required at an entrance carrying a large volume of heavy trucks in combination with a steep upgrade. In that case, it should be considered more as a truck climbing lane than an acceleration lane and should be designed accordingly. (See Chapter 5.00, Basic Design). One further use of parallel lanes occurs when it is desired to maintain the basic number of through freeway lanes at a two-lane entrance or exit. In order to maintain lane balance in such a case, an auxiliary lane is added to the freeway a minimum of 500 feet upstream from the beginning of the exit ramp taper. In the case of an entrance, an auxiliary lane is added downstream from the end of the acceleration lane for 2,000' (1,000' min.). The ramp terminal is then designed as a normal two-lane ramp.

4. Details of Ramp Terminals

For details of single lane ramp terminals, see Figures 6-N and 6-O. For two-lane ramps, simply add a 12-foot width of pavement on the right of the ramp and mainline pavements shown.

5. C-D Roads

Criteria for ramp terminals on continuous C-D roads are identical to those for freeway mainlines. On other C-D roads, weaving and merging criteria will generally govern.

6. Frontage Roads

The frontage road end of a freeway slip ramp exit (see Figure 6-C) is designed as an entrance terminal similar to the freeway entrance terminal, except that the taper can be sharpened to approximately 30 to 1. Similarly, at the frontage road end of a freeway slip ramp entrance, the freeway exit terminal design may be used with a 1.5° curve substituted for the 1° curve used on the freeway.

6.04.08 DISTANCE BETWEEN SUCCESSIVE RAMP TERMINALS

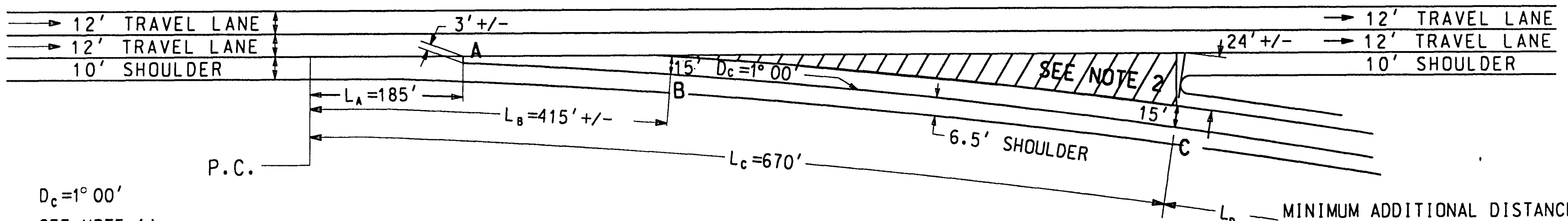
At interchanges, there are frequently two or more ramp terminals in close proximity along the through lanes. In some interchange designs, ramps split into two separate ramps or combine into one ramp. Minimum and desirable distances between successive ramp terminals are shown in Figure 6-P. In most cases, the distances required to provide full length speed change lanes would govern. The distances shown in the table are based on a decision and maneuver time of 5 to 10 seconds. In most cases for rural situations, at least the desirable values shown in the table in Figure 6-P should be used, and preferably greater distances should be provided in order to allow adequate room for giving drivers directional information with signs. In particular, certain minimum distances suggested for satisfactory signing are ½ mile between exits on a freeway and 600 feet between a freeway exit and an exit on a collector-distributor road.

6.04.09 CONTROL OF ACCESS

There should be control of access along the whole of all the ramps. To insure safety and free flow of traffic, ramps must be kept free of any intermediate roadway connections either by acquisition or outer frontage roads.

At the crossroad end of interchange ramps, control of access is required for several hundred feet along the crossroad. Figures 6-Q through 6-T show the minimum controls for cloverleaf, diamond and partial cloverleaf interchanges, as well as two-way ramps and isolated ramps. In cities and highly urbanized areas, the 300' minimum distance may be reduced to 100' if economic considerations dictate.

Figures 6-U and 6-V illustrate the minimum requirements for control of access at slip ramp terminals with one-way service roads. Two-way service roads should be treated as crossroads.



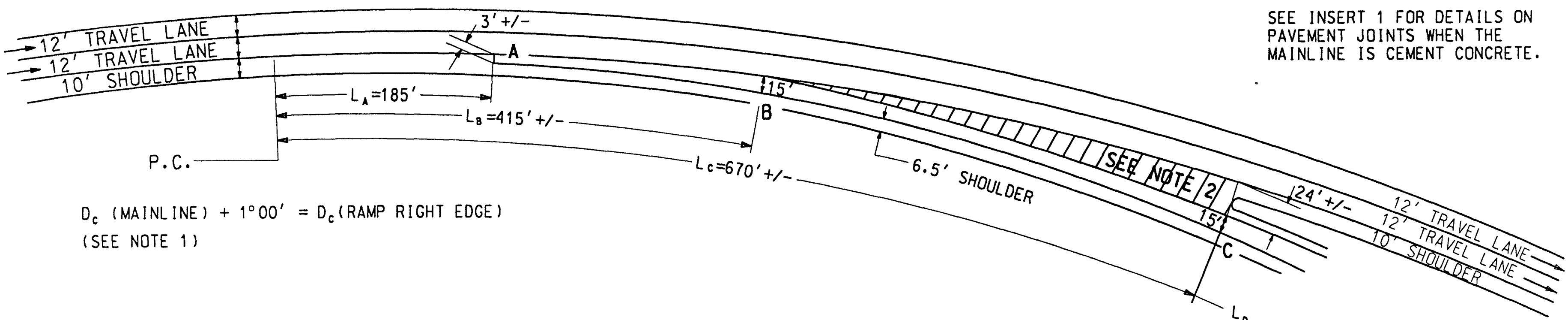
D_c=1° 00'
SEE NOTE 1)

MINIMUM ADDITIONAL DISTANCE REQUIRED FOR DECELERATION BEFORE INTRODUCING SHARPER RAMP CURVE OR STOP CONDITION.

(SEE TABLE 1)

MAINLINE - TANGENT

SEE INSERT 1 FOR DETAILS ON PAVEMENT JOINTS WHEN THE MAINLINE IS CEMENT CONCRETE.



D_c (MAINLINE) + 1°00' = D_c(RAMP RIGHT EDGE)
(SEE NOTE 1)

MINIMUM ADDITIONAL DISTANCE REQUIRED FOR DECELERATION BEFORE INTRODUCING SHARPER RAMP CURVE OR STOP CONDITION.

