FOR JKR INTERNAL USE ONLY

ARAHAN TEKNIK (JALAN) 12/87

JABATAN KERJA RAYA

A GUIDE TO THE DESIGN OF INTERCHANGES

CAWANGAN JALAN
IBU PEJABAT J.K.R.,
JALAN SULTAN SALAHUDDIN,
50582 KUALA LUMPUR.

HARGA: RM14.00

PREFACE

This Arahan Teknik (Jalan) on "A Guide To The Design Of Interchanges" is to be used for the geometric design of all grade-separated intersections. It is to be used in conjunction with Arahan Teknik (Jalan) 8/86 - "A Guide To Geometric Design Of Roads" and Arahan Teknik (Jalan) 11/87 - "A Guide To The Design Of At-Grade Intersections" and other relevant Arahan Tekniks.

While the geometric standards indicated in this Arahan Teknik is to be followed at all times, it is recognised that in some instances, due to site constraints or otherwise, the required standards may not be attainable except at a highly prohibitive cost. In such instances, the engineer/consultant should refer to his superior/client for a final decision, although the concepts of safety and design expressed in this Arahan Teknik should always be maintained.

The user is also encouraged to study the various references as indicated in the Appendix to fully understand some of the concepts and approaches adopted in this Arahan Teknik.

This Arahan Teknik will be updated from time to time and in this respect any feedback from users will be most welcome. Any comments should be sent to Cawangan Jalan, Ibu Pejabat JKR, Malaysia.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1: GENERAL PRINCIPLES

There are three general types of intersections:-

(a) at grade intersections

(b) highway grade separations without ramps

(c) interchanges

Each has a field of usage in which it is practical, but the limits are not sharply defined. There is much overlapping and the final selection of an intersection type frequently is a compromise after joint consideration of the class of intersecting roads, degree of access control, design traffic volume and pattern, cost, topography, and availability of right-of-way. The general principles as regards interchanges are as follows:-

1.1 TRAFFIC AND OPERATION

Through traffic has no difficulty or delays at highway grade separations. Ramps at interchanges have no severe effect on through traffic except where the capacity is inadequate, the merging or speed-change lanes are of inadequate length, or a full complement of turning roadways is not provided.

Turning traffic can affect operation and is accomodated to varying degrees on at-grade intersections and interchanges. At interchanges, ramps are provided for the turning movements. Where turning movements are light and some provision must be made for all turning movements, a one-quadrant ramp design may suffice. Ramps provided in two quadrants may be located so that crossings and the major highway is free of such interference. An interchange with a ramp for every turning movement is suited for heavy volumes of through traffic and for any volume of turning traffic, provided the ramps and terminals are designed with sufficient capacity.

Confusion by some drivers appears unavoidable on interchanges, but such difficulties are minor as compared with the benefits derived from in the reduction of delays, stops, and accidents. As interchange designs are improved, and as the quality and use of signing and other control devices are increased, confusion is minimised.

Interchanges are also adaptale to all kinds of traffic. The presence in the traffic stream of a high proportion of heavy trucks makes provision of interchanges especially desirable.

1.2 SITE CONDITIONS

In rolling or hilly topography, interchanges usually can be well fitted to the existing ground, and the through roads often can be designed to higher standards.

Interchange design is simple in flat terrain, but it may be necessary to introduce grades that may not favour operation. Interchanges in flat terrain generally are not as pleasing in appearance as those fitted to rolling terrain. When it is possible to regrade the whole of the interchange area and to landscape it properly, most of the deficiency in appearance can be overcome.

The right-of-way required for an interchange is dependent largely on the number of turning movements for which separate ramps need to be provided. The actual area required for any particular interchange also depends on the types of highway, topography, and the overall standards of interchange development. Related to right-of-way requirements is also the impact on property access that may occur with provision of an interchange.

1.3 TYPE OF HIGHWAY AND INTERSECTING FACILITY

As the hazard from stopping and direct turns at an intersection increases with the design speed, a high design-speed highway thus would warrant interchange treatment earlier than low-design-speed roads although with similar traffic volumes. The ramps provided for high design-speed highways should permit suitably high turning speeds, for which built-in facilities for changing speed assume greater importance.

The type of interchange will vary with the terrain, development along the highway, and right-of-way conditions, but in general it will be based on ramp layouts to expedite entrance to or exit from the expressway. Ramp connections may also involve frontage roads.

Local service can be readily provided on certain types of at-grade intersections, whereas considerable additional facilities may be necessary in the case of interchanges.

1.4 SAFETY

By separating the grades of the intersecting roadways, accidents caused by crossing and turning movements can be reduced. The grade separation structure, itself, is somewhat a hazard. However, this can be minimised by the use of adequate clear roadway widths and protective devices at bridge abutments and piers.

Depending on the type of interchange used, right-turns may be entirely eliminated or confined to the lower classified crossroad. Left-turning traffic can be accomodated on high design type of ramps which provide operation approaching the equivalent of free flow. Conflicts caused by crossing traffic will be eliminated or minimised.

1.5 STAGE DEVELOPMENT

THe projection of traffic for use in the design should be based on a period of 20 years after completion of the road. In areas where traffic estimation is difficult due to uncertainty in land use, planning or roadside interference, the design of the formation width shall be based on a period of 20 years, but some of the ramps and pavement construction may be staged basing on a 10 year period for the initial stage.

Where the ultimate development consists of a single grade-separation structure, stage construction may be uneconomical unless provisions are made in the original design for a future stage of construction. Ramps, however, are well adapted to stage development.

1.6 ECONOMIC FACTORS

1.6.1 Initial Costs

The interchange is the most costly type of intersection. The combined cost of the structure, ramps, through roadways, grading and landscaping of large areas, and possible adjustments in existing road-ways and utilities generally exceeds the cost of an at-grade intersection. Directional interchanges involve more than one structure, and their cost is usually greater than any simple interchange.

1.6.2 Maintenance Costs

Each type of intersection has distinct maintenance costs. Interchanges have large pavement and variable slope areas, the maintenance of which, together with that of the structure, signs, and landscaping, exceeds that of an at-grade intersection. Interchanges oftern involve maintenance and operation costs for lighting.

1.6.3 Vehicular Operating Costs

Through traffic at an interchange usually follows a direct path with only minor speed reduction. The added vehicular costs for the rise and fall in passing over and under the structure may need to be considered only when grades are steep. Left turning traffic is subject to added vehicular costs of deceleration and acceleration and may also be subject to the costs of operation on a grade, but travel distance is usually shorter than that on an at-grade intersection. Right-turning traffic is also subject to the added costs of acceleration and deceleration and usually to the added travel distance compared with that of the direct right turns at grade. Directional ramps may eliminate large speed changes and save travel distance and time as compared with at-grade intersections.

For intermediate-to-heavy traffic the total vehicle-operating costs at an intersection usually will be lower with an interchange than with an at-grade design, especially if the through movements predominate.

CHAPTER 2: WARRANTS FOR GRADE SEPARATION AND INTERCHANGES

An interchange is a useful and an adaptable solution for many intersection problems, but because of the high initial cost, it is used to eliminate existing traffic bottlenecks or to correct existing hazardous conditions and is limited to those cases where the required expenditure can be justified. Because of the wide variety of site conditions, traffic volumes, highway types, and interchanges layouts, the warrant that justify an interchange may differ at each location. The conditions that should be considered to reach a rational decision are the available warrants. These are as shown in the following items:

2.1 DESIGN DESIGNATION

The decision to develop a highway with full control of access between selected terminal becomes the warrant for providing highway grade separations or interchanges for all intersecting highways.

Although access control, provision of medians, and elimination of parking and pedestrian traffic are important, the separation of grades on highways provide the greatest increment of safety.

An intersection that might warrant only traffic signal control if considered as an isolated case, may warrant a grade separation or interchange when considered as a part of a highway.

2.2 ELIMINATION OF BOTTLENECKS OR SPOT CONGESTION

An inability to provide essential capacity with an at-grade facility provides a warrant for an interchange where development and available right-of-way permits. Even on facilities with partial control of access, the elimination of random signalization contributes greatly to improvement of free-flow characteristics.

2.3 ELIMINATION OF HAZARDS

Some at-grade intersections have a disproportionate rate of serious accidents. In the lack of inexpensive methods of eliminating hazards, a highway grade separation or interchange may be warranted. Accident-prone intersections frequently are found at the junctions of comparatively lightly travelled highways i sparsely settled rural areas where speeds are high. In such areas, structures usually can be constructed at little cost compared with 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.

2.4 SITE TOPOGRAPHY

At some sites grade-separation designs are the only type that can be made economically. The topography at the site may be such that any other type of intersection, to meet required standards, is physically impossible to develop or is equal or greater in cost.

2.5 ROAD USER BENEFITS

The road-user costs due to delays at congested at-grade intersections are large. Such items as fuel, tires, oil, repairs, travel time and accidents that require speed changes, stops, and waiting generate expenses well in excess of those for intersection permitting uninterrupted or continuous operation. In general, interchanges require somewhat more total travel distance than direct crossings at grade, but this may be offset by the gains due to the reduction in stopping delay. The relation of road-user benefits to the cost of improvement indicates an economic warrant for that improvement.

Furthermore, interchanges usually are adaptable to stage construction, and initial stages may produce incremental benefits that would compare even more favourably with incremental costs.

2.6 TRAFFIC VOLUME WARRANT

Although a specific volume of traffic cannot be predetermined as a warrant for an interchange, it is nevertheless an important guide, particularly when combined with the traffic distribution pattern and the effect of traffic behaviour. Volumes in excess of the capacity of an at-grade intersection would certainly be a warrant.

2.7 OTHER WARRANTS

Additional warrants for grade separation and interchanges that can be considered are as follows:-

(a) Local roads and streets that cannot be feasibly terminated outside the right-of-way limits of expressways.

(b) Access to areas not served by frontage roads or other means of access.

(c) Railroad grade separations.

(d) Unusual concentration of pedestrian traffic (for instance, a city park developed on both sides of a major arterial).

(e) Cycletracks and routine pedestrian crossings.

(f) Access to mass transit stations within the confines of a major arterial.

(g) Free-flow aspects of certain ramp configurations and completing the geometry of interchanges.

2.8 JUSTIFICATION FOR CLASS OF ROAD

If such warrants as the elimination of bottlenecks and hazards, as well as the site topography do not apply, and in addition specific information on the warrants for design designation, road user benefit, and traffic volume is not available, Table 2-1 would be helpful to judge whether an interchange is justified or otherwise. If two alternatives: IC (interchange) and S.I. (signalised intersection) are given in the table, provision of an interchange can be considered if the total projected peak hour traffic volume on the four approaches exceeds 8000 v.p.h.

According To Class Of Roads Crossing.

a. Rural Area

EXPRESSWAY	HIGHWAY	PRIMARY	SECONDARY	MINOR	
IC.	IC.	IC.	IC.	IC.	EXPRESSWAY
	IC.	I.C./S.I.	S.I./S.C.	S.C.	HIGHWAY
		S.I.	S.I./S.C.	S.C.	PRIMARY
			S.C.	S.C.	SECONDARY
				S.C.	MINOR

b. Urban Area

EXPRESSWAY	ARTERIAL	COLLECTOR	LOCAL STREET	
IC.	IC.	IC	IC	EXPRESSWAY
	IC./ S.I.	S.I.	S.I./ S.C.	ARTERIAL
		S.I.	S.I.	COLLECTOR
			S.C.	LOCAL STREET

IC. : INTERCHANGE.
S.I. : SIGNALIZED INTERSECTION.
S.C. : STOP CONTROL.

TABLE 2-1: SELECTION OF INTERSECTION TYPE

CHAPTER 3: GRADE SEPARATION STRUCTURES

3.1 TYPES OF SEPARATION STRUCTURES

3.1.1 General

The separation structure should conform to the natural lines of the highway approaches in alignment, profile, and cross-section. In addition to the above geometric considerations, other conditions such as the lengths of spans, depths of structures, foundation material at the site, aesthetics, safety and especially skew may substantially influence the engineering and cost feasibility of the structure being considered.

3.1.2 Overpass

For the overpass highway, the deck-type structure is most suitable. Although they may present both lateral and vertical clearance problems on the lower roadway, the supports relative to the upper roadway are underneath and out of sight.

3.1.3 Underpass

For the underpass highway, the most desirable structure from the standpoint of vehicular operation is one that will span the entire highway cross section and provide a lateral clearance of structural supports from the edge of pavement that is consistent with the design requirements.

On divided highways, center supports should only be used when the median is wide enough to provide sufficient lateral clearance or narrow enough to require protective barriers. In anticipation of future widening, the piers or abutment design must provide footings with sufficient cover after widening. A greater sense of openess also results with open end spans than will full depth abutments.

Underpasses are preferred in urban areas and should be used wherever possible. However, attention should be placed on foundation, geotechnical and drainage problems that may arise.

3.2 OVERPASS VERSUS UNDERPASS

3.2.1 General Design Considerations

A detailed study should be made at each proposed highway grade separation to determine whether the main road should be carried over or under the structure. Often, the choice is dictated by features such as topography or highway classification.

As a general rule, the design that best fits the existing topography is the most pleasing and economical to construct and maintain and this factor becomes the first consideration in design. The chief exception to this is the case where a major road is sufficiently predominant in design to overweigh topographic and crossroad controls.

Where topography does not govern, as is common in the case of flat topography, it will be necessary to study secondary factors. The following general points may be noted:-

(a) There is certain traffic warning advantage on an undercrossing highway. As a driver approaches, the structure looms ahead, makes obvious the upper level crossroad, and gives advance warning of likely interchange connections. However, where an undercrossing highway dips beneath a cross-road which is at ground level, this advantage is minimised.

(b) Through traffic is given aesthetic preference by a layout in which the more important road is the overpass. A wide overlook is possible from the structure and its approaches, giving drivers a minimum feeling of restriction.

(c) Where turning traffic is significant, the ramp profiles are best fitted when the major road is at the lower level. The ramp grades then assist turning vehicles to decelerate as they leave the major highway and to accelerate as they approach it, rather than the reverse.

(d) Where there is no pronounced advantage to the selection of either an underpass or an overpass, the type that provides the better sight distance on the major road (desirably safe passing distance if the road is two-lane) should be preferred.

(e) An overpass offers the best possibility for stage construction both for the highway and the structure, with minimum impairment of the original investment. By lateral extention of both or construction of a separate structure and roadway for a divided highway, the ultimate development is reached without loss of the initial facility.

(f) Troublesome drainage problems may be reduced by carrying the major highway over without altering the grade of the crossroad. In some cases the drainage problem alone may be sufficient as the reason for choosing to carry the major highway over rather than under the crossroad.

(g) Where topography control is secondary, a cost analysis that takes into account the bridge type, span length, roadway cross section, angle of skew, spoil conditions, and cost of approaches will determine which of the two intersection roadways should be placed on structure.

(h) An underpass may be more advantageous where the major road can be built close to the existing ground, with continuous gradient, and with no pronounced grade changes.

(i) Where a new highway crosses an existing route carrying a large volume of traffic, an overcrossing by the new highway causes less disturbance to the existing route and a detour is usually not required.

(j) The overcrossing structure has no limitations as to vertical clearance, which can be a significant advantage in the case of oversized loads requiring special permits on a major highway or route.

(k) Desirably, the roadway carrying the highest traffic volume should have the fewest number of bridges for better ridability and fewer conflicts when repair and reconstruction are necessary.

(l) In some instances, it may be necessary to have the higher volume facility depressed and crossing under the lower volume facility to reduce noise impact.

3.3 CROSS-SECTION OF STRUCTURES

3.3.1 Structure Widths

The widths for all structures should follow the requirements as laid down in Section 5.12 of Arahan Teknik Jalan 8/86 - "A Guide To Geometric Design Of Roads".

3.3.2 Clearances

The vertical as well as horizontal clearances should also follow the requirements as laid down in Section 5.12 of Arahan Teknik Jalan 8/86 - "A Guide To Geometric Design Of Roads".

3.3.3 Barriers

Protective barriers should be considered for the abutments, piers and medians when lateral clearances are minimum. The guidelines given in Arahan Teknik (Jalan) 1/85 - "Manual On Design Guide Lines Of Longitudinal Traffic Barriers" should be followed.

3.4 GRADE SEPARATION WITHOUT RAMPS

There are many situations where grade separations are provided without ramps. Ramps are ommited to avoid having interchanges so close to each other that signing and operation would be difficult, to eliminate interference with large major road volumes, and to increase safety and mobility by concentrating turning traffic at a few points where it is feasible to provide adequate ramp systems. On the other hand, undue concentration of turning movements at one location should be avoided where it would be better to have several interchanges.

Where ramp connections are difficult or costly, it may be partial to omit them at the structure site and accomodate turnig movements elsewhere by way of other intersecting roads.

CHAPTER 4: INTERCHANGE TYPES

4.1 THREE LEG DESIGN

A three leg design type interchange is one at an intersection with three intersecting legs consisting of one or more grade separations and one-way roadways for all traffic movements. 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.

Figure 4-1 illustrates the patterns of three leg interchanges with one grade separation while Figure 4-2 illustrates high type T and Y interchanges each with more than one structure or with one three-level structure that provides for all of the movements without loops.

Regardless of such factors as the intersection angle and through road character, any one basic interchange pattern may apply for a wide variant of conditions.

4.2 FOUR LEG DESIGN

4.2.1 General

Interchanges with four intersection legs may be grouped under four types:-

(a) ramps in one quadrant (ramps in two or three quadrants are discussed as partial cloverleafs)

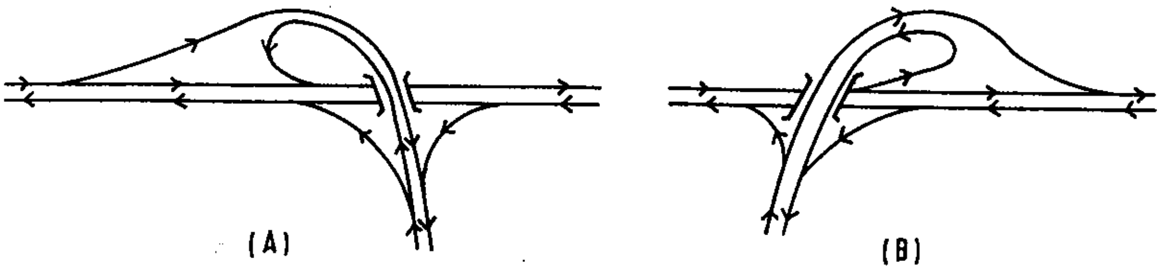
(b) diamond interchanges

(c) partial cloverleafs

(d) full cover leafs

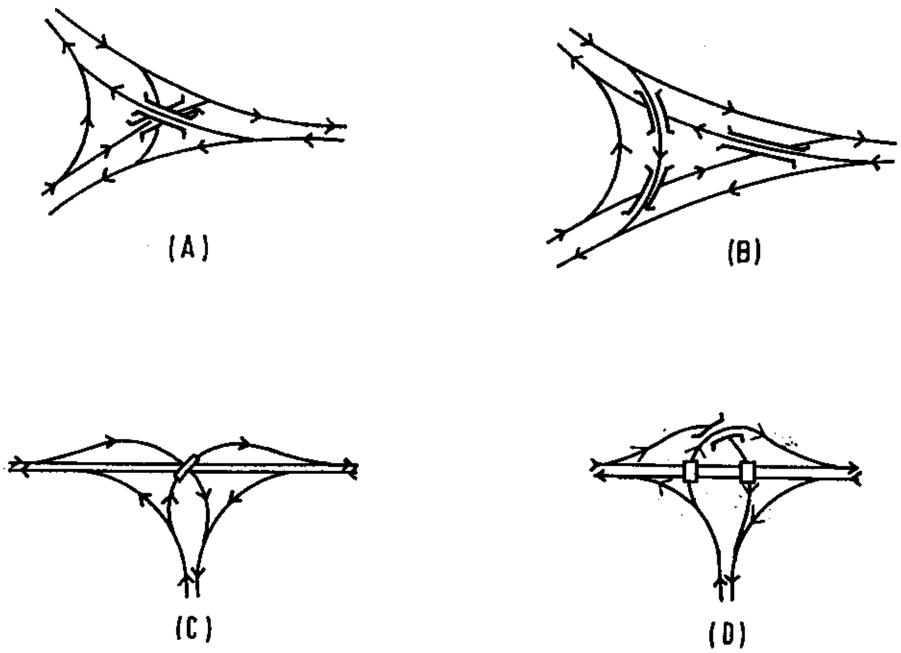
4.2.2 Ramps in one Quadrant

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 2 way ramp of minimum design will usually suffice for all turning traffic. The ramp terminals may be plain T intersections, as illustrated in Figure 4-3.



THREE - LEG INTERCHANGES WITH SIMPLE STRUCTURES

FIGURE 4-1



THREE - LEG INTERCHANGES WITH MULTIPLE STRUCTURES

FIGURE 4-2

Locations where this type of design are applicable are limited. At some interchanges, it may be necessary to limit ramp development to one quadrant because of topography, culture, or other controls, even though the traffic volume justifies more extensive turning facilities. In such a case, a high degree of channelisation at the terminals and the median, together with right-turn lanes on the through highway is required to control turning movements.

Where a one quadrant interchange is constructed as the first step in a stage construction program, the initial ramps should be designed as part of the overall development. One or both of the terminals can be interchanges of T type design. This results in offsetting the interchanges from the place of grade separation structures. The concept can be applicable to the cases where ramp connections at the separation structure are difficult.