(SEE TABLE 1)

MAINLINE - CURVE TO RIGHT

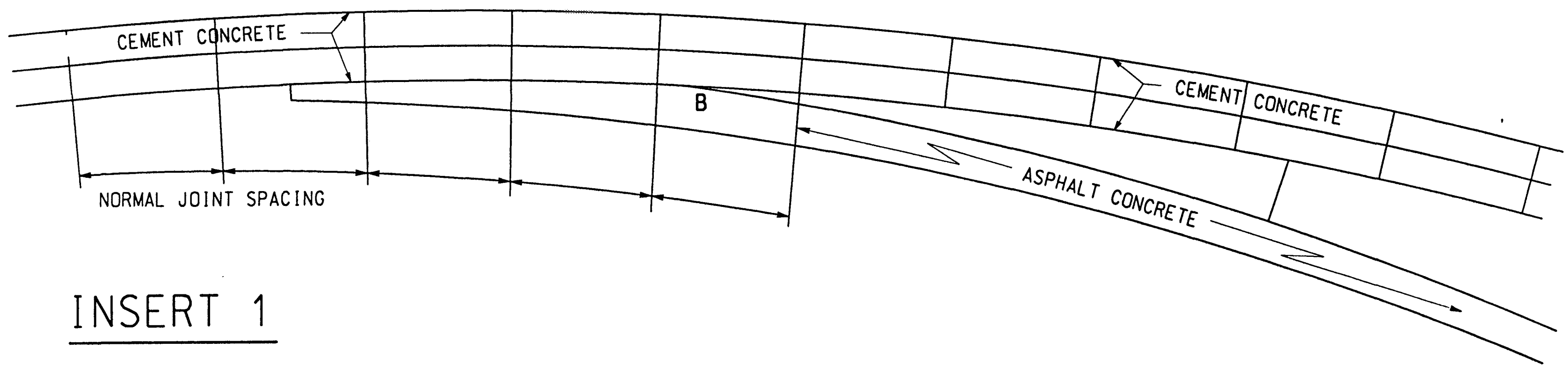
TABLE I

MINIMUM L_D LENGTHS FOR VARIOUS CONDITIONS

RAMP DATA (4)	DESIGN SPEED (mph)	STOP CONDITION	15	20	25	30	35	40	45	50
	MIN. CURVE RADIUS (ft)		50	90	150	230	310	430	550	690
GRADE (5)	DESIGN SPEED OF HIGHWAY (mph)	L_D (ft.) (6)								
+4% to +6%	60	0	0	0	0	0	0	0	0	0
	70	0	0	0	0	0	0	0	0	0
+2% to +4%	60	0	0	0	0	0	0	0	0	0
	70	55	35	10	10	0	0	0	0	0
-2% to +2%	60	15	15	0	0	0	0	0	0	0
	70	115	90	65	65	40	15	0	0	0
-2% to -4%	60	115	115	85	55	25	0	0	0	0
	70	235	205	175	175	145	115	25	0	0
-4% to -6%	60	190	190	155	125	90	55	0	0	0
	70	325	295	260	260	225	190	90	55	0

DESIGN OF
EXIT TERMINALS
NO SCALE

FIGURE 6-N.



INSERT 1

PAVEMENT JOINT LAYOUT

NO STANDARD DESIGN IS BEING INCLUDED FOR EXIT TERMINALS ON MAINLINE CURVES TO THE LEFT. THIS SITUATION SHOULD BE AVOIDED WHERE POSSIBLE. WHERE IT CANNOT BE AVOIDED, THE TERMINAL WILL BE DESIGNED AS A SPECIAL CASE AND SUBMITTED FOR APPROVAL AS PART OF THE NORMAL PLAN SUBMISSION.

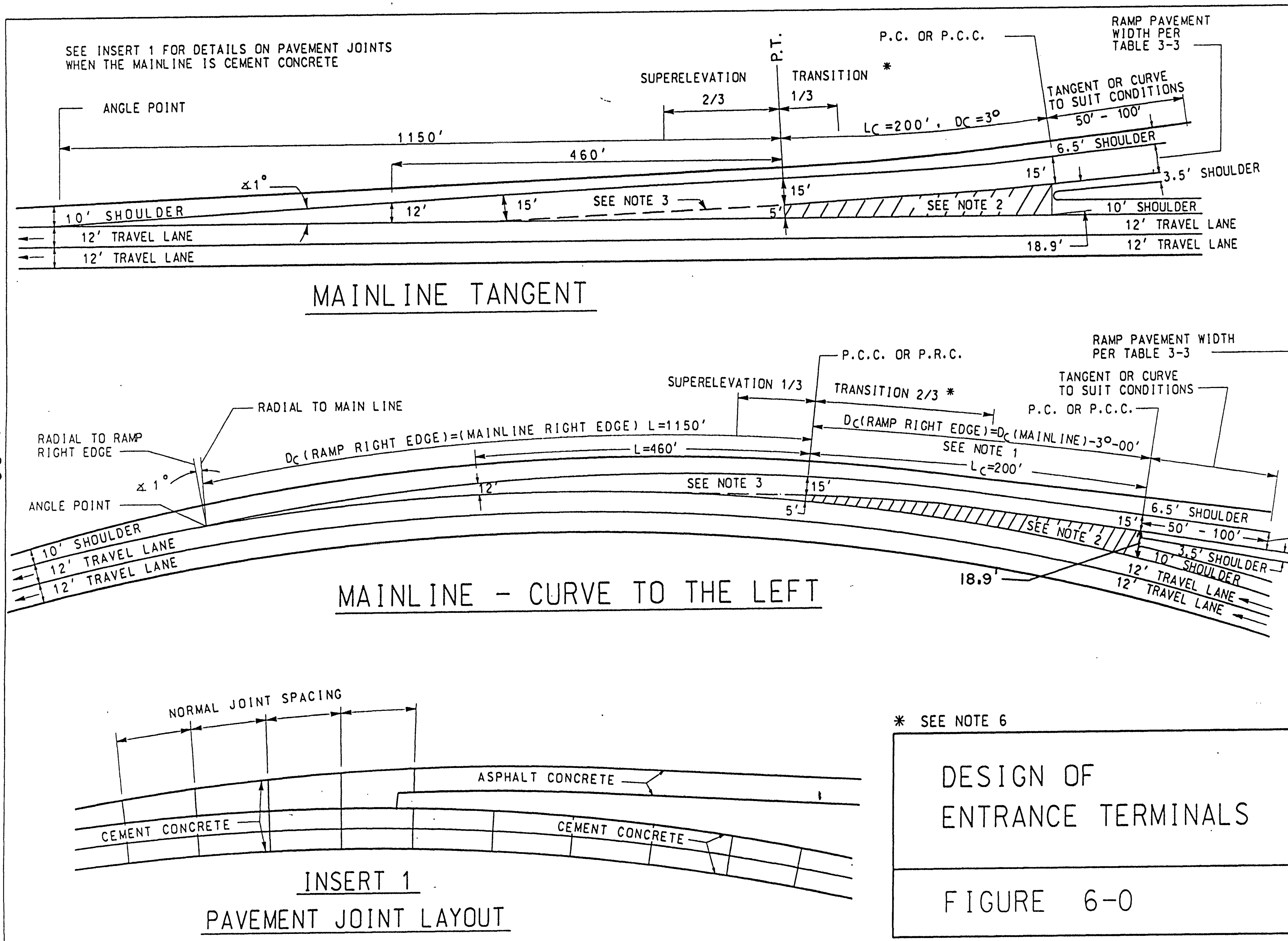
NOTES

1. For all cases, the right edge of ramp is established tangent to the mainline. The ramp edge will be one degree (1°) divergent from the mainline curvature (e.g. Mainline - 4° to the right, ramp - 5° to the right).
2. This area to be paved with full depth asphalt concrete. The surface should be appropriately striped. Care must be taken to see that this area is graded to drain. Some situations may require the use of an internal drainage system.
3. This design applies to all exit terminals except those at the end of combination speed-change lanes.
4. For Interstate Design Standards use a minimum ramp radius of 230 ft.
5. The average grade between points A and C will approximate the adjacent mainline gradient for most cases.
6. The layout of L_D should form a smooth, gentle transition from the exit terminal alignment (up to point C) to the normally, sharper ramp curvature. L_D alignment may be a tangent or a curve with a radius greater than 700 ft.

DESIGN OF
EXIT TERMINALS

NO SCALE

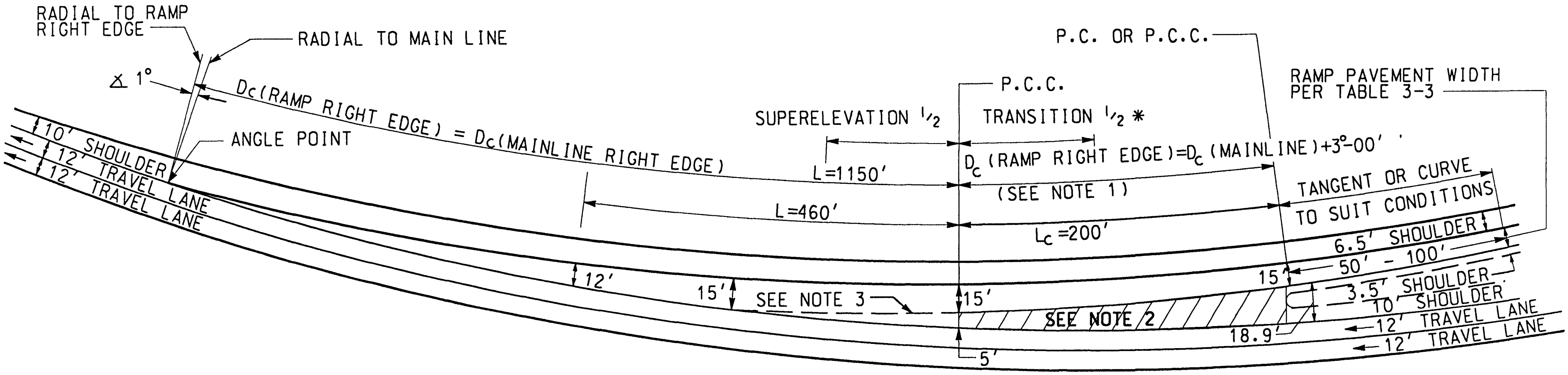
FIGURE 6-N



* SEE NOTE 6

DESIGN OF
ENTRANCE TERMINALS

FIGURE 6-0



MAINLINE - CURVE TO THE RIGHT

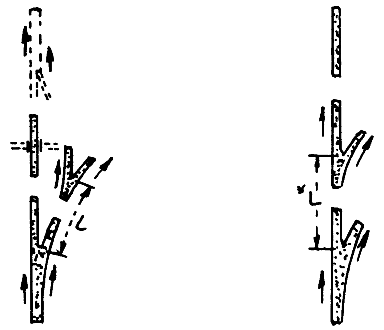
NOTES

1. For all curved cases, the right edge of ramp, for this length, is three degrees (3°) different in curvature than the mainline curvature.
2. This area to be paved with full depth asphalt concrete. The surface should be appropriately striped. Care must be taken to see that this area is graded to drain. Some situations may require the use of an internal drainage system.
3. The dashed line represents pavement striping.
4. This design applies to all entrance terminals <u>except</u> those at the beginning of combination speed - change lanes.
5. For Interstate Design Standards use a minimum ramp radius of 230 ft.
6. Superelevation transition for ramp to mainline shall be accomplished in 200': when the mainline is tangent, 1/3 of the transition shall be accomplished within L_c; when the mainline curves to the left, 2/3 of the transition shall be accomplished within L_c; when the mainline curves to the right, ½ of transition shall be accomplished within L_c.

* SEE NOTE 6

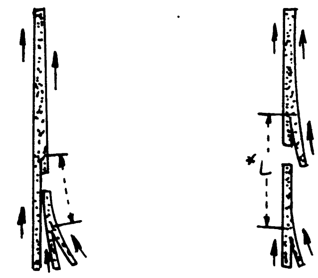
DESIGN OF
ENTRANCE TERMINALS

FIGURE 6-0



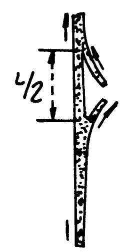
SUCCESSIVE EXIT TERMINALS

-A-



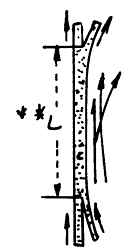
SUCCESSIVE ENTRANCE TERMINALS

-B-



EXIT TERMINAL FOLLOWED BY ENTRANCE TERMINAL

-C-



ENTRANCE TERMINAL FOLLOWED BY EXIT TERMINAL

-D-

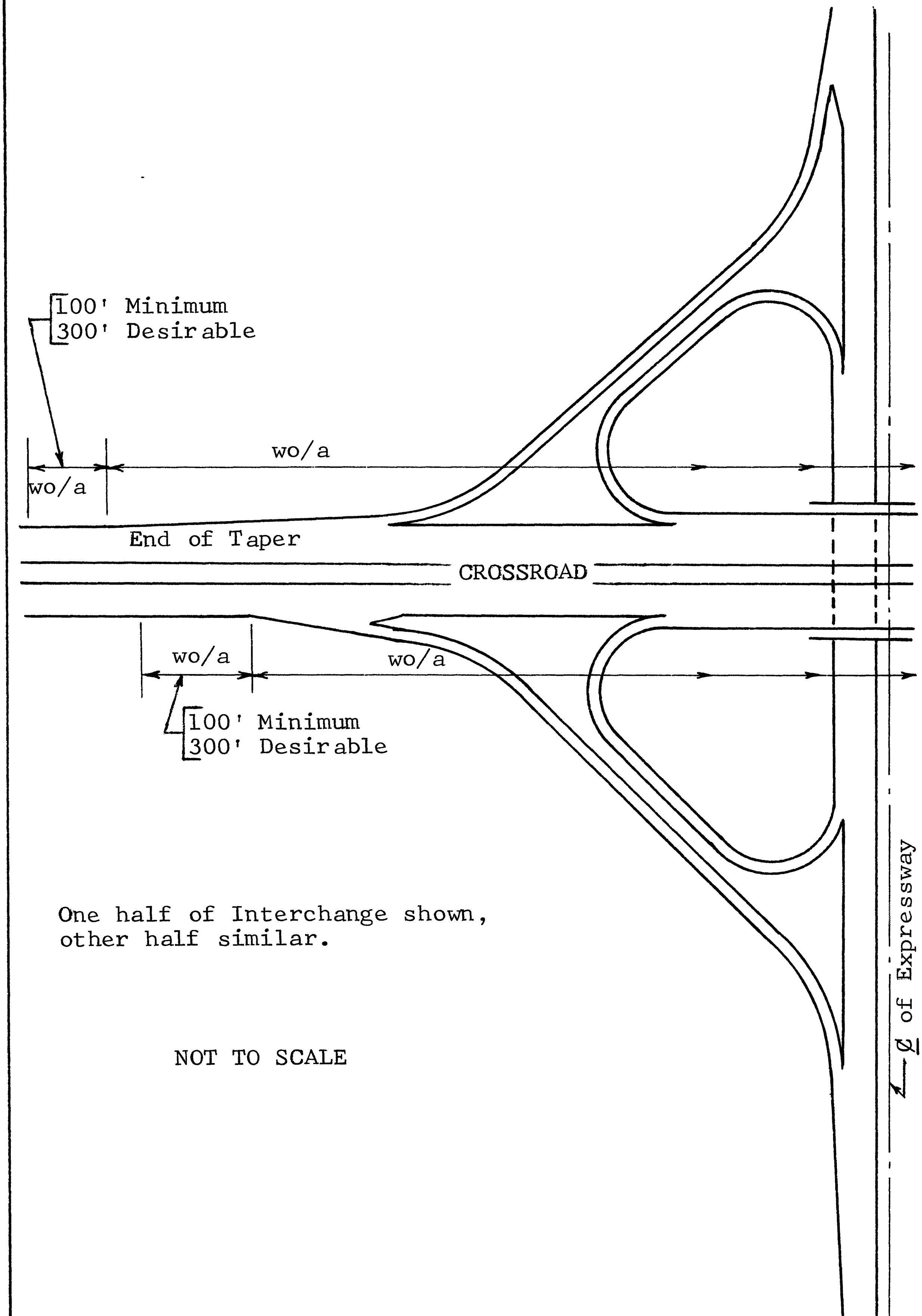
*L as in table but not less than length required for maneuvering or speed change as shown in figures 6-N and 6-O.