4.2.3 Diamond Interchanges

This is the simplest and most common type of interchange. A full diamond interchange is formed when a 1-way diagonal type ramp is provided in each quadrant. The ramps are aligned with free flow terminals on the major highway and the at-grade right turns are confined to the crossroad.

The diamond interchange has several advantages over a comparable partial cloverleaf in that all traffic can enter and leave the major road at relatively high speeds, right turning movements entail little extra travel and a relatively narrow band or right of way is required.

Diamond interchanges have application in both rural and urban areas. They are particularly adaptable to major-minor crossings where right turns at grade on the minor road can be handled without hazard or difficulty. The intersection on the crossroad presents a problem in traffic control so as to prevent wrong way entry from the crossroad. For this reason, a median on the crossroad is important to facilitate proper channelisation. Additional signing will also help to prevent improper use of ramps and should be incorporated in the design.

Diamond interchanges usually require signalisation when the crossroad carry moderate to heavy traffic volume. A single lane ramp usually serves traffic from the through highway, but it may have to be widened to two or three lanes in order to develop the necessary capac ity for the at grade condition. This would ensure that stored vehicles would not exceed too far along the ramps or onto the highway, thereby resulting in undesirable back-up effect on the major through road.

Diamond interchanges can assume a variety of patterns. Figure 4-4 illustrates examples of conventional types while Figure 4-5 illustrates diamond interchange arrangements to reduce traffic conflicts, by utilising split diamond or frontage roads and separate turnabout provisions. An undesirable feature of the split diamond is that traffic leaving the highway cannot return to the major through road at the same interchange and continue in the same direction. Figure 4-6 shows diamond interchanges with more than one structure.

Traffic signals are used in high volume situations, and their efficiency is dependent on the relative balance in right-turn volumes. They are normally synchronised to provide continuous movement through a series of right turns once the area is entered.

4.2.4 Cloverleafs

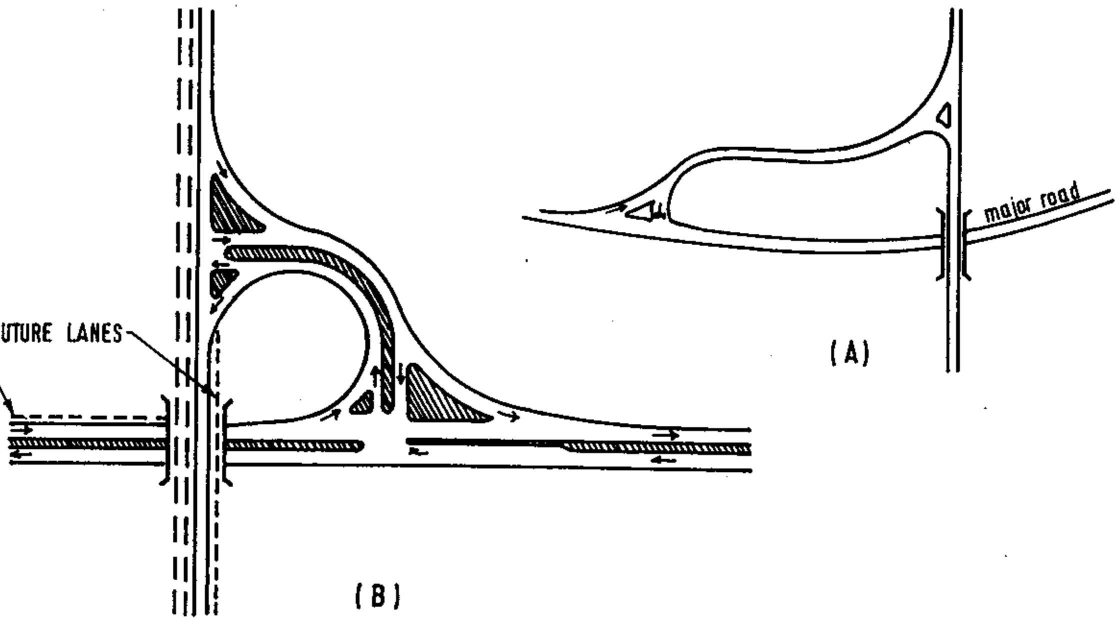
Cloverleafs are fourleg interchanges which employ loop ramps to accomodate right-turning movements. Interchanges with loops in all quadrants are referred to as ´full cloverleafs´ and all other as ´partial cloverleafs´.

The principal disadvantages of the cloverleaf are the extra travel distance required for right turning traffic, the weaving manoeuver generated, the very short weaving length typically available and the relatively large right of way required. When collector-distributor roads are not used, further disadvantages are weaving on the main carriageways, double exists and problems associated with signig for the second exit.

Because cloverleafs are so expensive - considerably more than diamond interchanges, they are less common in urban areas and are better suited to suburban or rural areas where space is available and there is a need to avoid restrictive at-grade right turns.

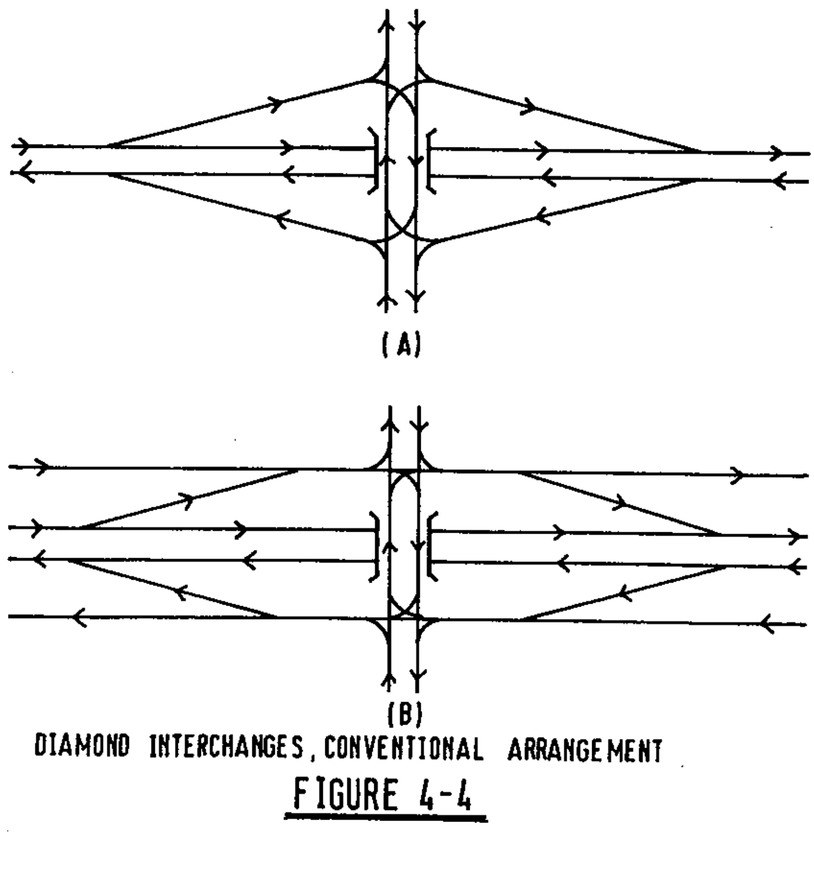
There is an advantage in cloverleafs of a higher speed, but this must be weighed against the disadvantages of extra travel time because of the longer distance and higher cost of an increased right of-way.

When the sum of traffic on two adjoining loops approaches about 1,000 vph, interference in weaving mounts rapidly, resulting in a reduction of capacity of the through traffic. Adequate weaving lengths are thus very important and should be provided. The weaving section should be transfered from the through lanes to



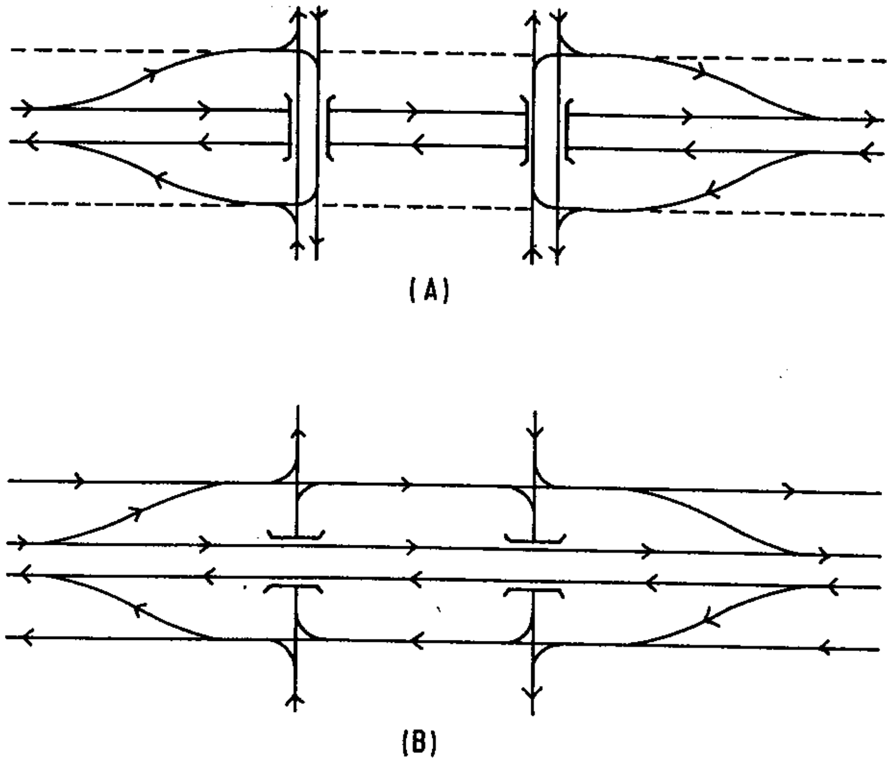
FOUR-LEG INTERCHANGES, RAMPS IN ONE QUADRANT

FIGURE 4-3



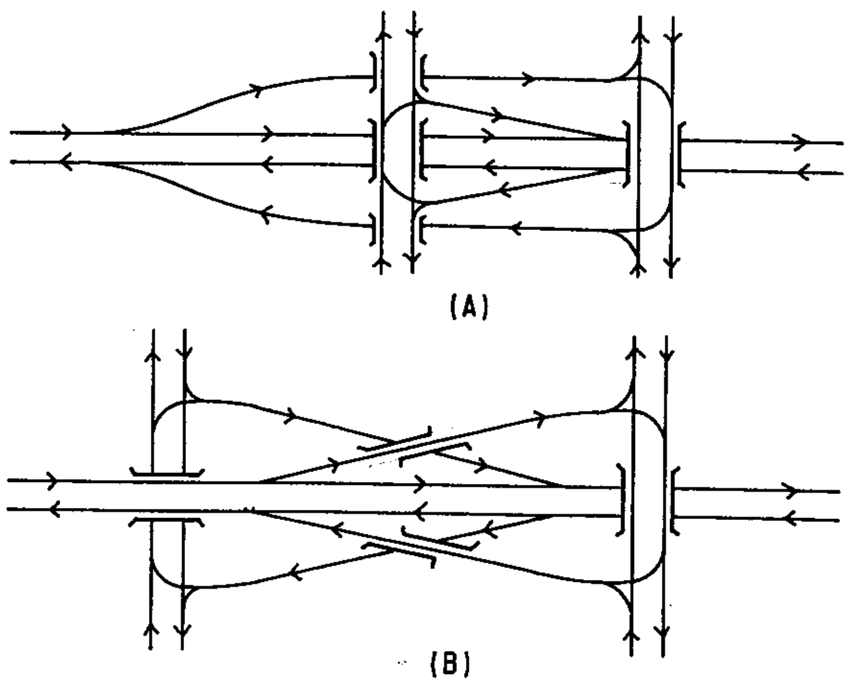
DIAMOND INTERCHANGES, CONVENTIONAL ARRANGEMENT

FIGURE 4-4



SPLIT DIAMOND INTERCHANGES

FIGURE 4-5



DIAMOND INTERCHANGES WITH ADDITIONAL STRUCTURES

FIGURE 4-6

a collector-distributor road when the weaving volume exceeds 1,000 vph.

A loop has a design capacity of 800 to 1,200 vph. Loop ramp capacity is therefore a major control in cloverleaf designs. Loops may be made to operate with two lanes, but this would involve careful attention to the design of the terminals. Two lane loops are not economical from the view point of right of way, construction cost, and direction of travel and are considered only in exceptional cases.

When a full cloverleaf interchange is used in conjunction with a highway, collector-distributor roads adjacent to the urgency should be used. For other highways, where weaving is moderate, full cloverleaf interchanges without collector-distributor roads are acceptable.

Figure 4-7 illustrates various patterns of cloverleaf interchanges.

4.2.5 Partial Cloverleaf Ramp Arrangements

For partial cloverleafs, ramps should be arranged so that the entrance and exit turns create the least interference to the traffic flow on the major highway. The following guidelines can be considered in the arrangements of ramps at partial cloverleafs.

(a) the ramp arrangement should enable the major turning movements to be made by left-turn exists and entrances.

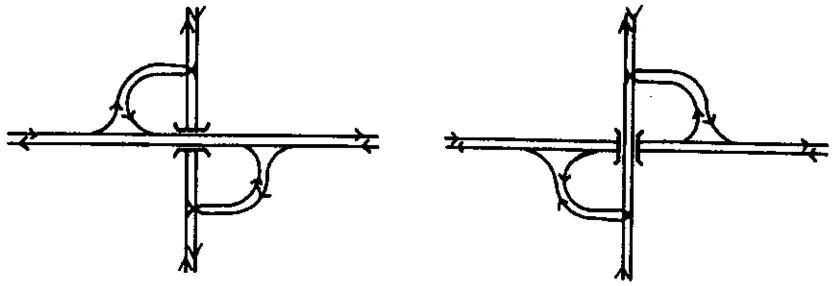
(b) where the through traffic volume on a major highway is decidedly greater than that on the intersecting minor road, preference should be given to an arrangement placing the left turns, either exit or entrance, on the major highway even though it results in a direct right turn off the crossroad.

At a particular site, topography and culture may be the factors that determine the quadrants in which the ramps and loops can be developed.

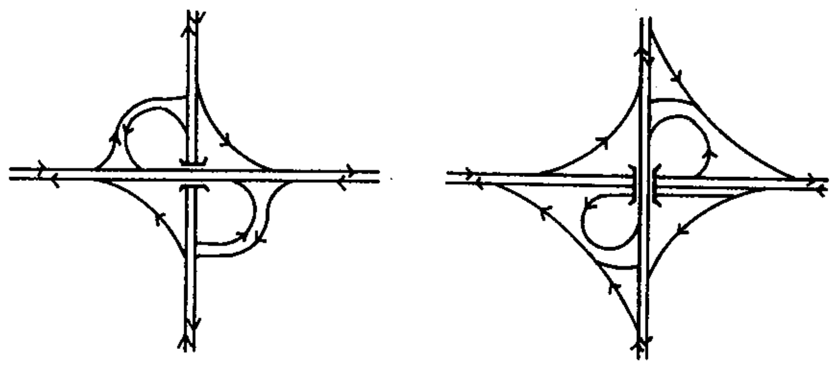
Figure 4-8 illustrates schematically some patterns of partial cloverleaf ramp arrangements.

4.3 DIRECTIONAL AND SEMI-DIRECTIONAL DESIGN

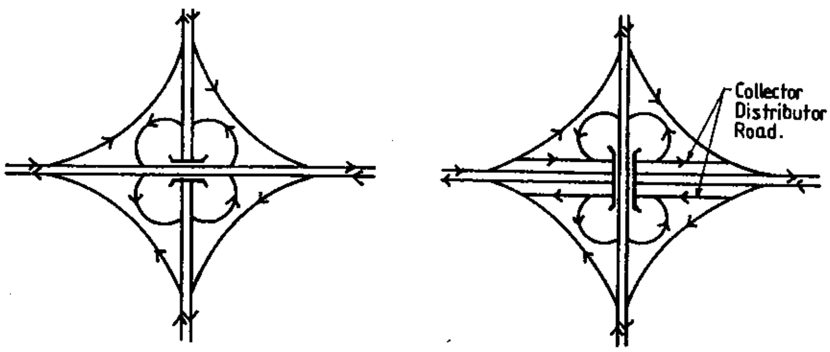
Direct or semidirect connections are used for important turning movements to reduce travel distance, increase speed, safety and capacity, eliminate weaving, and to



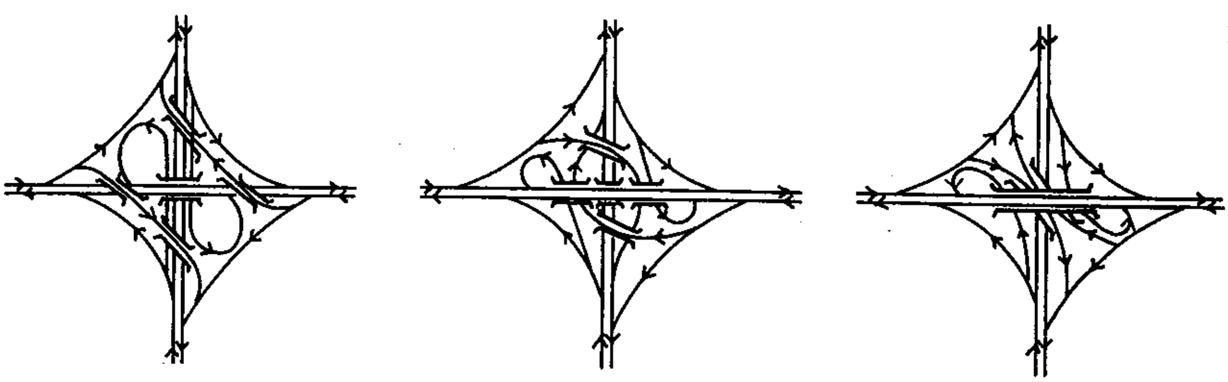
(A) PARTIAL CLOVERLEAF



(B) PARTIAL CLOVERLEAF WITH DIAGONAL RAMPS

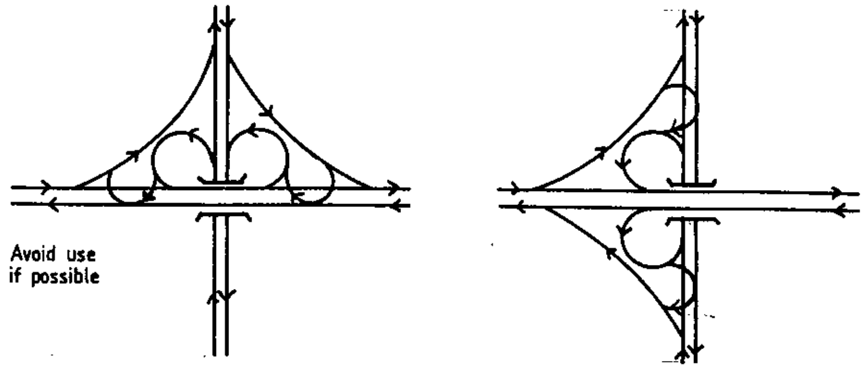


(C) FULL CLOVERLEAF

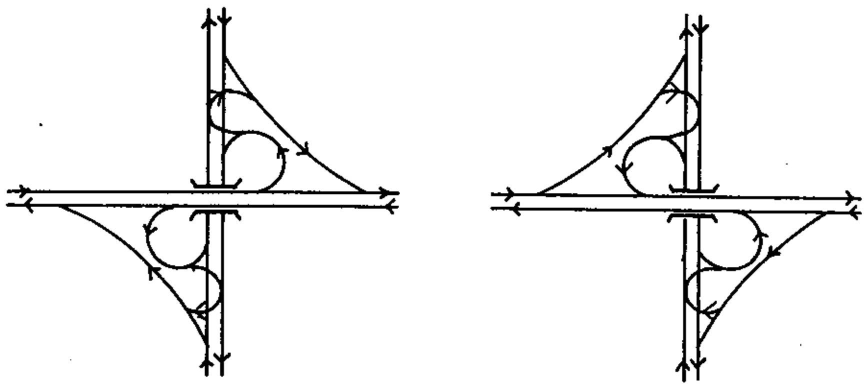


(D) MODIFIED CLOVERLEAF

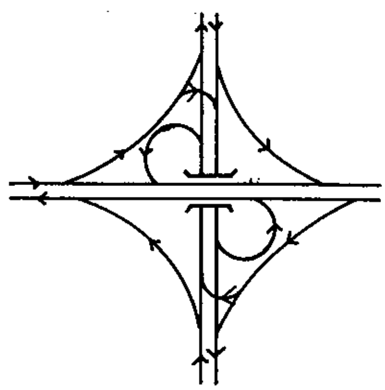
FIGURE 4-7



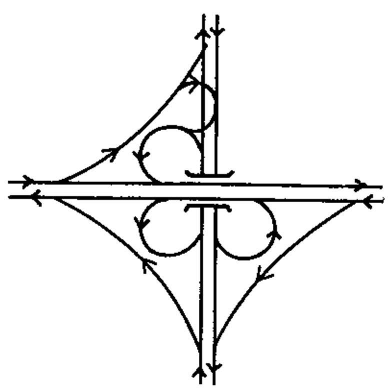
(A) TWO QUADRANTS



(B) TWO QUADRANTS DIAGONALLY OPPOSITE



(C) FOUR QUADRANTS



(D) THREE QUADRANTS

PARTIAL CLOVERLEAF RAMP ARRANGEMENTS

FIGURE 4-8

avoid the indirection of driving on a loop. Higher levels of service can also be realised on direct connections.

Where direct connections are designed with two lanes, the ramp capacity may approach the capacity of an equivalent number of lanes on the through highway.