**L 700 ft. min. but not less than length required for weaving; see section 6.05.01A.

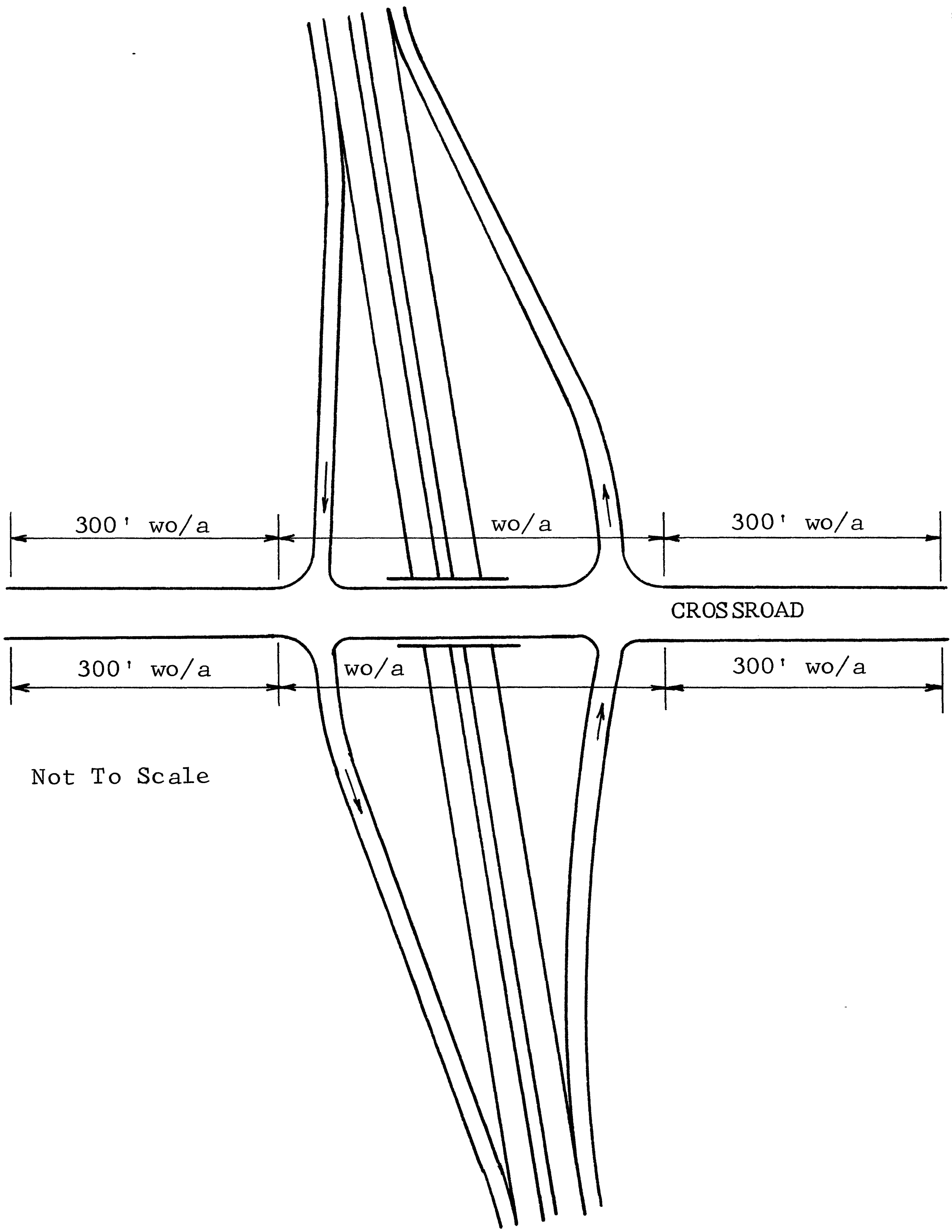
DISTANCE BETWEEN SUCCESSIVE RAMP TERMINALS

Design Speed, mph	30 or less	40 to 50	60 or 70	80
Av. running speed, mph	23 to 38	36 to 44	52 to 58	64
Distance L-FEET				
Minimum	200	400	500	900
Desirable	400	700	900	1200