In rural areas, there is rarely a volume justification for provision of direct connections in more than one or two quadrants. The remaining right turning movements are usually satisfactorily handled by loops or at grade intersections.

A direct connection is defined as a one-way roadway that does not deviate greatly from the intended direction of travel. Interchanges that use direct connections for the major right-turn movements are termed directional interchanges. When one or more interchange connections are indirect in alignment yet more direct than loops, the interchange is described on semi-directional. On direct or semi-direct interchanges, more than one grade separation is usually involved.

Semi-direct or direct connections are often required at major interchanges in urban areas. Interchanges involving two expressways nearly always call for directional layouts.

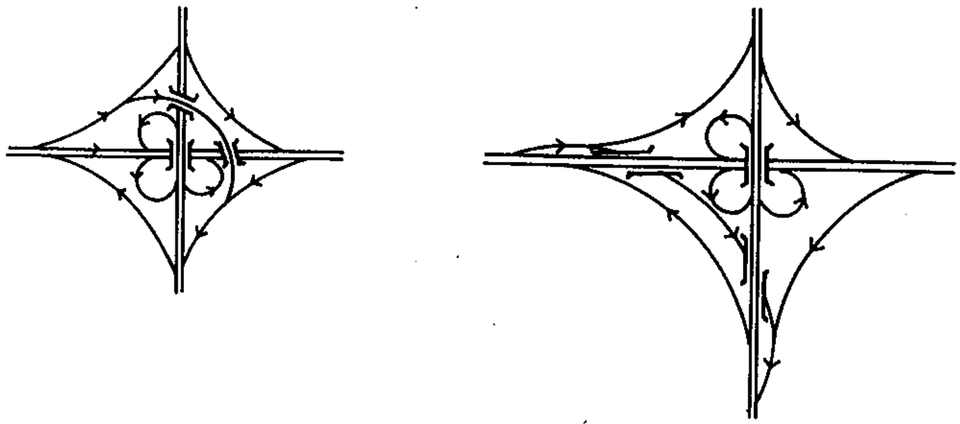
There are many scheme for directional interchanges that use various combinations of directional, semi-directional and loop ramps. Any one of them may be appropriate for a certain set of conditions, but only a limited number of patterns are used generally. These layouts fill the least space, have the fewest or least complex structures, minimised internal weaving and fit the common terrain and traffic conditions.

Basic patterns of semi-directional interchanges are illustrated in Figure 4-9. They basically are those with loops and involving no weaving maneouvers.

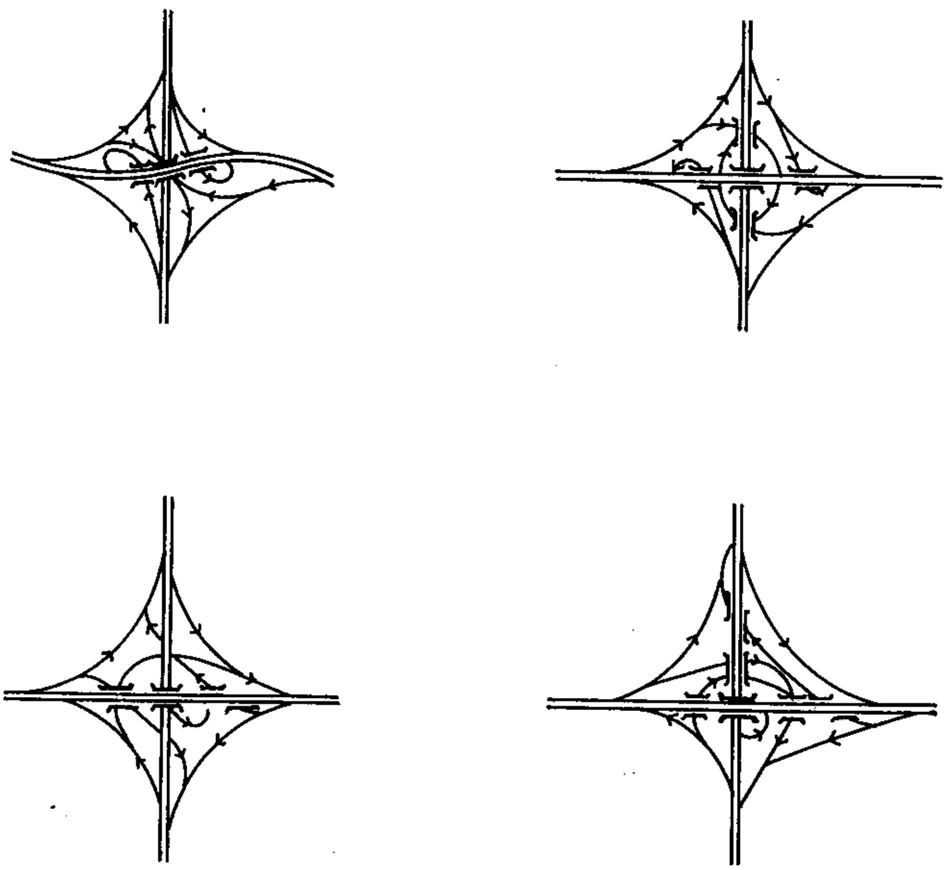
Fully directional interchange layouts have no weaving as this is extremely undesirable. Right hand exits and entrances are also undesirable but may be unavoidable because of site restrictions. The most widely used type of directional interchange is the four level layout system. Figure 4-10 illustrates patterns of full directional interchanges.

4.4 ROTARY DESIGN

A rotary interchange is a roundabout with the major through highway grade-separated. As this design eliminates some of the deficiencies of at-grade



(A) SEMIDIRECT INTERCHANGES (WITH WEAVING)



(B) SEMIDIRECT INTERCHANGES (WITH NO WEAVING)

FIGURE 4-9

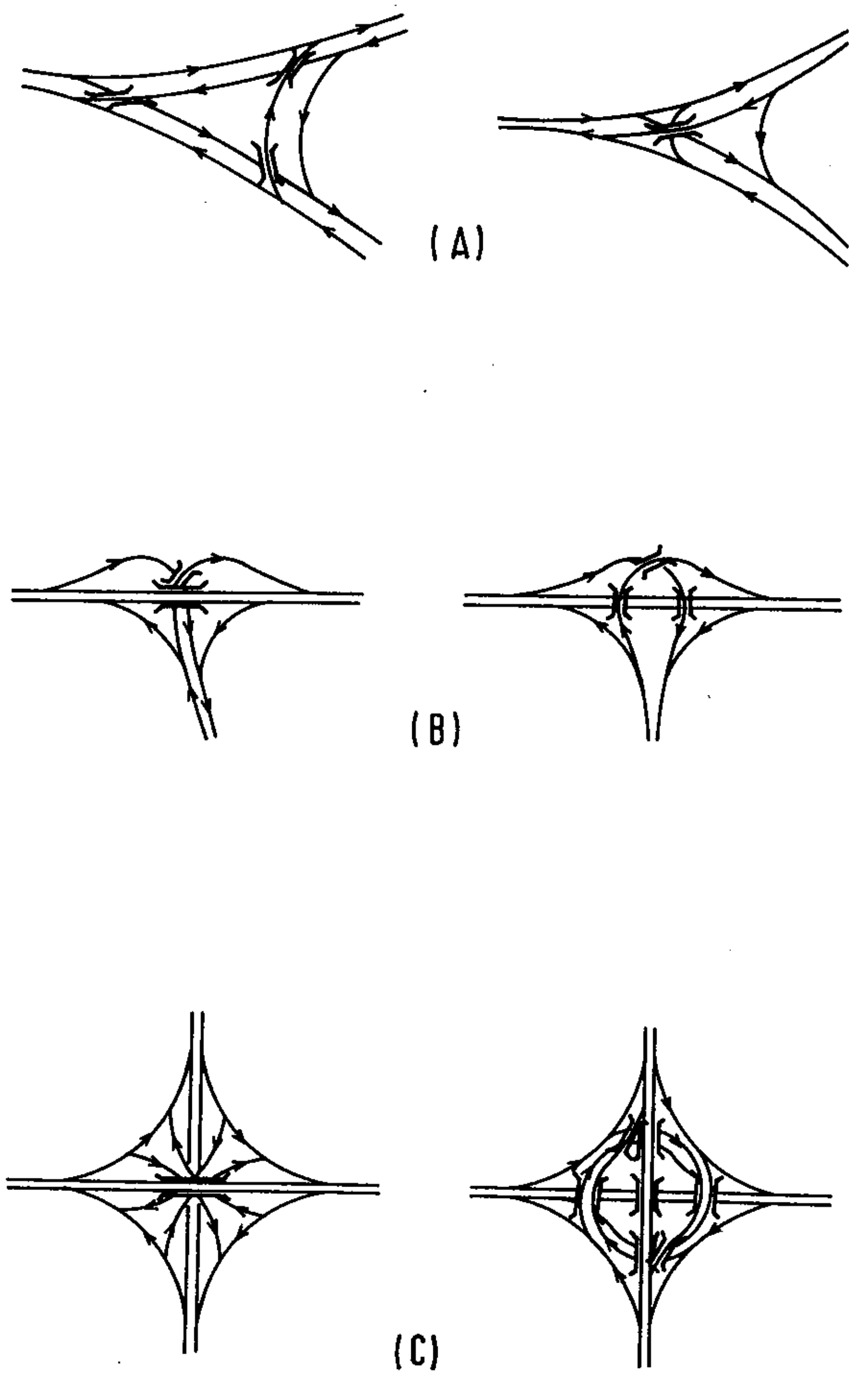


FIGURE 4-10: FULL DIRECTIONAL INTERCHANGES

intersections on the crossroad, smoother traffic flow than that of diamond interchange can be expected. However, it still has the same disadvantages of the roundabout and two grade separation structures are required in this design.

Through traffic on the minor crossroad must travel an extra distance on a circularly turning roadway. A vast area is also required to have sufficient weaving distances. With these drawnbacks, employment of rotary design is limited to extremely exceptional cases.

Ramps of cloverleaf can be arranged on the same size of area prepared for rotary interchange. Diamond interchanges can also replace this design.

Figure 4-11 illustrates some patterns of rotary interchanges.

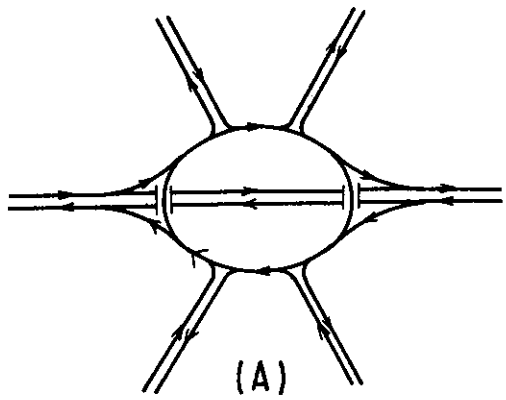
4.5 COMBINATION INTERCHANGES

When one or two turning movements have very high volumes with respect to the other turning movements, analysis may indicate the need for a combination of two or more of the above discussed types of interchanges. Typical examples are :-

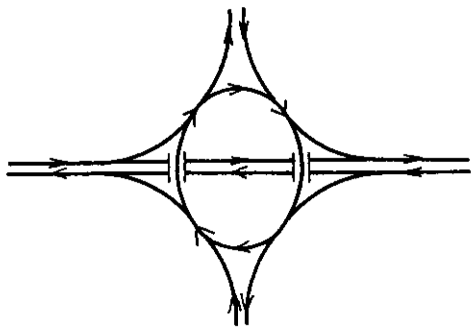
a) a diamond with a semi-direct connection

b) a cloverleaf with a semi-direct connection

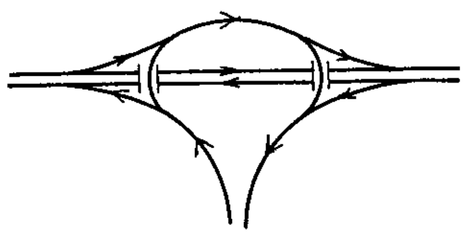
c) a one-half diamond with a one quadrant cloverleaf



(A)



(B)



(C)

FIGURE 4-11. ROTARY INTERCHANGES

CHAPTER 5: GENERAL DESIGN CONSIDERATIONS

5.1 INTERCHANGE TYPE DETERMINATION

(a) Systems Interchanges and Service Interchanges

Interchange types are covered in two categories, i.e. :- "systems interchanges" and "service interchange". Systems interchanges are used to identify interchanges that connect expressway to expressway while the term service interchange applies to interchanges that connect expressways to lesser facilities.

(b) Interchanges in Rural Area

In rural areas, the problem of interchange type selection is solved on the basis of service demand. When the intersecting roadways are expressways, all directional interchanges may be in order for high turning volumes.

A cloverleaf interchange is the minimum design that can be used at the intersection of two fully controlled access facilities or where right turns at grade are prohibited. A cloverleaf interchange is adaptable in a rural environment where right-of-way is not prohibitive and weaving is minimal.

The final configuration of an interchange may be determined by the need for route continuity, uniformity of exit patterns, single exit in advance of the reparation structure, elimination of weaving on the main facility, signing potential, and availability of right-of-way.

(c) Interchange In Urban Area

Generally, in urban areas, interchanges are so closely spaced that each interchange may be influenced directly by the preceding or following of interchange to the extent that additional traffic lanes may be required to satisfy capacity, weaving and lane balance.

On a continuous urban route all the interchanges shold be integrated into a system design rather than considered on an individual basis.

Generally, cloverleaf interchanges with or without collector-distributor road are not practicable for urban construction because of the excessive right-of-way requirements.

(d) Unbalanced Traffic Distribution

A combination of directional, semi-directional and loop ramps may be appropriate where turning volumes are high for some movements and low for others. When loop ramps are used in combination with direct and semi-direct ramp designs, it is desirable that the loops be arranged so that weaving sections are not created.

Partial cloverleaf designs may be appropriate where rights-of-way are not available in one or two quadrants or where one or two movements in the interchanges are disproportionate to the others, especially when they require right turns across traffic. In the latter case, loop ramps may be utilised to accomodate the heavy right turn volume.

(e) Cloverleaf Interchanges

In the decision process to use cloverleaf interchanges, careful attention should be given to the potential improvement in operational quality that would be realised if the design included collector-distributor roads on the major roadway.

(f) Diamond Interchanges

A capacity of a diamond interchange is limited by the capacity of the at-grade terminals of the ramps at the crossroad. High through and turning volumes could preclude the use of a simple diamond unless signalisation is used.

(g) Crossroad Capacity

The ability of the crossroad to receive and discharge traffic from the main roadway has considerable bearing on the interchange geometry. For example, loop ramps may be needed to elimiate heavy right turns on a conventional diamond interchange.

(h) Factors For Type Determination

Once several alternatives have been prepared for the system design, they can be compared on the following principles : (1) capacity, (2) route continuity, (3) uniformity of exit patterns, (4) single exits in advance of the separation structures, (5) with or without weaving, (6) potential for signing, (7) cost, (8) availability of right-of-way, (9) potential for stage construction, and (10) compatibility with the environment. The most desirable alternatives can be retained for plan development.

Figure 5-1 depicts interchanges that are adaptable on expressways as realted to classifications of intersecting facilities in rural and urban environments.

5.2 APPROACHES TO THE STRUCTURES

5.2.1 Alignment, Profile and Cross Section

Major Highways

The geometric standards at the highway grade separation should be higher than those for the highway to counterbalance any possible sense of restriction caused by abutments, piers, kerbs, and guardrails and to provide better sight distance for diverging and merging manoeuvers. Desirably, the alignment and profile of the through highways at an interchange should be relatively flat with high visibility. A sharp curve would induce the problem in the crossover crown between the superelevated portion through traffic lane and the pavement of the ramps located outside of the turning roadway. Sometimes it is possible to design only one of the intersecting roads on a tangent with flat grades. Preferably, the major highway should be so treated.

Table 5-1 shows the desirable lowest elements for alignment and profile design of the through highways at interchange area.

Cross Road

The gradients on intersecting road at an interchange should be kept to a minimum and in no case should they exceed the maximums established for open-highway conditions. Reduction of vehicle speeds by long upgrades encourage passing, which is hazardous in the vicinity of ramp terminals. Slow moving through vehicles also encourage abrupt cutting in by vehicles leaving and entering the highways.

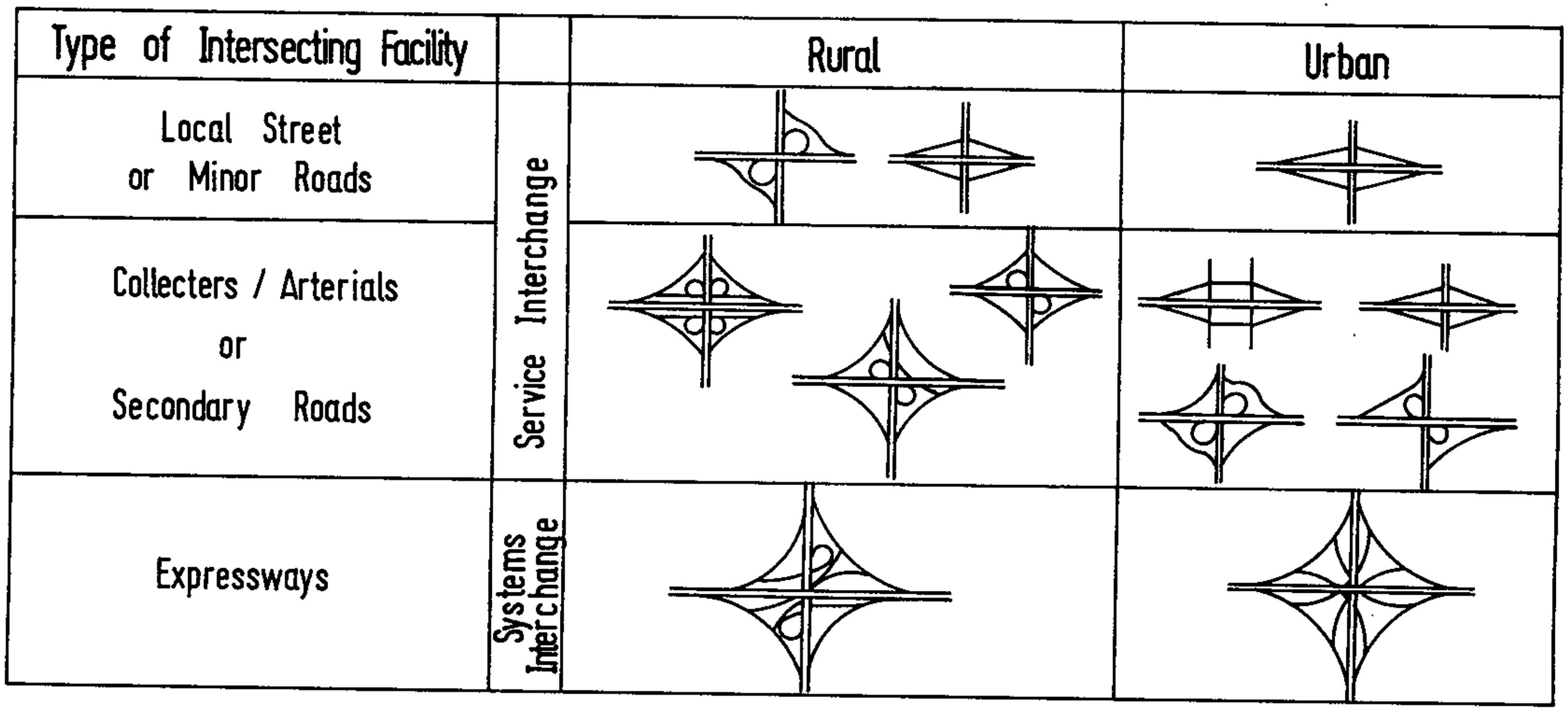


FIGURE 5-1: ADAPTABILITY OF INTERCHANGES ON EXPRESSWAYS AS RELATED TO TYPES OF INTERCHANGES FACILITY

TABLE 5-1: DESIRABLE LOWEST DESIGN ELEMENTS OF THROUGH HIGHWAYS AT INTERCHANGE AREA

Highway Design Speed (Km/h)	Minimum Radius (m)	Maximum Gradient (%)	Minimum Vertical Curve Length in K-Value	
			Crest	Sag
120	2,000	2	450	160
100	1,500	2	250	120
80	1,000	3	120	80
60	500	4.5	60	40
50	300	5	40	30

Alignment And Cross-Section

The alignment and cross section of the approaches to a grade separation without ramps involve no special problems except where a change in width is made to include a middle pier or where the median is narrowed for structure economy. With ramps, changes in alignment and cross section may be required to ensure proper operation and to develop the necessary capacity at the ramp terminals, particularly where there is not a full complement of ramps. Where a two-lane highway is carried through an interchange, provision of medians should be considered for high-speed or high-volume conditions to prevent wrongway right turns.

5.2.2 Sight Distance

Sight distance on the highways through a grade separation should be at least as long as that required for stopping and preferably longer. Where exists are involved, passing sight distance is preferred.

The horizontal sight distance limitations of piers and abutments at curves is a more difficult problem than that of vertical limitations. With curvature of the maximum degree for a given designs peed, the normal lateral clearance at piers and abutments of underpases does not provide the minimum stopping sight distance. Similarly, on overpasses with the sharpest curvature for the designs speed, sight deficiencies result from the usual practical offset to bridge railings. This factor emphasis the need for use of below maximum curvature on highways through interchanges. The minimum design elements of through highways at an interchange area described in 5.2.1, should be used. If there are still some problems in sight distance, the clearances to abutments, piers or rails should be increased as necessary to obtain the proper sight distance even though it is necessary to increase spans or widths.

5.3 INTERCHANGE SPACINGS

In areas of concentrated urban development, proper spacing usually is difficult to attain because of a traffic demand for frequent access. Minimum spacing of arterial interchanges (distance between intersecting stret with ramps) is determined by weaving volumes, ability to sign, signal progression, and required lengths of speed change lanes. A generalised rule of thumb for minimum interchange spacing is 1.5km in urban areas and 3.0 km in rural areas. In urban areas, spacing of less than 1.5km may be developed by grade separated ramps or by adding collector-distributor roads. On urban expressways, connection to the arterial is often made with a single independent ramp rather than with an interchange of full. Where independent ramps from different streets are closely and irregularly arranged on the expressway, the distance between ramp terminal must also be examined.

5.4 UNIFORMITY OF INTERCHANGE PATTERNS

If a series of interchanges is being designed, attention must be given to the group, as well as to each individually, as it is desirable to provide uniformity in exit and entrance patterns. Because interchanges are closely spaced in urban areas, shorter distances are available in which to inform drivers of the course to be followed in leaving an expressway. A dissimilar arrangement of exits between successive interchanges, such as an irregular sequence of the near and far side of structure exit ramp locations or some right off movements causes confusion resultig in slowing down on high-speed lanes and unexpected maneouvers.

The difficulty of right entrance merging with high-speed through traffic and the requisite lane changing for right exit ramps make these layouts undesirable. Except in highly special cases all entrance and exit ramps should be on the left.

Figure 5-2 give example of consistent and inconsistent pattern of exits.

5.5 ROUTE CONTINUITY

Route continuity refers to the provision of a directional path along and throughout the length of a designated route. The destiation pertains to a route number or a name of a major highway.

Route continuity is an extension of the principle of operational uniformity couple with the application of proper lane balance and the principle of maintaining basic number of lanes.

The principle of route continuity simplifies the driving task in that it reduces lane changes, simplifies signing, delineates the throughroute, and reduces the driver's search for directional signing. Location and design of interchanges, individually and as a group, should be tested for proper signing. Signs used should conform to the Manual On Traffic Control Devices - Arahan Teknik (Jalan) 2A and 2B/85.

Pavement markings, delineators and other markings also are important parts of driver communication at

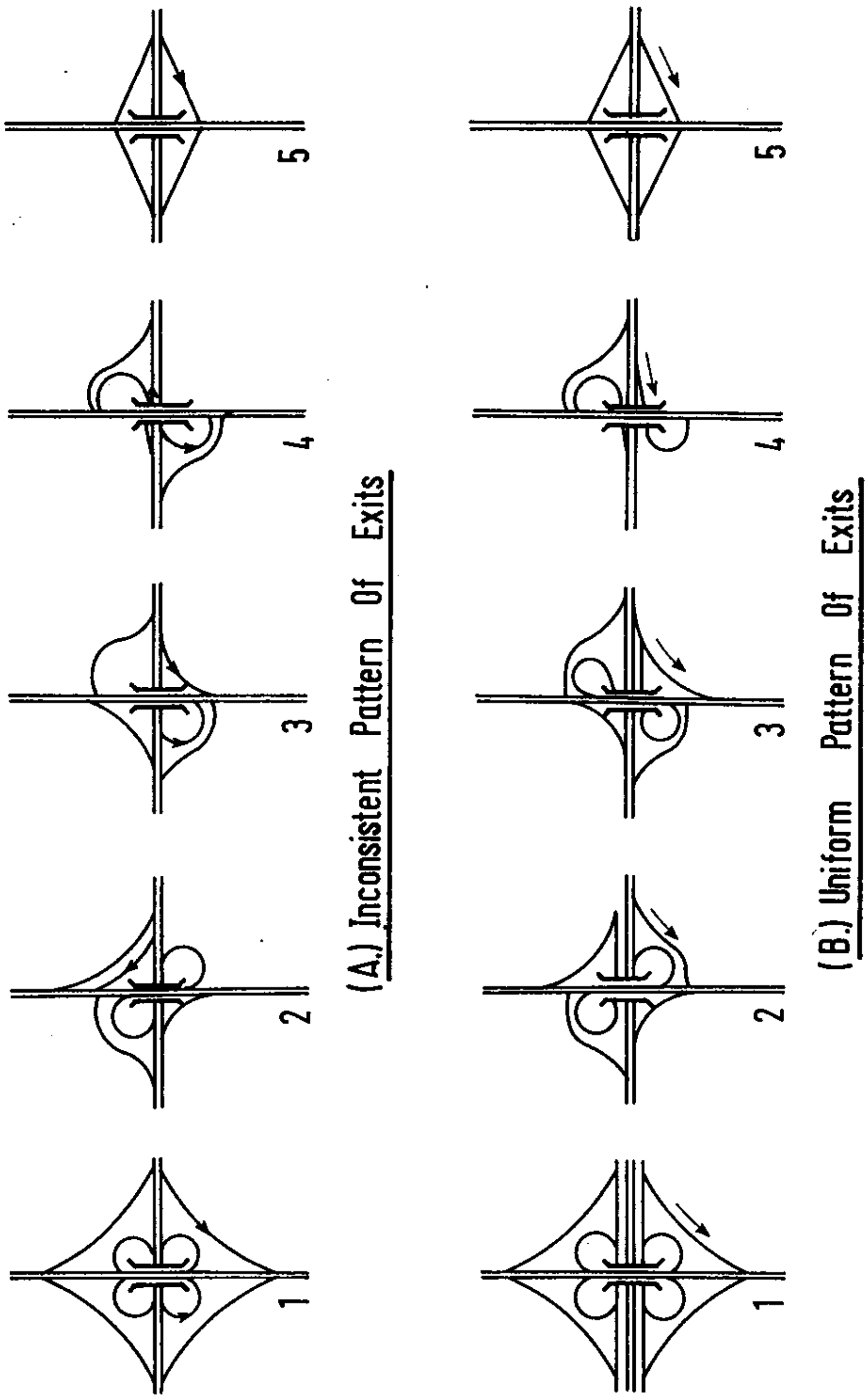

1
2
3
4
5
(A.) Inconsistent Pattern Of Exits
1
2
3
4
5
(B.) Uniform Pattern Of Exits

FIGURE 5-2

interchanges. These should be uniform and consistent following the standards and guidelines as in Arahan Teknik (Jalan) 2D/85 as above.

Figure 5-3 illustrates the principle of route continuity as applied to a hypothetical route as it intersects other major high volume routes.

5.6 SIGNING AND MARKINGS

Ease of operation at interchanges, that is, clarity of paths to be followed, safety, and efficiency, depends largely on their relative spacing, the geometric layout, and effective signing. The location of minimum distances on whether or not effective signing can be provided to inform, warn and control drivers. Location and design of interchanges, individually and as a group, should be tested for proper signing. Signs used should conform to the Manual On Traffic Control Devices - Arahan Teknik (Jalan) 2A and 2B/85.

Pavement markings, delineators and other markings also are important parts of driver communication at interchanges. These should be uniform and consistent following the standards and guidelines as in Arahan Teknik (Jalan) 2D/85 as above.

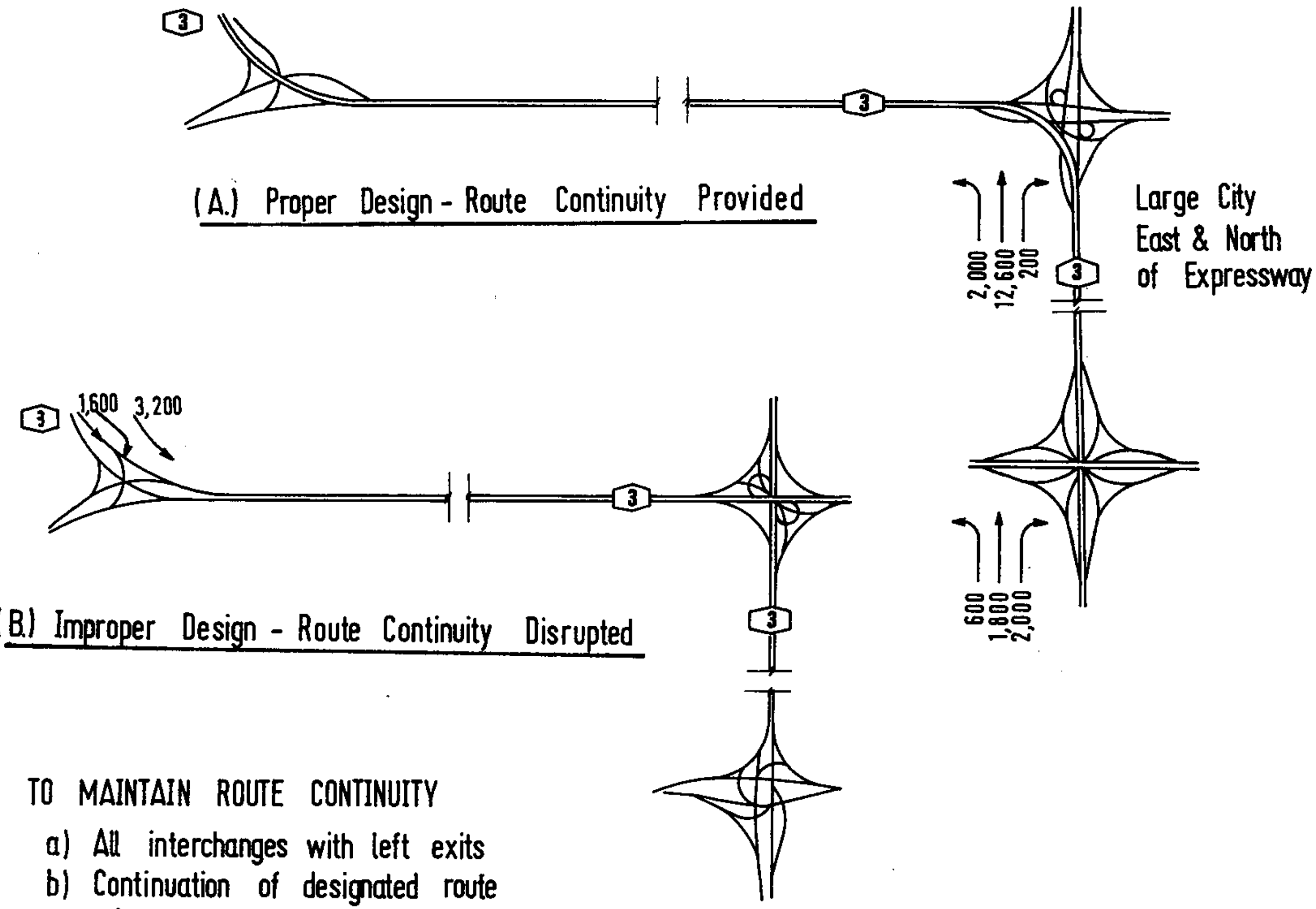
5.7 BASIC NUMBER OF LANES

Any route of arterial character should maintain a certain consistency in the number of lanes provided along it. The basic number of lanes is defined as a minimum number of lanes designated and maintained over a significant length of a route, irrespective of changes in traffic volume and requirements for a lane balance.

The number of lanes is predicted based on the general volume level of traffic over a substantial length of the facility. The volume considered here is the DHV (normally, representative of the morning or evening weekday peak).

An increase in the basic number of lanes if required where traffic builds up sufficiently to justify an extra lane and where such buildup raises the volume level over a substantial length of the following facility.

The basic number of lanes may be decreased where traffic is reduced sufficiently to drop a basic lane, provided there is a general lowering of the volume level on the expressway route as a whole.



(A.) Proper Design - Route Continuity Provided

(B.) Improper Design - Route Continuity Disrupted

TO MAINTAIN ROUTE CONTINUITY

a) All interchanges with left exits
b) Continuation of designated route takes precedence over larger volume junctions.

FIGURE 5-3

5.8 COORDINATION OF LANE BALANCE/BASIC NUMBER OF LANES

To realise an efficient trafic operation through and beyond an interchange, there should be a balance in the number of traffic lanes on the expressway and ramps.

After the basic number of lanes is determined for each roadway, the balance in the number of lanes should be checked on the basis of the following principles:

(a) At entrances the number of lanes beyond the merging of two traffic streams should be not less than the sum of all traffic lanes on the merging roadways minus one.

(b) At exits the number of approach lanes on the highway must be equal to the number of lanes on the highway beyond the exit plus the number of lanes on the exit, less one. An exception to this principle would be at cloverleaf loop ramp exits which follow a loop ramp entrance or at exits between closely spaced interchanges: i.e. a continuous auxiliary lane between the terminals is being used. In these cases, the auxiliary lane may be dropped in a single-lane may be dropped in a single-lane exit.

(c) The travelled way of the highway should be reduced by not more than one traffic lane at a time.

Figure 5-4 illustrates the application of the principles of lane balance.

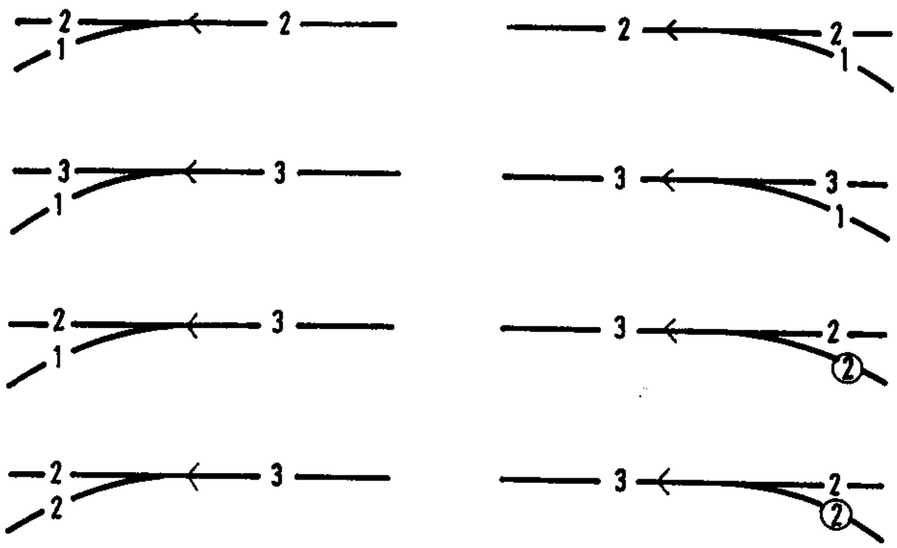
However, the principles of lane balance may conflict with the concept of continuity in the base number of lanes.

Figure 5-5 shows three different arrangements which illustrates this conflict :-

(a) In arrangement A, lane balance is maintained, but there is no compliance to the basic number of lanes. This pattern may cause confusion and erractic operaitons for through traffic.

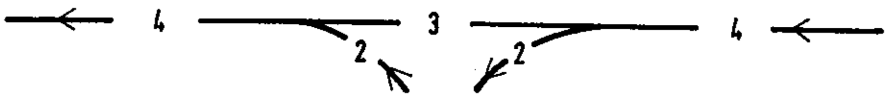
(b) In arrangement B, continuity in the basic number of lanes is provided but the pattern does not coform to the principles of lane balance. With this pattern, the large existing and entering traffic volume requiring two lanes would have difficulty in diverging or merging with the main line flow.

DIVERGING **MERGING**

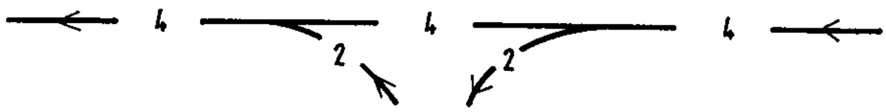


Examples of Lane Balance

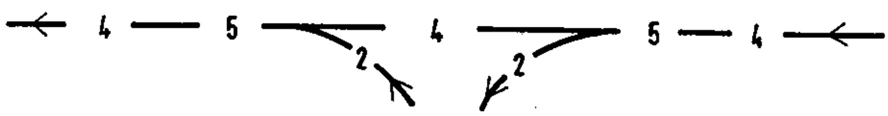
FIGURE 5-4



(A) Lane Balance but no Compliance with Basic No. of Lanes



(B) No Lane Balance but compliance with Basic No. of Lanes



(C) Compliance with Both Lane Balance and Basic No. of Lanes

COORDINATION OF LANE BALANCE AND BASIC No. OF LANES

FIGURE 5-5

(c) Arrangement C is a pattern where the concepts of lane balance and basic number of lanes are brought into harmony by means of building on the basic number of lanes, that is, by adding auxiliary lanes or removing them from the basic width of the travelled way. Auxiliary lanes may be added to satisfy capacity and weaving requirements between interchanges, to accomodate traffic pattern variations at interchagnes, and for simplification of operations (as reducing lane changing). The principles of lanes balance must always be applied in the use of auxiliary lanes.

5.9 AUXILIARY LANES

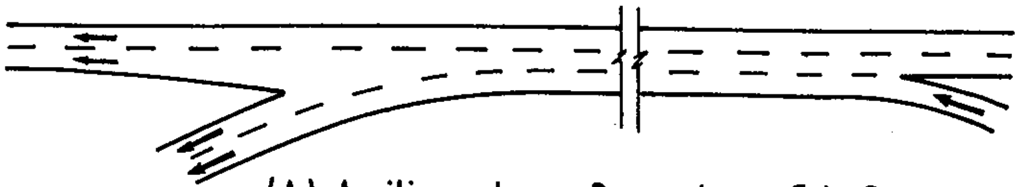
An auxiliary lane is defined as the portion of the roadway adjoining the travelled way for parking, speed change, storage for turning, weaving, truck climbing, and other purposes supplementary to through traffic movement. The width of an auxiliary lane should equal that of the through lanes. Adding or removing of auxiliary lanes is not counted as the change in basic number of lanes.

The usage of auxiliary lanes should follow the general principle outlines below:-

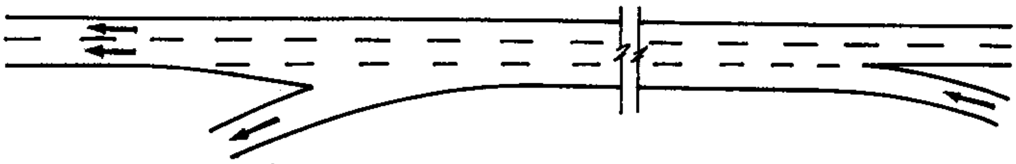
(a) Where interchanges are closely spaced (i.e. distance between the end of taper on the entrance terminal and the begining of taper on the exit terminal should be used to improve operational efficiency.

(b) The termination of the auxiliary lane may be accomplished by several methods. They may be dropped in a two-lane exit; a single lane exit or at the physical nose before tapering into the through roadway. For these methods, the exit gore should be visible throughout the length of the auxiliary lane. Figure 5-6 (A, B, C) illustrates these methods.

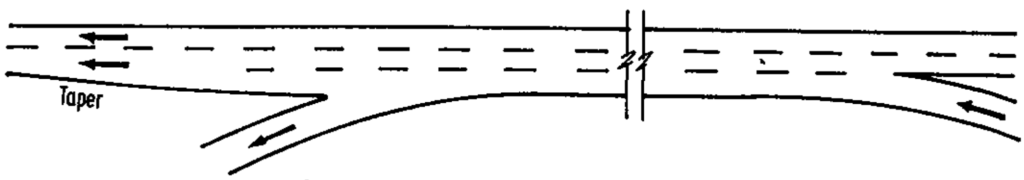
(c) Where local experience with the single exit design indicates problems with turbulence in the traffic flow caused by vehicles attempting to recover and proceed on the through lanes, the recovery lane shold be extended 150 to 300m before being tapered into the through lanes. Within large interchanges, this distance should be increased to 450m. When auxiliary lane is carried through one or more interchanges, it may be dropped as indicated above or it may be merged into the through roadway approximately



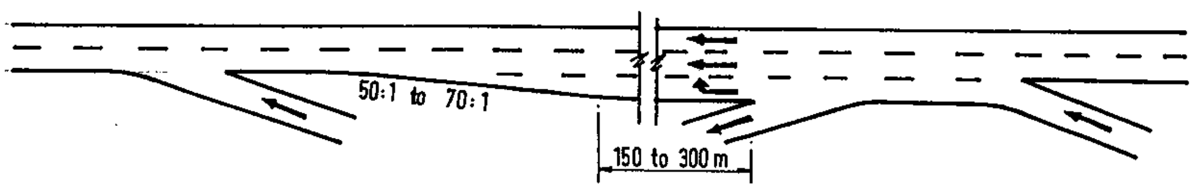
(A.) Auxiliary Lane Dropped on Exit Ramp



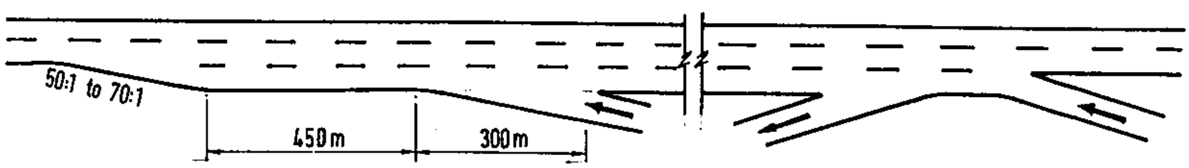
(B.) Auxiliary Lane Between Cloverleaf Loops or Closely Spaced Interchanges Dropped on Single Exit Lane.