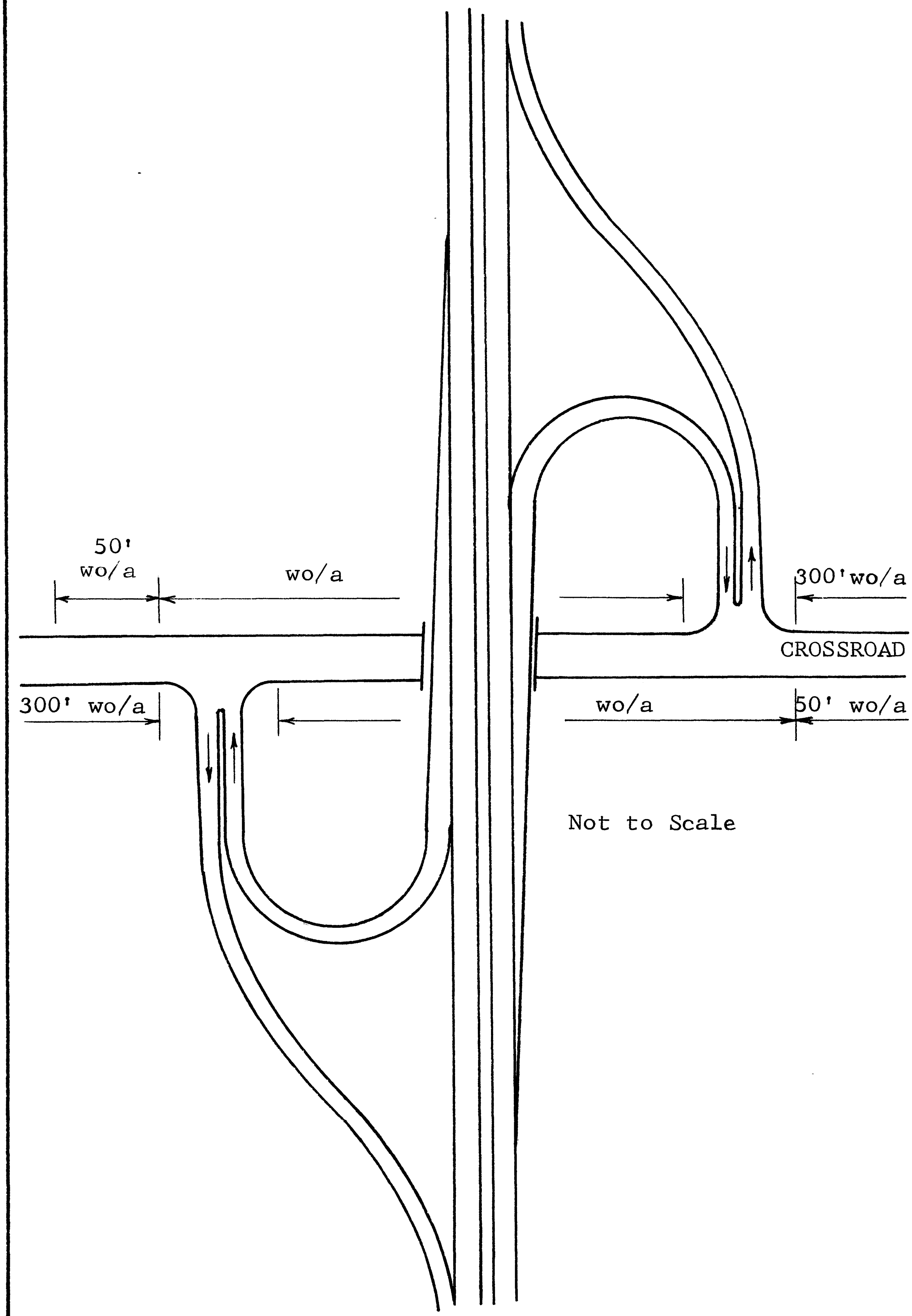
STATE OF NEW YORK DEPT. OF TRANSPORTATION	ARRANGEMENTS FOR SUCCESSIVE RAMP TERMINALS	FIGURE 6-P