(C) Auxiliary Lane Dropped at Physical Nose



(D.) Auxiliary Lane Dropped within an Interchange



(E.) Auxiliary Lane Dropped Beyond an Interchange

FIGURE 5-6: ALTERNATIVES IN DROPPING AUXILIARY LANES

750m beyod the influence of the last interchange. Figure 5-6 (D, E) illustrates the two methods above.

(d) When interchanges are widely spaced, the auxiliary lane originating at a two-lane entrance should be carried along the expressway for an effective beyong the mering point as shown in Figure 5-7 (A,B). An auxiliary lane introduced for a two-lane exit should be carried along the expressway for an effective distance in advance of the exit and extended onto the ramp as shown in Figure 5-7 (C,D).

(e) Lane taper lengths should not be less than thoseset up for single-lane ramps with adjustments for grades. A minimum distance of about 760m is required to produce the necessary operational effect to develop the full capacity of two lane entrances and exits.

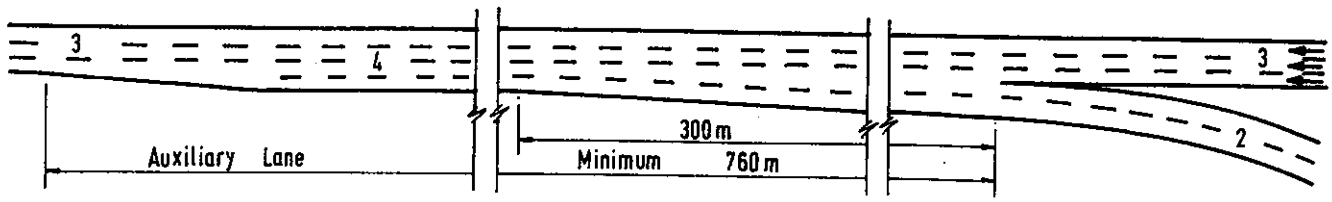
5.10 LANE REDUCTION

A reduction in the basic number of lanes may be effected beyond a principal interchange involving a major fork or a point downstream from the interchange with another expressway. This reduction may be made provided the exit volume is sufficiently large to change the basic number of lanes beyond this point on the expressway route as a whole. Another case where the basic number of lanes may be reduced is where a series of exits,as in outlaying areas of a city, causes the traffic load on the expressway to drop sufficiently to justify the lesser basic number of lanes. The lane reduction of a basic lane or an auxiliary lane may be effected at a two lane exit ramp or between interchanges.

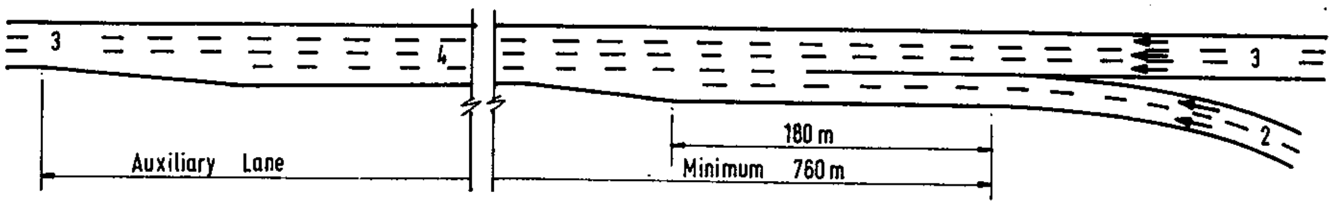
If a basic lane or auxiliary lane is to be dropped between interchanges, it must be accomplished at a distance of 600 to 900m from the previous interchange to allow for adequate signing.

The lane drop transition should be effected on a tangent horizontal alignment and on the approach side of any crest vertical curve. A sag vertical curve is also a good location for a lane drop because it provides good visibility. A left side lane reduction has advantages in that speeds are generally lower and the merging manoeuver from the left is more familiar to most motorists.

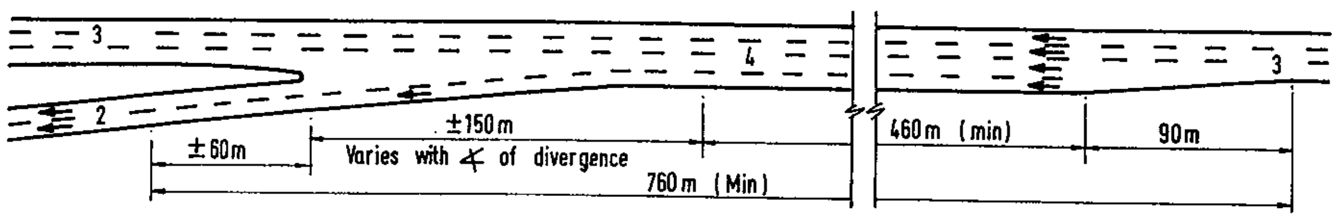
The end of the lane drop should be tapered into the highway in a manner similar to that at a ramp entrance. The minimum taper rate should be 50:1 and 70:1 is desirable.



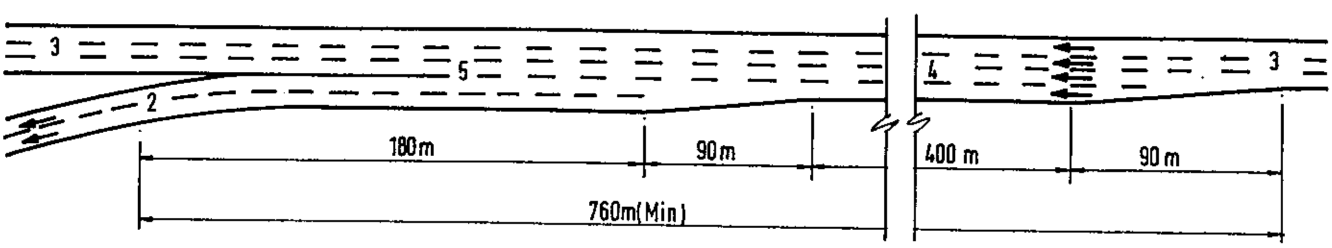
(A.) Tapered Design



(B.) Parallel Design



(C.) Tapered Design



(D.) Parallel Design

FIGURE 5-7 : APPLICATION OF AUXILIARY LANES AFTER EXITS AND IN ADVANCE OF EXITS

CHAPTER 6. DESIGN ELEMENTS

6.1 WEAVING SECTIONS

6.1.1 General

(a) Weaving sections are highway segments where the patterns of traffic entering and leaving at various points of access results in vehicle paths crossing each other. They occur where one-way traffic streams cross by merging and diverging maneouvers, and within interchanges are usually between entrance ramps followed by exit ramps of successive interchanges and on segments of overlapping roadways.

Weaving sections may be considered as simple or multiple. A multiple-weaving section consists of two or more overlapping weaving sections. Multiple weaving section occur frequently in urban areas where there is need for collection and distribution of high concentrations of traffic.

(b) Because considerable turbulence occurs throughout weaving sections, interchange designs that eliminate weaving or remove it from the main facility are desirable.

Although interchanges that do not involve weaving operate better than those that do, interchanges with weaving areas are less costly than those without. Designs to avoid weaving movements require a greater number of structures or larger and more complex structures, with some direct connections. Joint evaluation of the total interchange cost and the specific volumes to be handled is required to reach a sound decision between design alternatives.

(c) Where cloverleaf interchanges are used, consideration should be given to the inclusion of collector-distributor roads on the main facility, or possibly both facilities where warranted.

(d) Weaving sections should be designed, checked, and adjusted so that the level of service is consistent with the remaining expressway. The design level of service of a weaving section is dependent on its length, number of lanes, acceptable degree of congestion, and relative volumes of individual movements.

6.1.2 Design Considerations

The weaving section should have a length based on the appropriate level of service of the through expressway. The relationship between the volume of weaving traffic and the length of weaving section required is discussed in Chapter 7.

Where weaving takes place directly on the expressway lanes, the weaving length should not be less than 300 m.

6.2 COLLECTOR-DISTRIBUTOR ROADS

6.2.1 General

The purpose of a collector-distributor road is to eliminate weaving and reduce the number of entrance and exit points on the through roadways while satisfying the demand for access to and from the expressway. They may be provided within a single interchange, through two adjacent interchanges or continuously through several interchanges. Continuous collector-distributor roads are similar to continuous service roads except that access to abutting property is not permitted.

Any continuous collector-distributor roads should be integrated into the basic design to develop an overall system and capacity analysis and basic lane determination should be performed for the overall system rather than for the separate roadways.

Connections between the through roadways and collector-distributor roads are called transfer roads. Transfer roads may be either one lane or two lanes and the principle of lane balance applies to the design of transfer roads on both the through roadways and the collector-distributor roadways.

6.2.2 Design Considerations

(a) Collector-distributor roads may be one or two lanes in width, the determining factor being capacity requirements. Lane balance should be maintained at entrances and exits to and from the through roadways, but strict adherence is not mandatory on the collector-distributor road proper because weaving is handled at a reduced speed. The lane width should be the same as that of through roadways.

(b) The design speed is usually in the range of 60 to 80 km/hr, but should not be less than 20 km/hr below the design speed of the main through roadways. The same design standard as the through roadways, but with a reduced design speed should be applied.

(c) Both transfer roads and collector-distributor roads should have shoulders equal in width to those defined for the design speed. The outer separation between the through roadway and the collector-distributor roads should be as wide as practicable. The minimum width should be great enough for shoulder widths on the collector-distributor roads,and that on the through roadways and for a suitable barrier to prevent indiscriminate crossovers.

(d) Terminals of transfer roads should be designed in accordance with the requirements for ramp terminals.

(e) Operational problems will occur if collector-distributor roads are not properly signed, especially those servicing more than one interchange.

6.3 EXITS

6.3.1 In general, interchanges that are designed with single exits are superior to those with two exits, especially if one of the exits is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp.

The purposes for developing single exits, where applicable, are as follows:

(a) To remove weaving from the main facility and transfer it to a slower speed facility.

(b) To provide a high-speed exit from the main roadway for all exiting traffic.

(c) To simplify signing and the decision process.

(d) To satisfy driver expectancy by placing the exit in advance of the separation structure.

(e) To provide uniformity of exit patterns.

(f) To provide an adequate sight distance for all traffic exiting from the main roadway.

The provision for single exits is more costly because of the added roadway, longer bridges, and in some cases, additional separation structures. However, in urban areas where right of way is available, cost alone should not be the main determinant in the decision to omit or provide single exits where practicable.

The single exit design places the exit from the main line in advance of the structure and is conducive to a uniform pattern of exits. Where the through roadway overpasses the crossroad in a vertical curve, it is practically impossible to develop an adequate sight distance for the loop ramp exit of a conventional cloverleaf interchange. By using the single-exit design, decision sight distance would already be developed owing to the exit occuring on the upgrade.

6.3.2 There are some cases where a single exit does not work as well as two exits. This situation could apply to high-volume, high-speed directional interchanges. The problem usually occurs at the fork following the single exit from the expressway, particularly when the traffic volume is great enough to warrant a two-lane exit and the distance from the exit terminal to the fork is insufficient for weaving and proper signing. There is often some confusion at this second decision point, resulting in poor operation and a high accident potential. Because of this, it may be advantageous on some directional interchanges to provide two exits on each expressway leg.

6.4 RAMPS

6.4.1 Ramp Types

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 area are a terminal at each leg and a connecting road, usually with some curvature, and on a grade.

Figure 6-1 illustrates several types of ramps and their characteristic shapes. Numerous shape variations are in use but each can be classed broadly as one of the types shown. Each ramp generally is a one-way roadway.

(a) Diagonal

Diagonal ramps (Figure 6-1(A)) almost always are one-way but usually have both a left and right-turning movement at the terminal on the minor intersecting road. Although shown as a continuous curve, a diagonal ramp may be largely tangent or wishbone in shape with a reverse curve. Diamond-type interchanges generally have four diagonal ramps.

(b) Loop

The loop may have single turning movements (left or right) or double turning movements (left and right) at either or both ends. Figure 6-1(B) shows the case where only single turns are made at both ends of the ramp. With this loop pattern a right-turning movement is made without an at-grade crossing of the opposing through traffic. The loop usually involves more indirect travel distance than that for other type ramps.

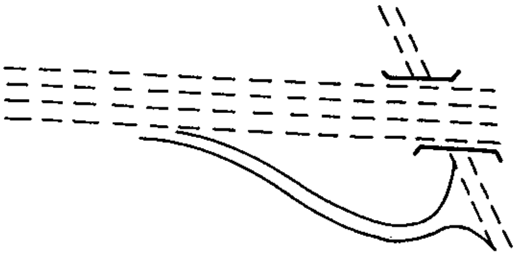
(c) Semidirect

With a semidirect connection (Figure 6-1(C), the driver makes a left turn first, swinging away from the intended direction, gradually reversing, and then completing the movement by following directly around and entering the other road. A descriptive term frequently associated with this type of ramp is "jug-handle", the obvious plan shape. Travel distance on this ramp is less than that for a comparable loop and more than that for a direct connection.

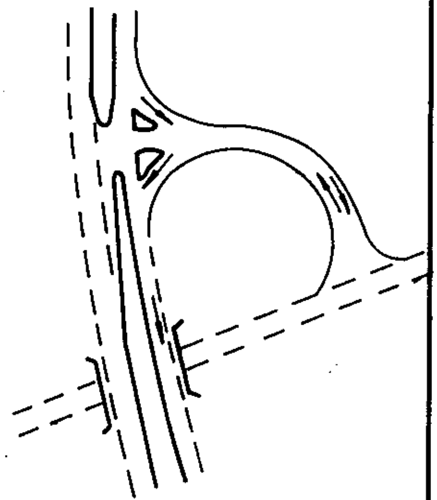
(d) Direct

Travel distance is shortest in a direct connection. However, at least three structures or a three-level structure is needed. Figure 6-1(D) is termed an outer connection, while Figure 6-1(E) is referred to as a direct connection.

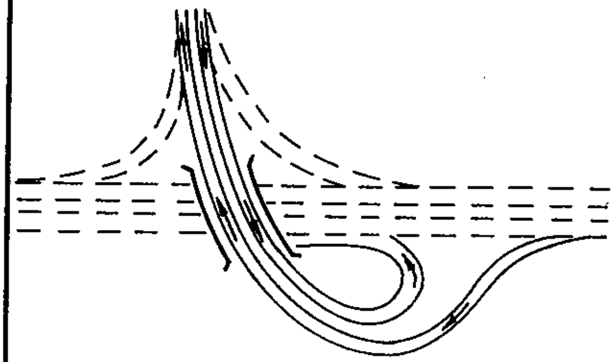
(e) The different ramp patterns of an intersection, i.e. the different types of interchanges, are made up of various combinations of these types of ramps. For example, the trumpet has one loop, one semidirectional ramp, and two right directional or diagonal ramps.



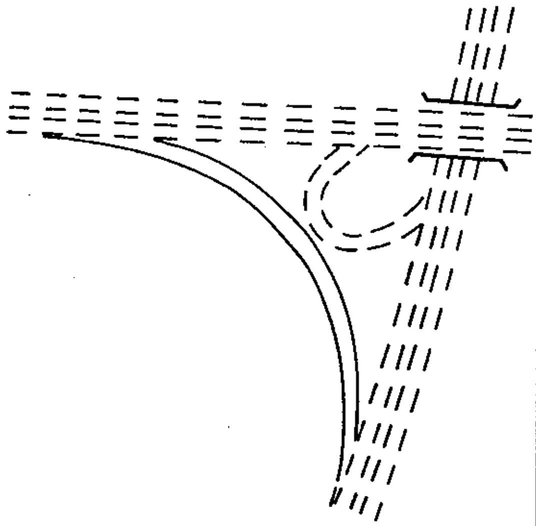
(A) DIAGONAL



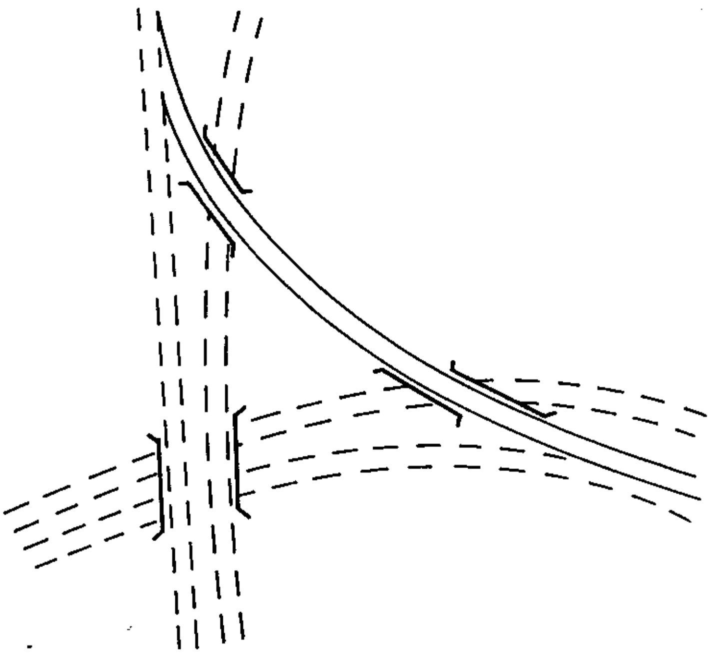
(B) ONE QUADRANT RAMP



(C) LOOP & SEMI-DIRECT



(D) OUTER CONNECTION



(E) DIRECTIONAL

FIGURE 6-1: GENERAL TYPES OF RAMPS

6.4.2 Design Considerations

(a) Design Speeds

Desirably, ramp design speeds should approximate the low-volume running speed on the intersecting highways. This design speed is not always practicable and lower design speeds may be necessary, but they should not be less than half of the design speed for the intersecting highways. Table 6-1 gives guide values for ramp design speeds.

The values in Table 6-1 apply to the sharpest of the controlling ramp curve, usually on the ramp proper. These speeds do not pertain to the ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the highway speed involved.

The highway with the greater design speed should be the control in selecting the design speed for the whole of a ramp. However, the ramp design speed may vary, the portion of the ramp closer to the lower speed highway being designed for the lower speed. The latter point is particularly applicable where the ramp is on an upgrade from the higher speed highway to the lower speed highway.

Ramps for Left Turns - Upper range design speed often is attainable and a value between the upper and lower range usually is practicable.

Loops - Upper-range values of design speed generally are not attainable. Ramp design speeds above 50 km/hr for loops require large areas (rarely available in urban areas) and long loops, which are costly and require right-turning drivers to travel considerable extra distance. Minimum values usually preferably should be not less than 40 km/hr (i.e. 50 m radius).

Semidirect connections - Design speeds between the middle and upper ranges shown in Table 6-1 should be used. A design speed less than 50 km/hr should not be used for highway design speeds of more than 80 km/hr.

Direct connections - Design speeds between the middle and upper ranges shown in Table 6-1 should be used. The minimum preferably should be 70 km/hr and in no case less than 60 km/hr for highway design speeds of more than 80 km/hr.

At-grade terminals - Where a ramp joins a major crossroad forming an intersection at-grade, Table 6-1 is not applicable to that portion of the ramp near the intersection because a stop sign or signal control is normally employed. The requirements as in Arahan Teknik (Jalan) 13/87 "A Guide to the Design of At-grade Intersection" should be followed.

(b) Curvature

The factors and assumptions of minimum-turning roadway curves for various speeds are discussed in Arahan Teknik (Jalan) 8/86 "A Guide to Geometric Design of Roads - (Chapter 4). They apply directly to the design of ramp curves. Compound curves and spiral transitions are desirable to effect the desired shape of ramps; to meet site conditions and other controls, and to fit the natural paths of vehicles. Caution should be exercised in the use of compound curvature to prevent unexpected and abrupt speed adjustments.

(c) Ramp Shapes

The general shape of a ramp evolves from the type of ramp selected, as previously described and shown in Figure 6-1. The specific shape of a ramp may be influenced by such factors as traffic pattern, traffic volume, design speeds, topography, intersection angle, and type of ramp terminal.

Loop and Outer Connection

Several forms may be used for the loop and outer connection of a semi-directional interchange, as shown diagrammatically in Figure 6-2. The loop, except for its terminal, may be a circular arc or some other symmetrical or asymmetrical curve. The asymmetrical arrangement may fit where the intersecting roads are not of the same importance and the ramp terminals are designed for different speeds, the ramp in part functioning as a speed-change area. Similar shapes may be dictated by right-of-way controls, profile and sight distance conditions, and terminal location. The practical radii of loops are approximately 30-45 m for minor movements on highways with design speeds of 80 km/hr or less and from 45-75m for more important movements on highways with higher design speeds. The most desirable alignment for an outer connection is one on a continuous curve (line A). This arrangement, however, may involve extensive right-of-way. Another desirable arrangement has a central tangent and terminal curves (lines B-B and C-C). Where the loop is more important than the outer connection, reverse alignment on the outer connection may be used to reduce the area of right-of-way, as shown by line D-D. Any combination of lines B, C, and D may be used for a feasible shape.

In Figure 6-2(A) the loop and the outer connection are separated, as is generally desirable. However, where the movements are minor and economy is desired, a portion of the two ramps may be combined into a single two-way pavement. Where this design is used, a barrier should separate the traffic in the two directions. This design is, however, discouraged.

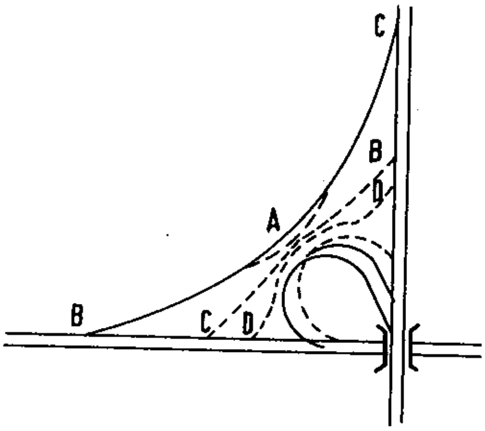
Diagonal Ramps

Diagonal ramps may assume a variety of shapes, depending on the pattern of turning traffic and right-of-way limitations. As shown in Figure 6-2(B), the ramp may be a diagonal tangent with connecting curves (solid line). To favor a left-turning movement the ramp may be on a continuous curve to the left with a spur to the right for right-turns. On restricted right-of-way along the major highway, it may be necessary to use reverse alignment with a portion of the ramp being parallel to the through roadway.

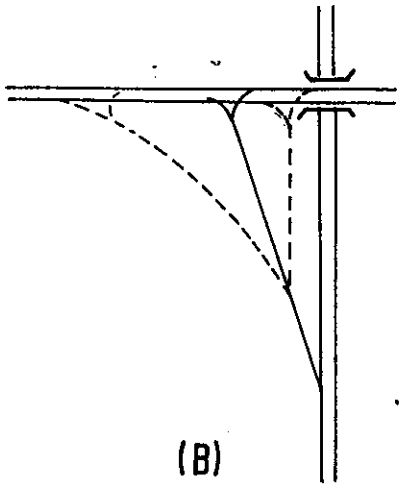
Diagonal ramps of a type usually called slip ramps or more properly termed cross connections connect with a parallel service road, as shown in Figure 6-2(C). Where this design is used, it is desirable to have one-way service roads. Cross connections to two-way service roads introduce the possibility of wrong-way entry onto the through lanes and should be avoided.

Semi-direct Connection

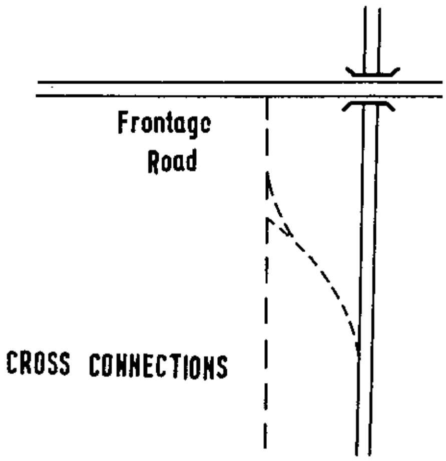
The shape of a semidirect connection (Figure 6-2(D) is influenced by the location of the terminals with respect to the structures, the extent to which the structure pavements are widened, and the curve radii necessary to maintain a desired turning speed for an important left-turning movement. The angular position or the curvature may be dictated somewhat by the relative design speeds of the intersection legs and by the proximity of other roadways.



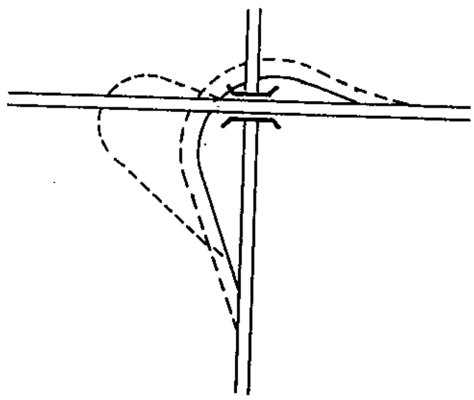
(A)



(B)



(C)



(D)

FIGURE 6-2: RAMP SHAPES

(d) Sight Distance

Sight distance along a ramp should be at least as great as the safe stopping sight distance. Sight distance for passing is not required. There should be a clear view of the whole of the exit terminal, including the exit nose and a section of the ramp pavement beyond the gore.

The sight distance on an expressway preceeding the approach nose of an exit ramp should exceed the minimum for the through traffic design speed, desirably by 25 percent or more. The ranges in design values for stopping sight distance on horizontal and vertical curves for turning roadways and open road conditions is as per Chapter IV of "A Guide on Geometric Design of Roads" - Arahan Teknik(Jalan) 8/86.

(e) Grade and Profile Design

The profile of a typical ramp usually consists of a central portion on an appreciable grade coupled with terminal vertical curves and connections to the profiles of the intersection legs. The following references to ramp gradient pertained largely to the central portion of the ramp profile. Profiles at the terminals largely are determined by the through-road profiles and seldom are tangent grades.

Ramp grades should be as flat as feasible to minimise the driving effort required in maneouvering from one road to another. Most ramps are curved and steep grades on them hamper the flow of traffic. The slowing down of vehicles on an ascending ramp is not as serious as on a through road, provided the speed is not decreased sufficiently to result in a peak-hour backup onto the through road. Most diamond ramps are only 120 to 360 m long, and the short central portion with steepest gradient has only moderate operational effect. Accordingly, gradients on ramps may be steeper than those on the intersecting highways, but a precise relation cannot be set. General values of limiting gradient can be shown, but for any one ramp the gradient to be used is dependent on a number of factors peculiar to that site and quadrant alone. The flatter the gradient on a ramp, the longer it will be, but the effect of gradient on the length of the ramp is less than generally thought.

On one-way ramps a distinction can be made between ascending and descending gradients. 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.

As a general criteria, it is desirable that ascending gradients on ramps should not be steeper than that shown in Table 6-2. Where topographic conditions dictate, grades steeper than desirable may be used. One-way descending gradients on ramps should be held to the same general maximums, but in special cases they may be 2 percent greater.

The cases in which gradient is a determining factor in the length of the ramp are as follows:

(i) for intersection angles of 70^{o} or less it may be necessary to locate the ramp farther from the structure than required for minimum alignment to provide a ramp of sufficient length for reasonable gradient;

(ii) where the intersection legs are on appreciable grade, with the upper road ascending and the lower road descending from the structure, the ramp will have to effect a large difference in elevation that increases with the distance from the structure;

(iii) where a ramp leaves the lower road on a down grade and meets the higher road on a down grade, the longer-than-usual vertical curves at the terminals may make a long ramp necessary to meet gradient limitations. The alignment and grade of a ramp must be determined jointly.

TABLE 6-1: GUIDE VALUES FOR RAMP DESIGN SPEEDS

Highway Design Speed (km/h)	Ramp Design Speed (km/h)		
	upper	middle	lower
120	90 , 80	70	60 (50)
100	80 , 70	60	50
80	70 , 60	50	40
60	50	40	30
50	40	30	25

( ) is applicable only for loops.

TABLE 6-2 : ALLOWABLE MAXIMUM ASCENDING GRADIENT

Ramp Design Speed (km/h)	90	80	70	60	50	40	30	25
Allowable Maximum Ascending Gradient (%)	3	4	4.5	5	5.5	6	7	7.5

TABLE 6-3 : MINIMUM VERTICAL CURVE LENGTHS IN K-VALUE

Ramp Design Speed (km/h)	90	80	70	60	50	40	30	25
Crest Vertical Curve	50	30	20	15	10	8	5	3
Sag Vertical Curve	35	20	15	10	7	5	4	3

TABLE 6-5: RATE OF CHANGE AND PAVEMENT EDGE ELEVATION

Design speed (km/hr)	25 and 30	40	50	60 Or more
Change in relative rate between centerline and pavement edge per station (percent)	0.75	0.71	0.67	0.65

(f) Vertical Curves

Usually, ramp profiles assume the shape of the letter S with a sag vertical curve at the lower end and a crest vertical curve at the upper end. Additional vertical curves may be necessary, particularly on ramps that overpass or underpass other roadways. Where a crest or sag vertical curve extends onto the ramp terminal, the length of curve should be determined by using a design speed between those on the ramp and the highway. Table 6-3 shows the minimum vertical curve lengths for each ramp design speed.

(g) Superelevation and Cross Slope

The following values should be used for cross slope design on ramps:

Superelevation rates, as related to curvature and design speed on ramps, are given in Table 6-4. The highest rate practicable should be used, preferably in the upper half or third of the indicated range particularly on descending ramps.

The cross slope on portions of ramps on tangent normally are sloped one way at a practical rate that may range from 1.5 percent to 2 percent for high-type pavements.

Superelevation runoff, or the change in superelevation rate per unit length of ramp, should not be greater than that in Table 6-5 The superelevation development is started or ended along the auxiliary pavement of the ramp terminal.

Another important control in developing superelevation along the ramp terminal is that of the crossover crown line at the edge of the through-traffic lane. The maximum algebraic difference in cross slope between the auxiliary pavement and the adjacent through lane is shown in Table 6-6.

Three segments of a ramp should be analyzed to determine superelevation rates that would be compatible with the design speed and the configuration of the ramp. The exit terminal, the ramp proper, and the entrance terminal should be studied in combination to ascertain

TABLE 6-4 : SUPERELEVATION RATES FOR CURVES.

Radius (m)	Range In Superelevation Rate For Interchange Ramps With Design Speed (km/h) Of							
	90	80	70	60	50	40	30	25
2,000	.02	.02	.02	.02	.02	.02	.02	.02
1,500	.03	.02	.02	.02	.02	.02	.02	.02
1,200	.04	.03	.02	.02	.02	.02	.02	.02
1,000	.04	.04	.03	.02	.02	.02	.02	.02
800	.05	.04	.04	.02 - .03	.02	.02	.02	.02
700	.06	.05	.04	.02 - .03	.02	.02	.02	.02
600	.07	.06	.05	.03 - .04	.02 - 03	.02	.02	.02
500	.08	.07	.05	.03 - .04	.02 - .03	.02	.02	.02
400	.09	.08	.06	.04 - .05	.03 - .04	.02 - .03	.02	.02
350	.10	.09	.07	.05 - .06	.03 - .04	.02 - .03	.02	.02
350	Rmin. = 310	.09	.08	.05 - .06	.03 - .05	.02 - .03	.02	.02
250		.10	.09	.06 - .07	.04 - .06	.02 - .04	.02 - .03	.02
200		Rmin. = 230	.10	.08 - .09	.04 - .07	.03 - .05	.02 - .03	.02
180			.10	.08 - .09	.04 - .07	.03 - .05	.02 - .04	.02 - .03
160			Rmin = 175	.09 - .10	.05 - .08	.03 - .06	.02 - .04	.02 - .03
140				.09 - .10	.05 - .08	.03 - .06	.02 - .04	.02 - .03
130				.09 - .10	.05 - .09	.03 - .07	.02 - .05	.02 - .03
120				Rmin = 125	.05 - .09	.03 - .07	.02 - .05	.02 - .04
110					.06 - .10	.03 - .07	.02 - .05	.02 - .04
100					.06 - .10	.03 - .08	.02 - .06	.02 - .04
90					.06 - .10	.03 - .08	.02 - .06	.02 - .05
80					Rmin = 85	.04 - .09	.02 - .07	.02 - .05
70						.04 - .09	.02 - .07	.02 - .06
60						.04 - .10	.02 - .08	.02 - .06
50						.04 - .10	.02 - .09	.02 - .07
40						Rmin = 50	.02 .10	.02 - .08
30							.02 = .10	.02 - .09
20							Rmin = 30	.02 - .10
								Rmin = 20

the design speed and superelevation rate. Three ramp configurations are considered for this discussion.

Diamond Ramp

Deceleration to controlling curve speed should occur on the auxiliary lane of the exit, and continued deceleration to stop or yield conditions would occur on the ramp proper. Superelevation comparable to open road conditions would not be appropriate on the ramp proper or the forward terminal.

Loop Ramp

The curvature of the ramp proper could be a simple curve or combination of curves and transitions. The design speed and superelevation rate would be determined by the curvature of the ramp proper. Superelevation would have to be gradually developed into and out of the curve for the ramp proper.

Direct and Semi-direct Ramps

Superelevation rates comparable to open-road conditions are appropriate for high-speed direct and semidirect ramps.

Developing Superelevation

The method of developing superelevation at free-flow ramp terminals is demonstrated diagrammatically in Figure 6-3.

Figure 6-3(A) shows a tapered exit from a tangent section with the first ramp curve falling beyond the required deceleration length. The normal crown is projected onto the auxiliary pavement, and no superelevation is required until the first ramp proper curve is reached.

Figure 6-3(B) shows a parallel-type exit from a tangent section that leads into a flat exiting curve. At point b,the normal crown of the through roadway is projected onto the auxiliary pavement. At point c,the crown line can be gradually changed to start the development of superelevation for the exiting curve. At point d, two breaks in the crossover crown line would be conducive to developing a full superelevation in the vicinity of the physical nose.

Figures 6-3(C) and 6-3(D) show ramp terminals on which the superelevation of the through roadway would be projected onto the auxiliary pavement.

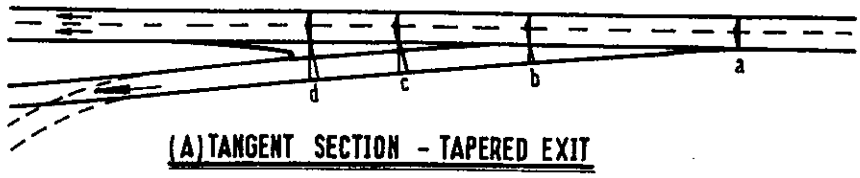
Figures 6-3(E) shows a parallel entrance on the high side of a curve. At point d, the ramp would possibly be flat, and full superelevation would be attained at point c.

Figure 6-3(F) shows a parallel exit from a tangent section with sharp curvature developing in advance of the physical nose. This design is typical for cloverleaf loops. Part of the cross slope change can be accomplished over the length of the parallel lane with about half of the total superelevation being developed at point b. Full superelevation of the ramp proper is reached beyond the physical nose.

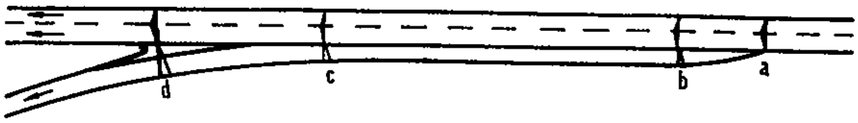
(h) Gores

General

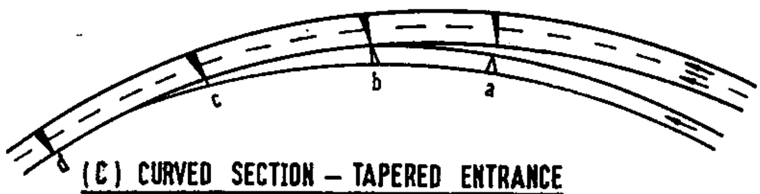
The term "gore" indicates the area immediately beyond the divergence of two roadways, bounded by the edges of those roadways. The approach nose or an ramp nose is an end of an island, or area between diverging roadways, which faces approaching traffic passing to one or both sides. In common use the term gore refers to both (1) the paved triangular area upstream from the approach nose and (2) the graded area for about 30 m downstream from the approach nose. The geometric layout of the first of these is a very important part of exit ramp terminal design. It is the decision point area that must be clearly seen and understood by approaching drivers. The separating ramp roadway not only must be clearly evident but also must be of a geometric shape to fit the likely speeds at that point.



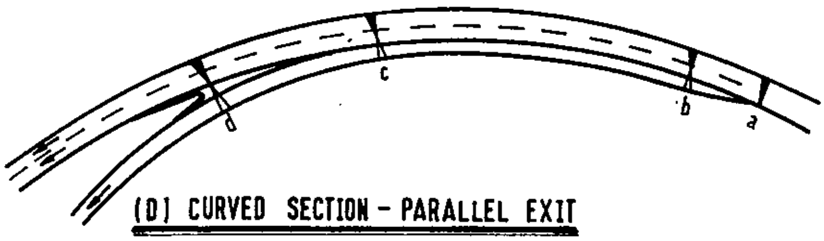
(A) TANGENT SECTION - TAPERED EXIT



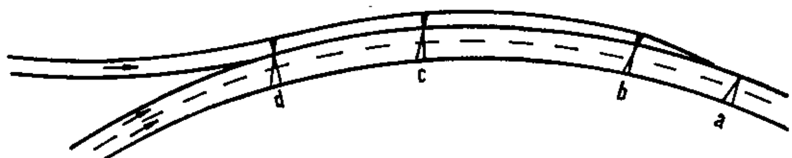
(B) TANGENT SECTION – PARALLEL EXIT



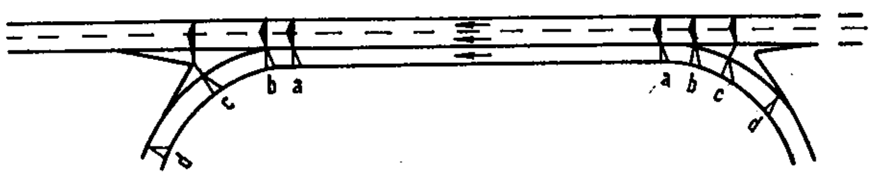
(C) CURVED SECTION – TAPERED ENTRANCE



(D) CURVED SECTION - PARALLEL EXIT



(E) CURVED SECTION PARALLEL ENTRANCE ON HIGH SIDE OF CURVE



(F) TANGENT SECTION - CLOVER LEAF ENTRANCE AND EXIT

NOTE:

PAVEMENT SURFACE ABOVE LEVEL LINE

PAVEMENT SURFACE BELOW LEVEL LINE

FIGURE 6-3

DEVELOPMENT OF SUPEREVELATION AT FREE FLOW RAMP TERMINALS

In a series of interchanges along an expressway the gores should be uniform and have the same appearance to drivers.

The paved triangular area for a left-hand ramp includes the space that is (1) the continuation of the through-lane shoulder which is interrupted by the ramp pavement, (2) the beginning of right-edge shoulder of the ramp pavement, and (3) the elongated triangular area between theoretical edges of the spaces 1 and 2. The whole of this area should be paved to provide an emergency manuoever area that can be traversed safely by drivers who deviate from their proper path. The physical approach nose at the base of the paved area, usually rounded or of limited width, is located with some offset and wedge taper on each side to the separate shoulders downstream from it. The ground-mounted exit sign, should be centered in the gore downstream from the physical approach nose in the typical exit-ramp case.

The width at the base of this triangular paved area, which is at the physical nose, will vary depending on the type of speed change lane, that is, taper-type or parallel-type, and on the alignment of the ramp roadway beyond the gore. The speed of traffic using the highway is also a factor. As a general guide, the width at the physical nose should be between 6 and 9 m including the paved shoulders, measured between the travelled way of the main line and that of the ramp. This dimension should be increased by setting the physical nose further back if the ramp roadway curves away from the expressway immediately beyond the physical nose, particularly so if the speed-change lane is of the parallel type. The dimension should also be increased if running speeds in excess of 100 km/hr are expected to be common.

Markings

The whole of this paved triangular area should be conspicuously striped to delineate the proper paths on each side and to assist the driver in identifying the gore area. This is done by providing chevron markings.

Rumble strips may be placed in the gore area but should not be located too close to the gore nose because such placement renders them ineffectual for warning high-speed vehicles.

Physical nose

The physical nose, which constitutes the beginning of the unpaved area within the gore, is generally rounded to provide wedges of paved areas on either side extending downstream in a manner to complete the recovery area and afford space for a vehicle that has made a wrong move to reenter the traffic stream.

Unpaved Area

The unpaved area beyond the nose should be graded as nearly level with the roadway as is practicable so that vehicles inadvertently entering will not be upset or abruptly stopped by steep slopes. Heavy sign supports, street light standards, and roadway structure supports should be kept well out of the graded gore area. Yielding or breakaway-type supports should be employed for the standard exit sign, and concrete footings, where used, should be kept flush with adjoining ground level.

There will be situations where placement of a major obstruction in a gore is unavoidable. Cushioning or energy dissipating devices for use in front of hazardous fixed objects should be considered and adequate space should be provided for their installation whenever it is found necessary to construct obstructions in a gore on a high speed highway.

Merging End

In an entrance terminal the point of convergence (beginning of all paved area) is defined as the "merging end". In shape, layout, and extent, the triangular maneouver area at an entrance terminal is much like that at an exit. However, it points downstream and separates traffic streams already in lanes, thereby being less of a decision area. The width at the base of the paved triangular area is narrower, usually being limited to the sum of the shoulder widths on the ramp and expressway plus a narrow physical nose 1 to 2 m wide.

Figure 6-4 diagrammatically details typical gore designs for expressway exit ramps. Figures 6-4(A) and (B) depict a full 4 m recovery lane adjacent to the outside through lane and moderate right offset of the ramp pavement.