NEW YORK STATE DEPT. OF TRANSPORTATION	LIMITS OF FULLY CONTROLLED ACCESS AT EXPRESSWAY RAMPS	FIG. 6-Q

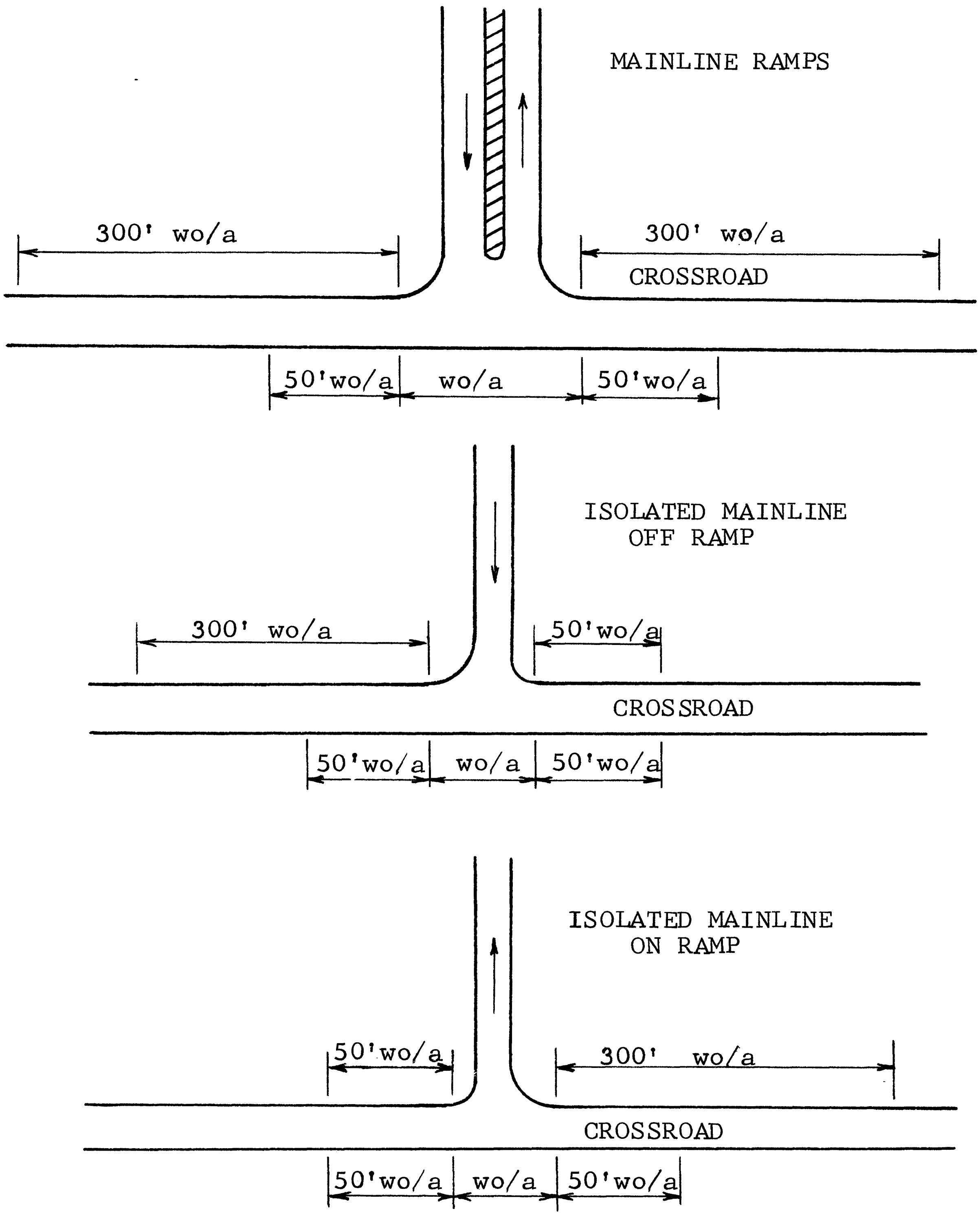


NEW YORK STATE DEPT. OF TRANSPORTATION	LIMITS OF FULLY CONTROLLED ACCESS AT EXPRESSWAY RAMPS	FIG. 6-R



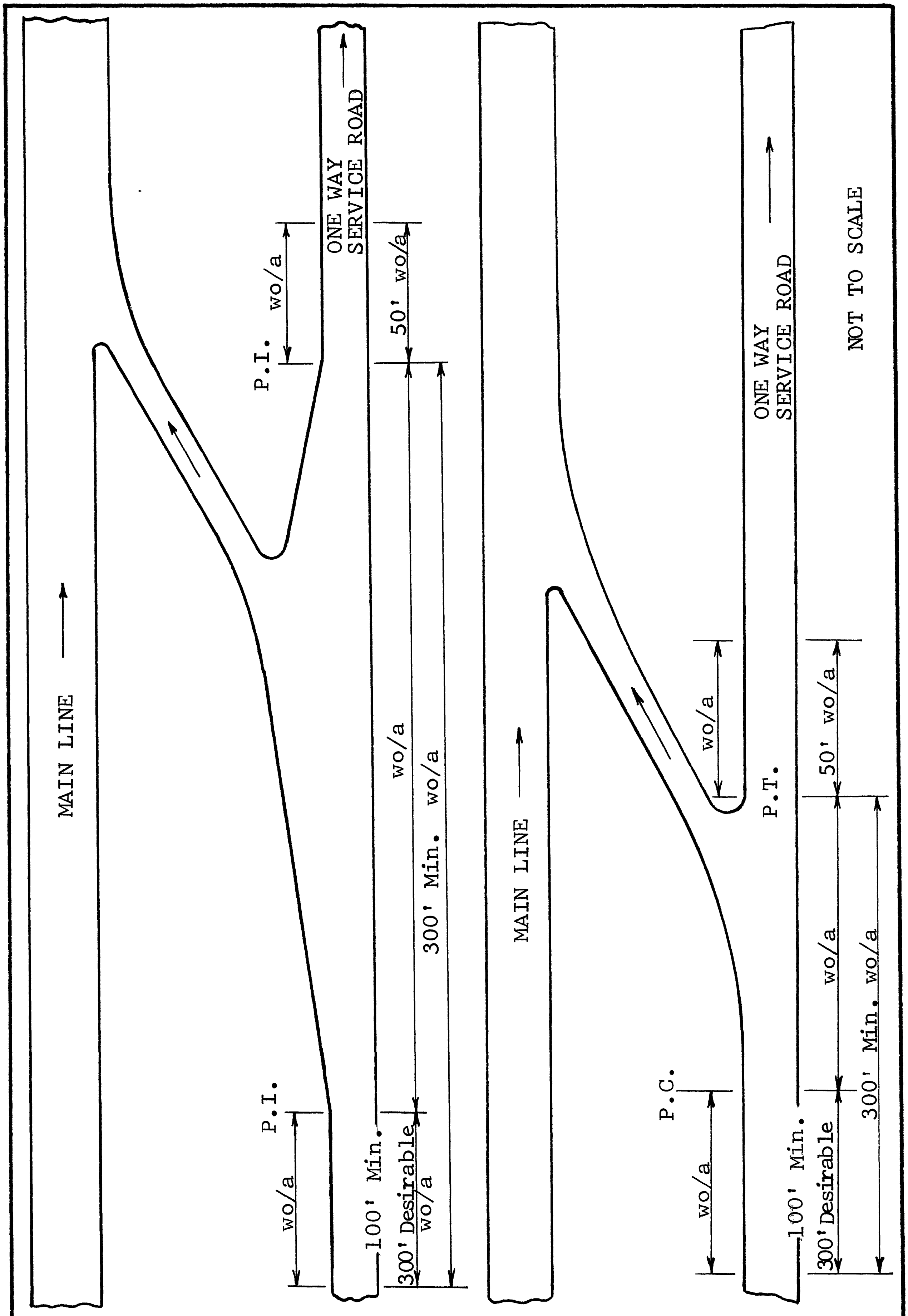
NEW YORK STATE DEPT.OF TRANSPORTATION	Limits of Fully Controlled Access at Expressway Ramps	FIG 6-S

Revised Nov., 1972

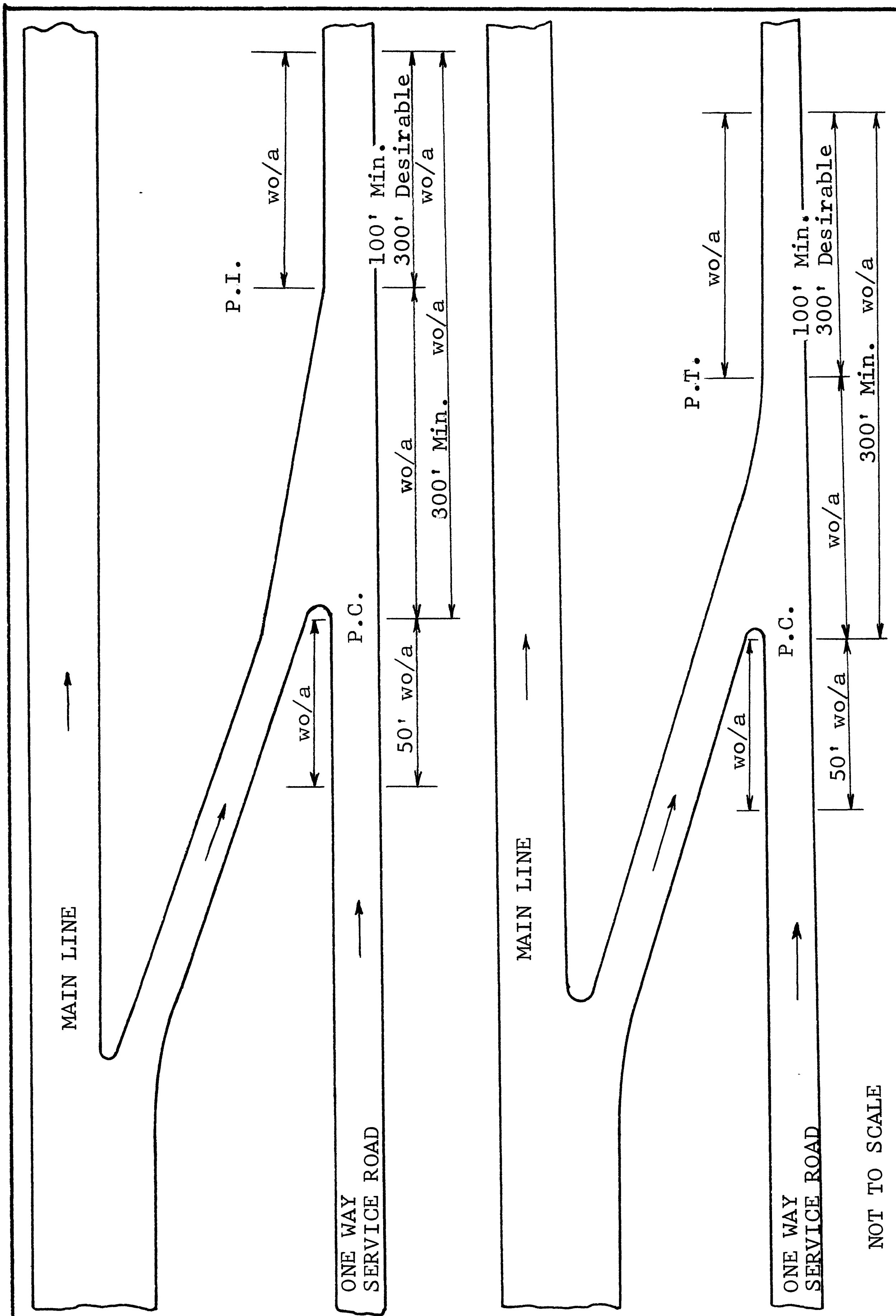


NOT TO SCALE

NEW YORK STATE DEPT.OF TRANSPORTATION	Limits of Fully Controlled Access at Expressway Ramps	FIG 6-T



NEW YORK STATE DEPT. OF TRANSPORTATION	LIMITS OF CONTROLLED ACCESS FOR "ON" RAMPS	FIG. 6-U



NEW YORK STATE DEPT. OF TRANSPORTATION	LIMITS OF CONTROLLED ACCESS FOR "OFF" RAMPS	FIG. 6-V

6.05 PREFERRED DESIGN PRINCIPLES FOR INTERCHANGES

6.05.01 DESIRABLE FEATURES

The basic principles that should be followed in designing an interchange are as follows: (listed in approximate order of importance)

A. MINIMUM WEAVING

B. SINGLE EXITS

C. NO LEFT-HAND EXITS OR ENTRANCES

D. EXITS PRECEDE ENTRANCES

E. SINGLE ENTRANCES

F. DESIRABLE RAMP DESIGN SPEEDS

A. MINIMUM WEAVING

Weaving sections severely reduce speed and capacity and, on high-speed, high-volume highways, cause sharply increased accident rates and congestion. In general, on rural freeways, the distance between any entrance and the following exit should be sufficient to eliminate weaving as a design control. On urban freeways and rural multilane primary highways, some weaving may be tolerated. However, the Quality of Flow as shown in Figure 7.4 on page 166 of the 1965 Highway Capacity Manual should be a minimum of II and for freeways, the absolute minimum distance between any entrance and the following exit shall be 1,800 feet. The section should also be checked against the criteria contained in Chapter 8 of the 1965 Highway Capacity Manual to insure that the requirements for Level of Service C (B for Rural Highways) are met. On Collector-Distributor (C-D) roads and the weaving sections of rotary interchanges, the Quality of Flow should be III (absolute minimum IV in urban areas).

On urban streets, including frontage roads, Level of Service D (Quality of Flow IV) may be acceptable provided that no back-up of traffic onto the freeway lanes results.

As can be seen from the foregoing, direct cloverleaf interchanges with associated short weaving sections are not acceptable on freeways and undesirable on multilane rural primary highways. Cloverleafs with C-D roads may be used on these facilities provided all other criteria are met.

The absolute minimum length of weaving section at locations other than Freeway Mainlines shall be 700 feet.

B. SINGLE EXITS

With two exits the driver must make two decisions:

1. To exit or not.

2. To use first exit or second.

With single exit interchanges, the driver has only one decision to make while on the freeway in the midst of high-speed, high-volume traffic: to exit or not. His second decision is made off the freeway among lower-speed, lower-volume traffic on the ramp or C-D road. It is recognized that incorporation of single exit designs will increase construction costs in many cases due to the need for additional and/or wider structures. This increase in cost can normally be justified by the added safety and the improvement in traffic flow.

C. NO LEFT-HAND EXITS OR ENTRANCES

Left-hand exits and entrances are considered undesirable for the following reasons:

1. Decisions and maneuvering take place in the high-speed lanes.

2. Entering drivers are forced to merge to their right side where, with left-hand drive vehicles, they have reduced visibility and thus more difficulty in making accurate judgments. This problem is greatly magnified when the entering vehicle is a truck.

3. In view of the preponderance of right-hand exits and entrances, left-side moves tend to confuse and surprise drivers even with proper signing.

4. Trucks, which traditionally are restricted to the right-hand freeway lane, are forced to maneuver across several traffic lanes to reach a left-hand exit or to return to the right lane from a left-hand entrance.

The above factors do not, of course, apply to major forks in which the volumes of traffic are so nearly equal as to make it difficult to decide which roadway is main line and which is ramp. In those cases, consideration should be given to placing the higher commercial traffic volumes in the right-hand fork or to using the left fork for the continuous route.

D. EXITS PRECEDE ENTRANCES

The desirability of exits preceding entrances in complementary ramp pairs is quite apparent. Congestion will be reduced by removing traffic before adding new traffic, and main line weaves will be eliminated.

E. SINGLE ENTRANCES

Every entrance creates disturbance and friction in the flow of main line traffic. This results in reduction of main line speed and capacity. It is thus apparent that a reduction in the number of entrances is desirable.

Application of this principle must, however, be tempered by one additional consideration. When traffic volumes on the ramp exceed the capacity of a one-lane entrance, some thought should be given to providing two one-lane entrances, rather than one two-lane entrance. This is particularly desirable when the number of through traffic lanes must be maintained, that is, when it is impossible to provide an additional freeway lane downstream from a two-lane entrance. The minimum separation between two successive one-lane entrances is 1,000 feet.

F. DESIRABLE RAMP DESIGN SPEEDS

Higher ramp design speeds reduce travel time in interchanges, and improve ramp terminal conditions by reducing the magnitude of speed changes. Ramps should be designed for the "desirable" design speed as listed in Table 6-4, or higher.

6.05.02 APPLICATION TO INTERCHANGE TYPES

The principles listed in Section 6.05.01 may be applied to the various interchange types discussed in Section 6.03 as follows:

A. TRUMPET

Trumpet interchanges incorporate all the principles except F, since loop ramps are generally designed for minimum rather than desirable speeds.

B. DIAMOND

Diamond interchanges incorporate all the listed criteria and require the least amount of property acquisition. However, they have potentially serious operational problems due to the at-grade intersections created at the crossroad-ramp terminals, and especially due to the conflict involved with left turns at the terminals. Left turns on the crossroad have varying effects depending on crossroad and ramp traffic volumes. With low volumes and "stop" control on the ramp, long sight distances (see Section 6.04.07 A) must be provided along the crossroad to permit safe left turns. As volumes increase, multi-phase traffic signals must be installed which reduces ramp capacity. At the extreme, traffic will back up on the ramp to the point where the freeway lanes are blocked. These capacity problems may, however, be alleviated by widening the ramp in the vicinity of the crossroad terminal to two or more lanes in order to provide storage and added capacity.

C. CLOVERLEAF

Cloverleaf interchanges violate principles A, B, D, E and F. However, when used in conjunction with C-D roads, they then conform to all principles with the possible exception of the design speed on the inner loops. R.O.W. requirements for cloverleafs are high (a minimum of 25 acres are needed for the typical freeway cloverleaf).

D. DIRECTIONAL

Directional interchanges can be designed to incorporate all the listed criteria by use of "semi-direct" connections. "Direct" connections (see Section 6.04.01) involve left-hand exits and entrances, and should generally be avoided unless major fork criteria are applicable. R.O.W. requirements for directional interchanges are generally the same as or smaller than cloverleafs. Construction costs for directional interchanges are normally substantially greater than for any other form of interchange due to the multiple long span and/or multi-level structures required.

E. ROTARY

Rotary interchanges may be generally acceptable if the main line through roadways are separated from the rotary and only interchanging traffic and minor side roads are brought directly into the circle. In that case, the effect on the main line is somewhat similar to that caused by a cloverleaf with C-D roads. Signing for rotary interchanges is difficult and requires careful consideration.

6.05.03 PREFERRED INTERCHANGE TYPE

In summary, the following should be considered as the standard Freeway Interchanges for highway design in New York State.