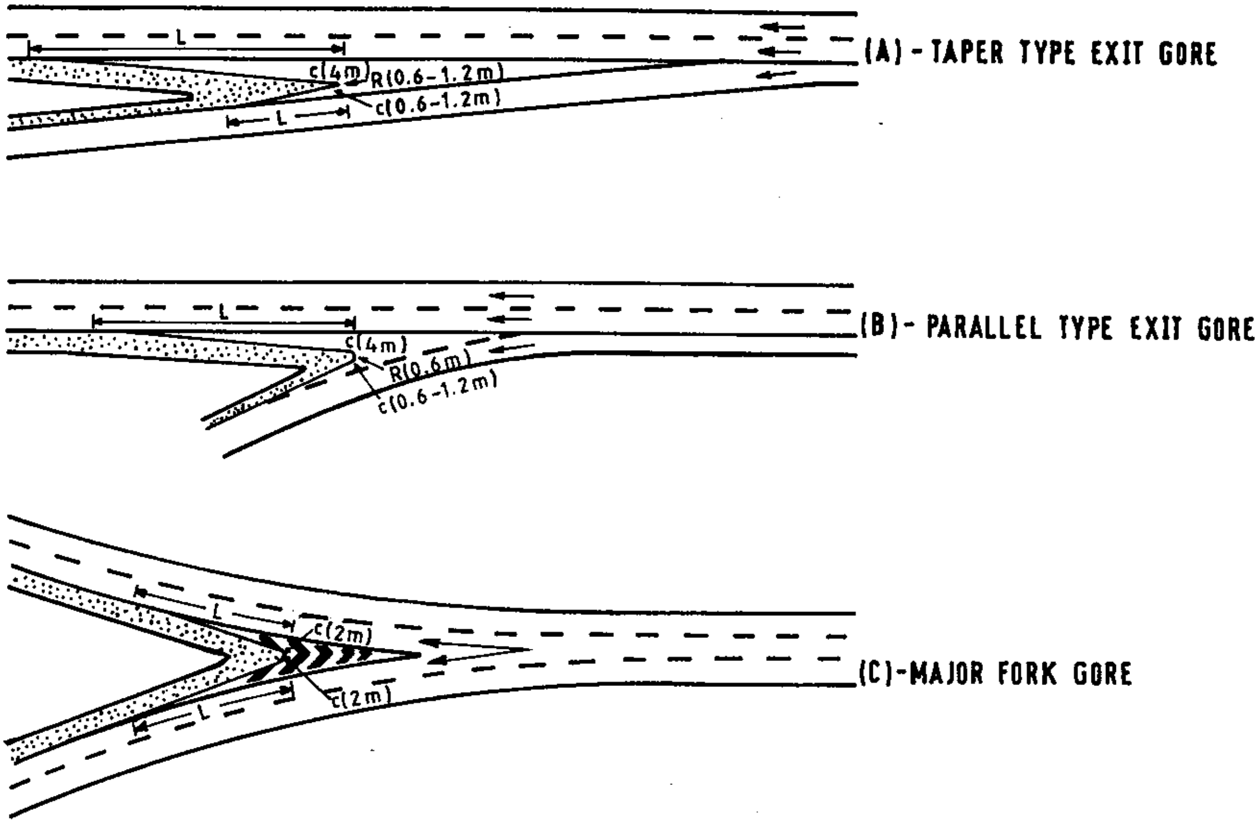
Figures 6-4(C) shows a major fork, with neither diverging roadway having priority. Offset is equal for each roadway,and striping or rumble strips are placed upstream from the physical nose.

Table 6-7 gives minimum lengths for tapers beyond the offset nose.

6.4.3 Pavement Widths

(a) Width and Cross-Section

Ramp pavement widths are governed by the type of operation, curvature, and volume and type of traffic. Design widths of ramp pavements for various design traffic conditions are given in Table 6-8. The three general design traffic conditions are :-



TYPICAL GORE DETAILS

FIGURE 6-4

TABLE 6-6: ALGEBRAIC DIFFERENCE IN CROSS SLOPE.

Design speed of exit or entrance curve. (km/hr.)	Maximum algebraic difference in cross slope crossover. (percent)
25 and 30	5 to 8
40 and 50	5 to 6
60 and Over	4 to 5

TABLE 6-7: MINIMUM LENGTH OF TAPER BEYOND AND OFFSET NOSE.

Design Speed Of Approach Highway (km/hr)	L - Length in metre of nose taper per metre of nose offset (c) as in Figure 6-4
50	5 x C
60	6 x C
80	8 x C
100	9 x C
120	10 x C

TABLE 6-9: TRAFFIC CONDITIONS FOR RAMP DESIGN.

Area	Class of Road	Traffic Condition
Rural	Expressway	C
	Highway	C / B
	Primary	B / A
Urban	Expressway	C
	Arterial	C / B
	Collector	B / A

RADIUS ON INNER EDGE OF PAVEMENT, R (m)	PAVEMENT WIDTH(m)								
	CASE I ONE LANE, ONE WAY OPERATION-NO PROVISION FOR PASSING A STALLED VEHICLE			CASE II ONE LANE, ONE-WAY OPERATION-WITH PROVISION FOR PASSING A STALLED VEHICLE			CASE III TWO LANE OPERATION -EITHER ONE-WAY OR TWO-WAY.		
	DESIGN TRAFFIC CONDITION								
	A	B	C	A	B	C	A	B	C
15	5.5	5.5	7.0	7.0	7.6	8.8	9.5	10.7	12.7
25	5.0	5.2	5.8	6.4	7.0	8.2	8.8	10.0	11.3
30	4.6	5.0	5.5	6.1	6.7	7.6	8.5	9.5	10.7
45	4.3	5.0	5.2	5.8	6.4	7.3	8.2	9.2	10.0
60	4.0	5.0	5.0	5.8	6.4	7.0	8.2	8.8	9.5
90	4.0	4.6	5.0	5.5	6.1	6.7	7.9	8.5	9.2
120	4.0	4.6	5.0	5.3	6.1	6.7	7.9	8.5	8.8
150	3.7	4.6	4.6	5.5	6.1	6.7	7.4	8.5	8.8
TANGENT	3.7	4.6	4.6	5.2	5.8	6.4	7.0	8.2	8.2

NOTE:

A = PREDOMINANTLY P VERHICLES, BUT SOME CONSIDERATION FOR SU TRUCKS.

B = SUFFICIENT SU VEHICLES TO GOVERN DESIGN, BUT SOME CONSIDERATION SEMITRAILER VEHICLE.

C = SUFFICIENT BUS AND COMBINATION-TYPES OF VEHICLES TO GOVERN DESIGN.

TABLE 6-8: RAMP PAVEMENT WIDTHS

Traffic condition A - predominantly P vehicles, but some consideration for SU trucks.

Traffic Condition B - sufficient SU vehicles to govern design, but some consideration for semitrailer vehicles.

Traffic Condition C - Sufficient bus and combination types of vehicles to govern design.

Traffic conditions A, B and C are described in broad terms because design data regarding traffic in volume, or percentage of the total, for each type of vehicle are not available to define these traffic conditions with precision in relation to pavement width. In general, traffic condition A can be assumed as having a small volume of trucks or only occasional large trucks, traffic condition B as having a moderate volume of trucks, say in the range of 5 to 10 percent of the total traffic; and traffic condition C as having more and larger trucks.

Without specific information, traffic condition for the design of an interchange ramps should be taken for the class of road where the interchange is located as shown in Table 6-9.

If the roads connected by the interchange are of different classes the higher class should be used.

Selection of the design case is based on the operation of the ramp as below:-

Case I - Light to moderate traffic volume within the service volume of one lane ramp.

Case II - Moderate to near service volume condition.

<u>Case III</u> - Traffic volume exceeds the service volume of one lane ramp or two way operation.

(b) Shoulders and Lateral Clearances

Shoulders and lateral clearances on the ramps should have the following design standards:-

For one way ramps, a paved shoulder width of 1.5 m is desirable on the left and 0.5 m on the right.

Directional ramps with a design speeds of over 60 km/hr should have a paved left and right shoulder of 2.5 m and 1.0 m respectively.

For expressway ramp terminals where the ramp shoulder is narrower than that on the expressway, the paved shoulder width of the through lane should be carried into the exit terminal, and should begin within the entrance terminal, with the transition to the narrower ramp shoulder effected gracefully on the ramp end of the terminal. Abrupt changes should be avoided.

Ramps should have a lateral clearance on the left outside of the edge of the paved shoulder of at least 1.8 m (preferably 3m) and on the the right a lateral clearance of at least 1.2 m beyond the edge of the paved shoulder.

Where ramps pass under structures, the total width including the paved shoulders should be carried through without change. There should be a lateral clearance of at least 1.2 m beyond the edge of the paved shoulder, to the piers or abutments.

Ramps on overpasses should also have the full approach roadway width and shoulders carried over the structure.

Edge lines and/or some type of colour or texture differentiation between the travelled way and shoulder is desirable.

(c) Median

Ramps of two-way operation should have a median to separate opposing traffic and to provide the space for the installation of some traffic safety devices. The minimum width of median required is:-

(i) 3 m for Rural Areas

(ii) 2 m for Urban Areas

(d) Kerbs

Kerbs should be considered only to facilitate particularly difficult drainage situations as in urban areas where enclosed drainage is required because of restricted right-of-way. It should be located at the edge of the paved shoulders.

In some cases kerbs are used at the ramp terminals but are omitted along the central ramp portions. The use of kerbs on facilities designed for intermediate or higher design speeds is not recommended except in special cases. Mountable kerbs should desirably be placed at the outer edge of the paved shoulder. Barrier kerbs are seldom used in conjunction with shoulders, except where pedestrian protection is required. Because of fewer restrictions and more liberal designs, the need for kerbs in rural areas seldom arises and is strongly discouraged.

6.4.4 Ramp Terminals

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

Terminals are further classified according to the number of lanes on the ramp at the terminal, either single or multilane, and according to the configuration of the speed-change lane, either taper or parallel type.

(a) Right Hand Entrances and Exits

Right-hand entrances and exits are contrary to the concept of driver expectancy when intermixed with left-hand entrances and exits.

Right hand entrances and exits in the design of interchanges should be avoided. Even in the case of major forks and branch connections, the less significant roadway should exit and enter on the left.

(b) Terminal Locations

Where diamond ramps and partial cloverleaf arrangements intersect the crossroad at grade, an at-grade intersection is formed. Desirably, this intersection should be located at an adequate distance from the separation structure to provide sight distances that permit safe entry or exit on the crossroad.

Diamond ramp terminals create two neighboring intersections on the crossroad in a short distance. Turning traffic at these intersections may be significant in urban areas. Queueing lengths of right-turning vehicles must be considered in determining the distance. Coordination of the traffic control at two intersections may also be necessary.

Drivers prefer and expect to exit in advance of the separation structure. The use of collector-distributor roads and single exits on partial cloverleafs and other types of interchanges automatically locates the main line exit in advance of the separation structures, and is encouraged.

Designs that employ exits concealed behind crest vertical curves should be avoided, especially on high-speed facilities.

High-speed entrance ramp terminals should be located on descending grades to aid truck acceleration. Adequate sight distance at entrance terminals should be available so that merging traffic on the ramp can adjust speed to merge into gaps on the main facility.

(c) Ramp Terminal Design

Profiles of highway ramp terminals are to be designed with a platform on the ramp side of the approach nose or merging end. This platform should be a length of about 60 m or more on which the profile does not greatly differ from that of the adjacent through-traffic lane.

A platform area should also be provided at the at-grade terminal of a ramp. The length of this platform should be determined from the type of traffic control involved at the terminal and the capacity requirements.

(d) Traffic Control on Minor Crossroads

On minor highways, intersections formed at the terminals would be designed and operated in the same manner as at-grade intersections. The right-turning movements leaving the crossing highway preferably should have median lanes. For low-volume crossroads,the right-turning movements from the ramps normally should be controlled by stop signs. The left-turning movements from the ramps into multilane crossroads should be provided with an acceleration lane or generous taper or should be controlled by stop or yield signs. Ramps approaching a stop sign should have close to 90° alignment and be nearly level for storage of several vehicles.

Signal controls should be avoided on express-type highways and confined to the minor crossroads on which other intersections are at-grade, and some include signal controls. In or near urban areas, signal control has considerable application at ramp terminals on roads crossing over or under an expressway. Here, the turning movements usually are sizable, and the cost of right-of-way and improvements thereon is high. As a result, appreciable saving may be effected by the use of diamond ramps with high-type terminals on the expressway and signalized terminals on the cross roads.

(e) Distance between terminal and structure

The terminal of a ramp should not be near the grade-separation structure. If it is not possible to place the exit terminal in advance of the structure, the existing terminal on the far side of the structure should be well removed in order that, when leaving, drivers have some distance, after passing the structure, in which to see the turnout and begin the turnoff maneouver. Passing sight distance is recommended.

Ramp terminals on the near side of a grade separation need not be as far removed as those beyond the structure. Both the view of the terminal ahead for drivers approaching on the through road and the view back along the road for drivers on an entrance ramp are not affected by the structure. Where an entrance ramp curve on the near side of the structure requires an acceleration lane, the ramp terminal should be located to provide length for it between the terminal and the structure, or the acceleration lane could be continued through or over the structure.

(f) Distance between Successive Ramp Terminals

On urban expressways there are frequently two or more ramp terminals in close succession along the through lanes. To provide sufficient maneouvering length and adequate space for signing, a reasonable distance is required between terminals.

Spacing between successive outer ramp terminals is dependent on the classification of the interchange involved, the function of the ramp pairs (entrance or exit), and weaving potential.

Figure 6-5 shows the minimum values for spacing of ramp terminals for the various ramp-pair combinations as they are applicable to the interchange classifications.

Where an entrance ramp is followed by an exit ramp, the minimum distance between the successive noses is governed by weaving

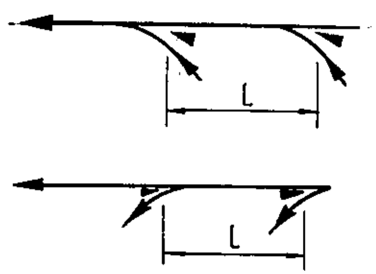
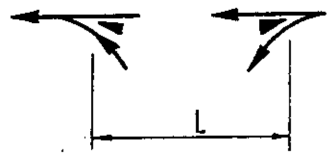
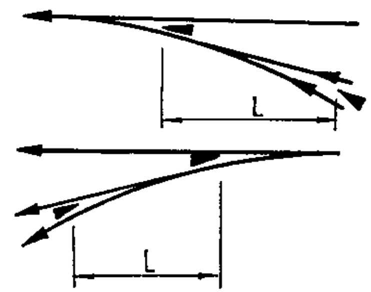
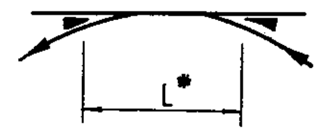
EN – EN OR EX – EX		EX – EN		TURNING ROADWAYS		EN – EX (WEAVING)			
 		 		 		 * NOT APPLICABLE TO CLOVERLEAF LOOP RAMPS			
FULL EXPRESSWAY	C – D ROAD OR DISTRIBUTOR	FULL EXPRESSWAY	C – D ROAD OR DISTRIBUTOR	SYSTEM INTERCHANGE	SERVICE INTERCHANGE	SYSTEM TO SERVICE INTERCHANGE		SERVICE TO SERVICE INTERCHANGE	
						FULL EXP.	C-D Rd OR DIST.	FULL EXP.	C-D Rd OR DIST.
MINIMUM LENGTH MEASURED FROM PHYSICAL NOSE TO PHYSICAL NOSE (m)									
300	250	150	120	240	180	600	480	480	300

FIGURE 6 – 5 : RAMP TERMINAL SPACING

requirements but should not be less than about 500 m to avoid overlapping maneouver areas. A notable exception to this length policy for EN-EX ramp combinations is the distance between loop ramps of cloverleaf interchanges. For these interchanges the distance between EN-EX ramp noses is primarily dependent on loop ramp radii and roadway and median widths. A recovery lane beyond the nose of the loop ramp exit is desirable.

When the distance between the successive EN-EX noses is less than 500 m the speed-change lanes should be connnected to provide an auxiliary lane.

(g) Speed Change Lanes

An auxiliary lane, including tapered areas primarily for the acceleration or deceleration of vehicles entering or leaving the through-traffic lanes, is termed a speed-change lane.

A speed change lane should, as a minimum requirement, have sufficient length to enable a driver to make the necessary change between the speed of operation on the highway and the speed on the turning roadway in a safe and comfortable manner. In the case of an acceleration lane, there should be additional length sufficient to permit adjustments in speeds of both through vehicles and entering vehicles so that the driver of the entering vehicle can position himself opposite a gap in the through-traffic stream and maneouver into it before reaching the end of the acceleration lane.

Speed-change lanes are designed in two general forms, the taper type and the parallel type. The taper type works on the principle of a direct entry or exit at a flat angle; whereas the parallel type has an added lane for speed change. Although either type, when properly designed, will operate satisfactorily, style should be fixed for all entrances and exits on a route. This consistency would help drivers' expectancy. Generally, parallel type for entrance, taper type for exit terminal except on relatively sharp curves to the right, where parallel type may be more suitable, are recommended.

(h) Single-Lane Free Flow Terminal (Entrances)

Taper Type

By relatively minor speed adjustment, the entering driver on a taper type entrance can see and use an available gap in the through-traffic stream. A typical single-lane taper-type entrance terminal is shown in Figure 6-6(A).

The entrance is brought into the expressway with a long, uniform taper. Operational studies show a desirable rate of taper of about 50:1 to 70:1 (longitudinal to lateral) between the outer edge of the acceleration lane and the edge of the through-traffic lane. This rate of convergence provides adequate merging length and also outlines a proper path for an entering vehicle.

The geometrics of the ramp proper should be such that motorists may attain a speed approximately equal to the average running speed of the expressway less 10 km/hr by the time they reach the point where the right edge of the ramp joins the travelled way of the expressway. For consistency of application this point of convergence of the right edge of the ramp and the left edge of the through lane may be assumed to occur where the left edge of the ramp traveled way is 3.5 m from the left edge of the marginal strip of the expressway.

Table 6-10 shows minimum lengths of acceleration distances for entrance terminals and their derivation. Where ramps occur in grades, the lengths should be adjusted in accordance with Table 6-11.

Parallel Type

The parallel-type entrance provides an added lane of sufficient length to enable a vehicle to accelerate to near expressway speed prior to merging. A taper is provided at the end of the added lane. The process of entering the expressway is similar to a lane change to the right under forced conditions.

A typical design of a parallel-type entrance is shown in Figure 6-6(B). A curve with a radius of 300 m or more and a length of at least 60 m should be provided in advance of the added lane. If this curve has a short radius, motorists tend to drive directly onto the expressway without using the acceleration lane. This behavior results in poor merging operations and greatly increases the accident potential.

The taper at the downstream end of a parallel-type acceleration lane should be a suitable length to guide the vehicle gradually onto the through lane of the expressway. A length of approximately 90 m is suitable for design speeds up to 120 km/hr.

The length of a parallel-type acceleration lane is generally measured from the point where the right edge of the travelled way of the ramp joins the travelled way of the expressway to the beginning of the taper. The minimum acceleration lengths for entrance terminals are given in Table 6.10, and the adjustments for grades are given in Table 6-11.

The operational and safety benefits of long acceleration lanes are well recognized, particularly where both the expressway and ramp carry high traffic volumes. An acceleration lane length of at least 360 m, plus the taper, is desirable wherever it is anticipated that the ramp and expressway will frequently carry traffic volumes approximately equal to the design capacity of the merging area.