A. THREE-LEG INTERCHANGES

The trumpet interchange will be standard with the freeway being the through route and the side road using the loop. The Trumpet-A (Figure 6-A) is the preferred shape since it places the exit ahead of the crossroad structure.

When all three legs are freeway or equivalent high-volume and/or high-speed facilities, the directional T or directional Y should be used to avoid the speed reduction required by the loop.

B. FOUR LEG-INTERCHANGES

1. Rural Freeway - Two-Lane Rural Crossroad

The conventional diamond is the standard interchange for this condition. The crossroad through the interchange area should be divided with a median of sufficient width to provide left turn storage lanes and full right shoulders should be provided on the crossroad through the interchange area. This treatment should take place at all diamond interchanges in this classification, since it is necessary for the safety of the traveling public and since the mere presence of a Freeway Interchange may well stimulate development of the surrounding area with motorist service facilities and industry which will generate traffic volumes far in excess of what can be predicted by normal forecasting methods. Exit ramps should be designed to provide the required number of lanes for the turning movements (or as required for storage). Bypassing vehicles waiting for a left turn opportunity should be considered. Except under very low volumes two lanes would normally be required at the crossroad terminus of a ramp. Exit ramp crossroad terminals must be carefully designed and signed both on the crossroad and the ramp to eliminate, insofar as possible, the chance of wrong-way entry onto the freeway.

2. Rural Freeway - Rural Multilane Primary Highway

The basic interchange shape for this condition is the cloverleaf with C - D roads on both highways. The C - D road may be omitted on the crossroad in rare cases where weaving volumes are extremely low. The crossroad must, in those cases, be fully divided through the interchange area even though no median divider exists on the approaches. When availibility of R. O. W. is restricted, or when left turning volumes are low, a diamond interchange with signal controls at the crossroad ramp terminals may be used. The above criteria should be applied to interchanges between two rural multilane primary highways and with suburban surface arterials.

3. Urban Freeways - Local Streets

When one-way frontage roads are provided, interchanges between urban freeways and local streets should be made by means of diamond type slip ramps connecting the freeway to the frontage road. When frontage roads are not provided, the standard type of interchange between freeways and the local street system should be the split diamond, preferably with one-way crossroads. At urban surface arterials, diamond interchanges are usually still sufficient. However, when left turning volumes become extremely high, cloverleaf interchanges with C - D roads on the freeway, and even directional interchanges, may be warranted.

4. Freeway - Freeway Interchanges

Freeway to freeway connections normally will be directional interchanges designed to the criteria listed herein, that is, no weaving, all right-hand ramps, single exits and where feasible, single entrances. In rural areas, when interchanging volumes are low, cloverleafs, with C - D roads on both highways, may sometimes be substituted for the directional interchange.

6.05.04 OTHER FACTORS TO BE CONSIDERED IN INTERCHANGE DESIGN

A. DESIRABILITY OF PLACING CROSSROAD OVER MAIN LINE

When designing a major-minor interchange, it is generally considered preferable, all other factors being equal, to so design the profiles as to have the crossroad (minor) elevated in relation to the main line (major). This configuration has several advantages: first, the driver gets additional visual identification of the approaching interchange when he sees the overpass structure in the distance; second, main line off ramps will be on upgrades and on ramps on downgrades, thus giving gravity assist to the necessary speed changes; third, in the case of new freeway construction in urban and suburban areas, if the desired relationship is accomplished by keeping the crossroad at or near existing grade and depressing the main line, reduced public opposition will normally result because depressed highways are generally preferred since they are not as visible and there is a reduction in noise level.

B. UNIFORMITY OF OPERATION

When travelling along the freeway system, the driver is usually confronted with several different interchange configurations and ramp locations, i.e. diamond, partial cloverleaf, full cloverleaf, semi directional, full directional, trumpet, loop ramps, diagonal ramps, right-hand exits, left-hand exits, single exits, double exits, weaving sections, etc. The unfamiliar driver, who typically knows only the name of the community for which he is heading and the name or number of the crossroad at which he will exit, but rarely knows whether his destination is north or south (or east or west) from the facility on which he is travelling, becomes confused through not knowing in advance whether his exit is before or after the crossroad structure, whether he wishes to take the first or second or even whether there is one or two exits at his interchange. Since even one confused driver slowing down, or in extreme cases even stopping, on the through lanes of a high speed facility creates a serious hazard, it is highly desirable to minimize this confusion, to the greatest extent possible, by designing all interchanges in such a manner that they operate uniformly from the viewpoint of the driver on the through facility. Since the majority of interchanges on the freeway system are diamonds, which are single exit interchanges with the exit ramp in advance of the crossroad structure, this pattern should generally be followed for all other interchanges. It will be noted that this conclusion is consistent with the principles established in Section 6.05.01.

C. GRADING OF INTERCHANGE AREAS

Complete contour grading plans at 1" = 50' scale with 2' contour interval, must be prepared for all interchange areas. This requirement applies even in those cases where contour grading plans are not prepared for the balance of the project. This grading plan, which is to be submitted for review with the Phase V Advance Detail Plans, is to be prepared with a view towards esthetically fitting the finished interchange into the surrounding topography through the use of gentle roundings to provide a smoothly rolling country-side. Existing shrubbery is to be retained wherever possible, subject to safety and sight distance criteria. This objective cannot be met through the use of "railroad cuts and fills" but rather must be accomplished through a smooth blending of existing and proposed contours to give a finished product which is both pleasing to the eye and safe for the travelling public.

6.06 SAFETY CONSIDERATIONS

6.06.01 GENERAL

Interchanges are areas of conflict and generally more hazardous than the open highway. It is , therefore, incumbent on the designer to pay careful attention to the details of the design in interchange areas. Minimum geometric criteria which are acceptable on the open highway, should not be used within the zone of influence of an interchange. Interchange areas should be designed to the highest feasible standards.

6.06.02 STOPPING SIGHT DISTANCE

A. EXIT RAMPS

In general, exit ramp noses should not be located beyond the high point of a crest vertical curve or behind an overpass. If, however, overriding considerations make this impossible, the desirable stopping sight distance as listed in Section 5.05.03 should be used.

B. ENTRANCE RAMPS

Good visibility is required at an entrance ramp to alert through traffic that vehicles are entering and to permit entering drivers to select acceptable gaps for merging. The sight distance criteria at entrance ramps is therefore similar to that for exit ramps. The desirable stopping sight distance as shown in Section 5.05.03 should be used.

C. ABSOLUTE MINIMUM

In no case shall the stopping sight distance in an interchange area be lower than the minimum prescribed for the same highway away from interchanges.

6.06.03 GORES

Leading gores should be gently graded and kept free of fixed objects for a distance of 400 feet from the pavement edge line intersection or 250 feet from the actual pavement split depending upon which provides the longer recovery area in the gore. Curbs, if used, should be a maximum of one inch in height at the nose. The low curb should be used around the radius of the nose plus 15 feet on each side. The only structures normally permitted within the gore area as defined above, are frangible "Exit" signs. Under certain conditions, normally in urban areas, such as an exit ramp on a structure, it may be impossible to keep the gore area free of fixed objects. In those cases, impact attenuation devices must be provided to protect on coming traffic and one inch high curb should be used to a line extended from the back of the attenuators.