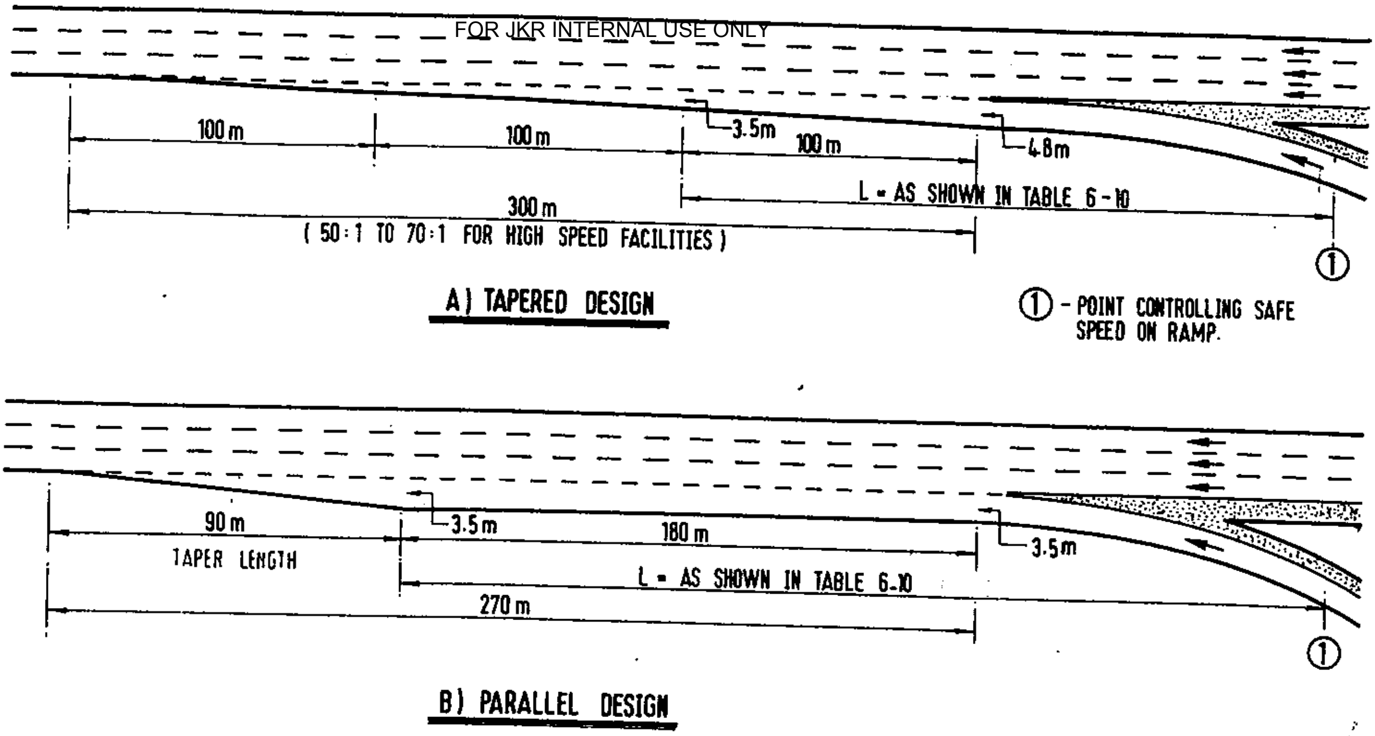


FIGURE 6-6 : SINGLE LANE ENTRANCE RAMPS

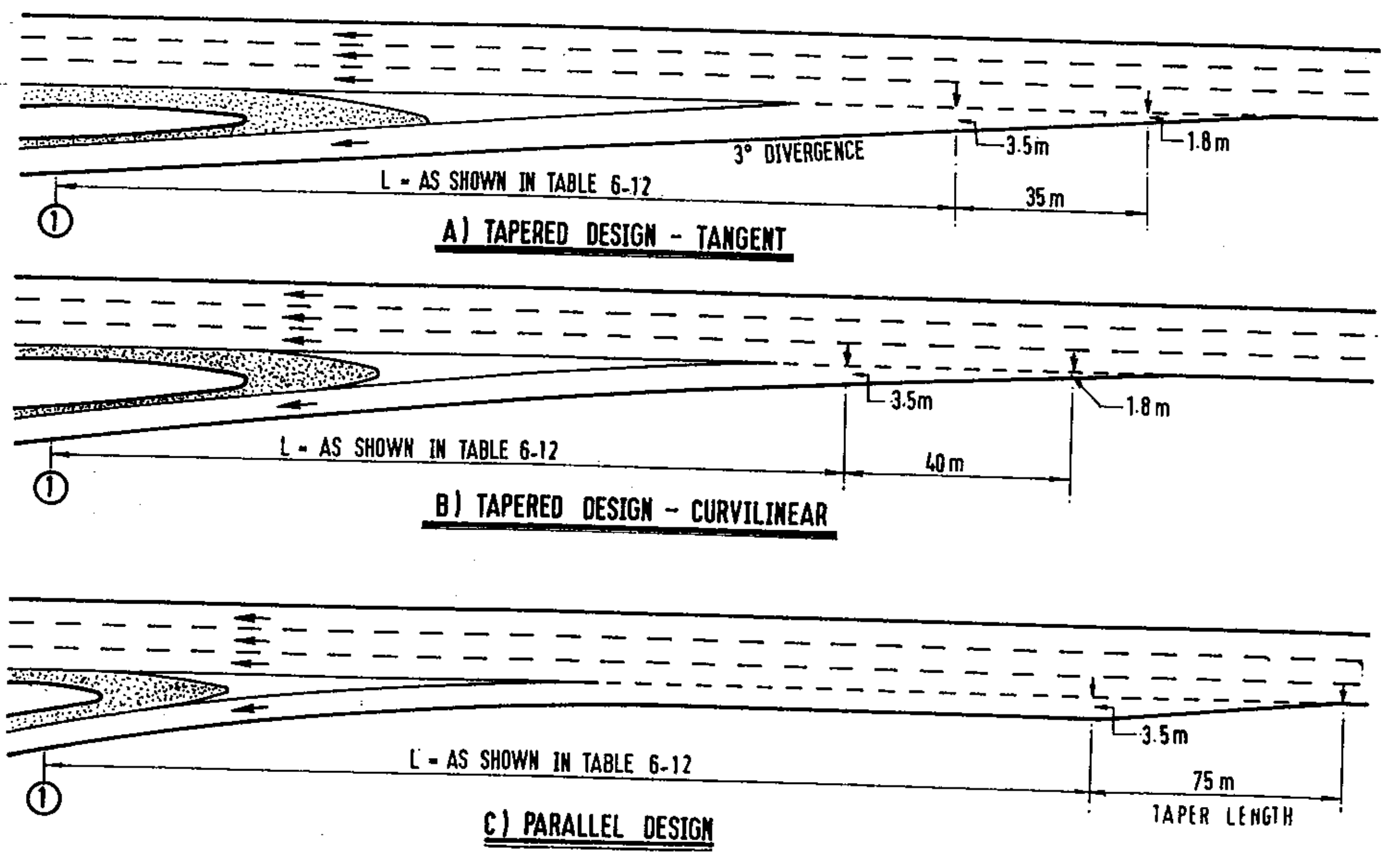
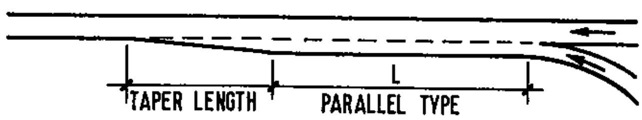


FIGURE 6-7 : SINGLE LANE EXIT RAMPS

Highway Design Speed (km/hr)	Acceleration Length, L (m) For Entrance Curve Design Speed. (km/hr)								
	Stop Condition	25	30	40	50	60	70	80	90
50	60	–	–	–	–	–	–	–	–
60	115	100	75	65	40	–	–	–	–
80	235	215	190	180	150	115	50	–	–
100	360	325	330	300	275	245	180	120	50
120	485	470	460	430	405	375	310	250	180

TAPER LENGTH OF
PARALLEL TYPE ACCELERATION LANE.

Highway Design Speed (km/hr)	Taper Length (m)
50	50
60	60
80	70
100	80
120	90



NOTE:

UNIFORM 50:1 TO 70:1 TAPERS ARE RECOMMENDED WHERE LENGTHS OF ACCELERATION LANE, EXCEED 400m OR ELSEWHERE IF APPROPRIATE AND SPACE PERMITS.

MINIMUM ACCELERATION LENGTHS FOR
ENTRANCE TERMINALS (GRADES $\leq$ 2%)

TABLE 6 - 10

(i) Single-Lane Free Flow Terminals (Exits)

Taper Type

The taper-type exit fits well the direct path preferred by most drivers, permitting them to follow an easy path within the diverging area. The taper-type exit terminal beginning with an outer edge alignment break usually provides a clear indication of the point of departure from the through lane and in general has been found to operate smoothly on high-volume expressway. The divergence rate of taper should be between 50:1 and 70:1.

Figure 6-7 (A) shows a typical design for a taper-type exit.

The length available for deceleration may be assumed to extend from a point where the left edge of the tapered wedge is about 3-5 m from the left edge of the marginal strip, to the point controlling the safe speed for the ramp. Deceleration may end in a complete stop, as at a crossroad terminal for a diamond interchange, or the critical speed may be governed by the curvature of the ramp roadway. Minimum deceleration lengths for various combinations of design speeds for the highway and for the ramp roadway are given in Table 6-12. Grade adjustments are given in Table 6-11.

Parallel Type

A parallel-type exit terminal usually begins with a taper, followed by a derived length of added full lane. A typical parallel-type exit terminal is shown in Figure 6-7 (C). This type of terminal provides an inviting exit area, because the foreshortened view of the taper and the added width are very apparent. However, this design assumes that drivers will exit near the beginning of the added lane, and effect speed change thereafter. It requires a reverse-curve maneouver that is somewhat unnatural. Under low-volume conditions a driver may choose to avoid the reverse-curve exit path and turn directly off the through lane in the vicinity of the exit nose. Such a maneouver may result in undesirable deceleration of the through lane, in

Deceleration Lanes

Design Speed of Highway (km/hr)	Ratio of Length on Grade to Length on Level for[a]	
All speeds	3 to 4 percent upgrade 0.9	3 to 4 pecent downgrade 1.2
All speeds	5 to 6 percent upgrade 0.8	5 to 6 percent downgrade 1.35

Acceleration Lanes

Design Speed of Highway (km/hr)	Ratio of Length of Grade to Length of Level for[a] Design Speed of Turning Roadway Curve (km/hr) 30	50	60	80	All Speeds
	3 to 4 percent upgrade				3 to 4 percent downgrade
60	1.3	1.3	–	–	0.7
80	1.3	1.4	1.4	–	0.65
100	1.3	1.5	1.5	1.6	0.6
120	1.5	1.6	1.7	1.8	0.6
	5 percent upgrade				5 to 6 percent downgrade
60	1.5	1.5	–	–	0.6
80	1.5	1.7	1.9	–	0.55
100	1.7	1.9	2.2	2.5	0.5
120	2.1	2.3	2.6	3.1	0.5

[a]Ratio from this table multiplies 2 by length in Table 6-10 or Table 6-12 gives length of speed change lane on grade

Taper lengths need not be corrected for grade.

RATIO OF LENGTH OF SPEED CHANGE LANES ON GRADE TO LENGTH LEVEL ACCELERATION OR DECELERATION LANE

TABLE 6-11

undesirable conflict on the deceleration lane, or in excessive speed in the exit-nose area.

The length of a parallel-type lane is usually measured from the point where the added lane attains a 3.5 m width to the point where the alignment of the ramp roadway departs from the alignment of the expressway. Where the ramp proper is curved, it is desirable to provide a transition at the end of the deceleration lane. A compound curve may be used with the initial curve desirably having a long radius of, 300 m or more. A transition or a long radius curve is also desirable if the deceleration lane connects with a relatively straight ramp. In such cases a portion of the ramp may be considered as a part of the deceleration length, thus shortening somewhat the required length of continuous parallel lane. Minimum lengths are given in Table 6-12 and adjustments for grades in Table 6-11.

Longer parallel-type deceleration lanes are more likely to be used properly. Lengths of at least 240 m are desirable.

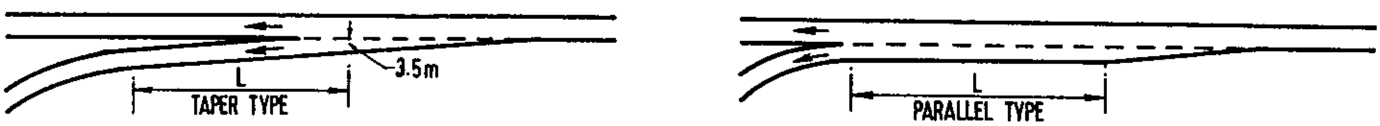
The taper portion of a parallel-type deceleration lane desirably should have a taper of 15:1 to 25:1.

(j) Free Flow Terminals on Curves

Where the curves on a expressway are relatively sharp and it is necessary to provide exits and entrances on these curves, some adjustments in design may be desirable to avoid operational difficulties.

On expressway having design speeds of 100 km/hr or more the curves are sufficiently gentle so that either the parallel type or the taper type of speed-change lane is suitable. With the parallel type the design is about the same as that on tangent and the added lane is usually on the same curvature as the main line. With the taper type the dimensions applicable to terminals located on tangent alignment are suitable for use on curves. The ramp is tapered at the same rate relative to the through-traffic lanes on the curved section as the tangent section.

Highway Design Speed (km / hr)	Deceleration Length, L (m) for Exit Curve Design Speed (km / hr)								
	Stop Condition	25	30	40	50	60	70	80	90
50	70	55	50	45	–	–	–	–	–
60	95	90	80	70	60	50	–	–	–
80	130	125	120	110	95	85	70	55	–
100	160	155	150	140	130	125	105	90	75
120	190	180	175	170	155	150	130	120	105



MINIMUM DECELERATION LENGTHS FOR EXIT TERMINALS (GRADES $\leq$2%)

TABLE 6-12

Wherever a part of a tapered speed-change lane falls on a curved alignment, it is desirable that the entire length be within the limits of the curve. Where the taper is introduced on tangent alignments just upstream from the beginning of the curve, the outer edge of the taper will appear as a kink at the point of curvature.

At ramp terminals on relatively sharp curves such as those that may occur on expressways having a design speed of 80 km/hr, the parallel type of speed-change lanes has an advantage over the taper type. At exits the parallel type is less likely to confuse through traffic, and at entrances this type will usually result in smoother merging operations.

Parallel-type speed-change lanes at ramp terminals on curves are shown diagrammatically in Figure 6-8.

Entrances on curved sections of on expressway are generally less of a problem than exits. Figures 6-8 (A)and 6-8 (B) show entrances with the expressway curving to the right and left, respectively. It is important that the approach curve on the ramp has a very long radius as it joins the acceleration lane. This aligns the entering vehicle with the acceleration lane and lessens the chances of motorists entering directly onto the through lanes. The taper at the end of the acceleration lane should be long, preferably about 90 m. When reverse curve alignment occurs between the ramp and speed change lane, an intervening tangent should be used to aid in superelevation transition.

An exit may be particularly troublesome where the expressway curves to the right (Figure 6-8 C) because traffic on the outside lane tends to follow the ramp. Exits on right-turning curves should be avoided, if possible. Caution must be used in positioning a taper-type deceleration lane on the outside of a right-turning main line curve. The design should provide a definite break in the left edge of pavement to provide a visual cue to the through driver so that he is not inadvertently led off the through roadway. In order to make the deceleration lane more apparent to approaching motorists, the taper should be shorter, preferably no more than 30 m in length. The deceleration lane should begin either upstream or downstream from the point of curve. It should not begin right at the point of curve as the deceleration lane appears to be an extension of the tangent, and motorists are more likely to be confused. The ramp proper should begin with a section of tangent or a long-radius curve to permit a long and gradual reversing of the superelevation.

An alternate design, which will usually avoid operational problems, is to locate the exit terminal a considerable distance upstream from the point of curve. A separate and parallel ramp roadway is provided to connect with the ramp proper.

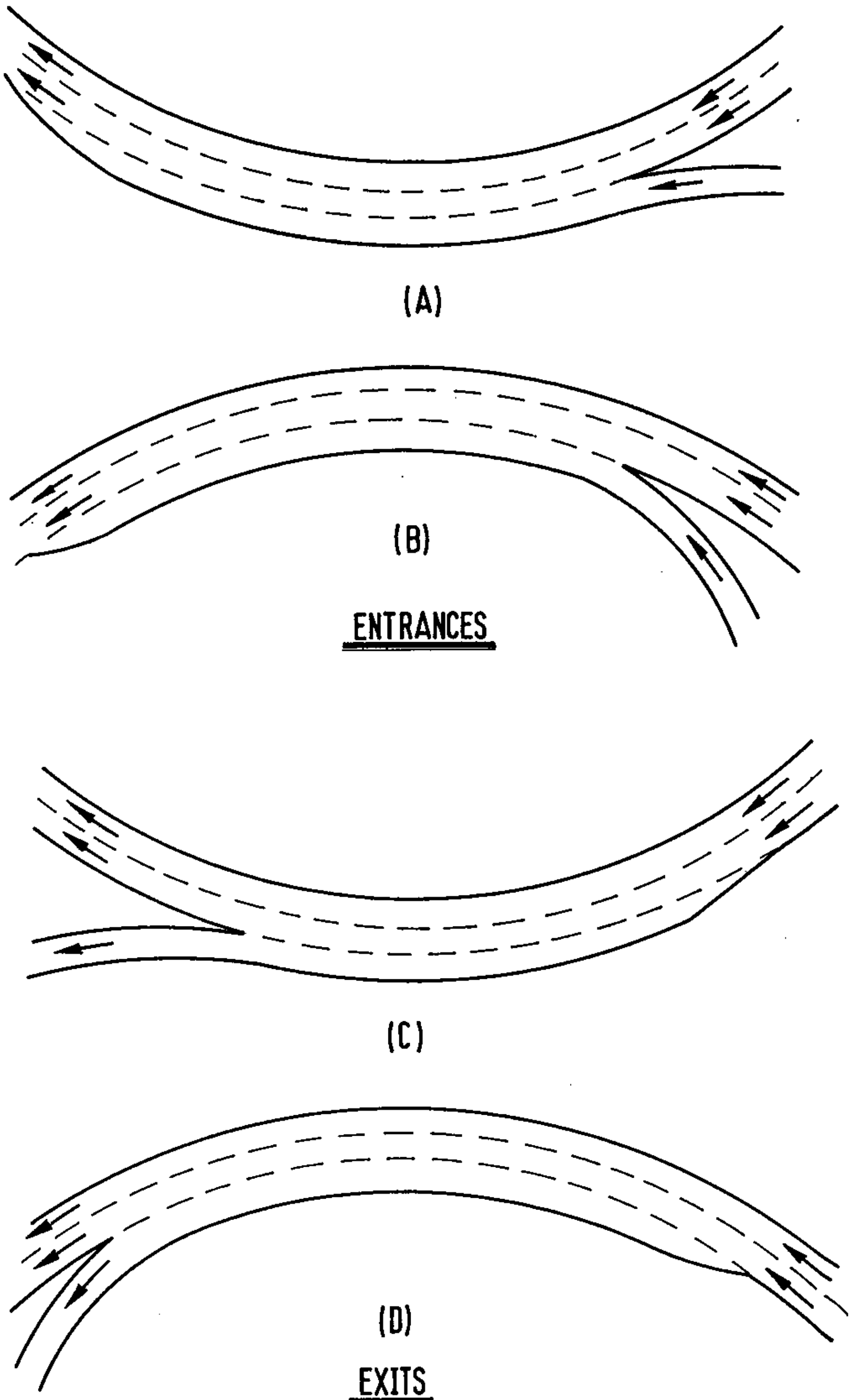
With the highway curving to the left and the exit located on the left (Figure 6-8 D) there is also a tendency for vehicles to exit inadvertently. Again, the taper should be short to provide additional "target" value for the deceleration lane. With this configuration the superelevation of the deceleration lane is readily effected by continuing the rate from the through pavement and generally increasing it to the rate required on the ramp curve.

(k) Multi-Lane Free Flow Terminals

Multilane terminals are required where traffic is too great for single-lane operation. Other considerations that may call for multilane terminals are through-route continuity, queuing on long ramps, lane balance, and design flexibility. The most common multilane terminals consist of two-lane entrances and exits at expressway. Other multilane terminals are sometimes termed "major forks" and "branch connections". The latter terms denote a separating and joining of two major routes.

Two-lane entrances

Two-lane entrances are warranted for two situations: either as branch connections or because of capacity requirements of the on-ramp. In order to satisfy lane-balance requirements, at least one additional lane must be provided downstream. This addition



PARALLEL – TYPE RAMP TERMINALS ON CURVES (DIAGRAMMATIC)

FIGURE 6 - 8

may be a basic lane if also required for capacity, or an auxiliary lane that may be dropped 750 to 900 m downstream or at the next interchange. In some instances two additional lanes may be necessary because of capacity requirements.

When only one additional lane is required, either the inside tapered or parallel design will be satisfactory. The choice would be dependent on local usage. Intermixing of the two designs is not recommended within a system route or an urban-area system.

If the two-lane entrance is preceeded by a two-lane exit, there is probably no need to increase the basic number of lanes on the expressway from a capacity standpoint. In this case the added lane that results from the two-lane entrance is considered an auxiliary lane, and it may be dropped approximately 750 m or more downstream from the entrance.

Figure 6-9 illustrates simple two-lane entrance terminals where a lane has been added to the expressway. The number of lanes on the expressway has little or no effect on design of the terminal.

Figure 6-9(A) shows a taper-type entrance and Figure 6-9(B), a parallel-type entrance.

The basic form or layout of a two-lane taper-type entrance, as shown in Figure 6-9(A), is that of single-lane taper, with a second lane added to the left or outer side and continued as an added or auxiliary lane on the expressway. As in the case of a single-lane tapered entrance, an angle of convergence of about 1 or slightly more, corresponding to a rate of convergence of about 1:50 represents a near-optimum design. The length of a two-lane taper-type entrance is approximately the same as that for a single-lane entrance.

With the parallel type of two-lane entrance, as shown in Figure 6-9 B, the right lane of the ramp is continued onto the expressway as an added lane.

The left lane of the ramp is carried as a parallel lane and terminated with a tapered section of about 90 m long. The length of the left lane should be in the range of 180 to 360 or more. Major factors in determining the needed length are the traffic volume on the ramp and the volume on the expressway. With traffic from the ramp only slightly in excess of the design capacity of a single-lane ramp, a length of 180 m, plus taper, may suffice. Where the combined volume for the expressway and ramp is near the design capacity of the expressway downstream from the merging area, a length of up to 900 m may be needed to blend the traffic smoothly into a freely following stream.

Traffic operation at a parallel type of two-lane entrance is entirely different from that at a taper type of two-lane entrance. With the two-lane parallel-type entrance the majority of traffic from the ramp will use the right lane of the ramp, which is continued onto the expressway as an added lane.

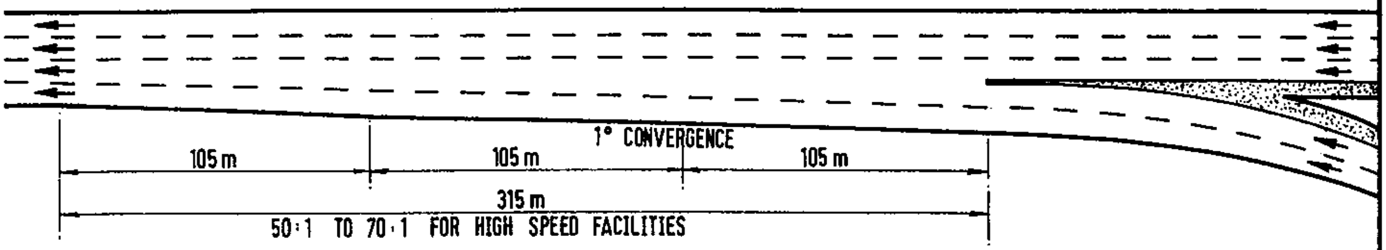
With the taper type of two-lane entrance, drivers tend to use the left lane rather than the right lane, as they do when using the parallel type of entrance. Either form of two-lane entrance is satisfactory if used exclusively within an area or a region, but they should not be intermixed along a given route.

Two-lane exits

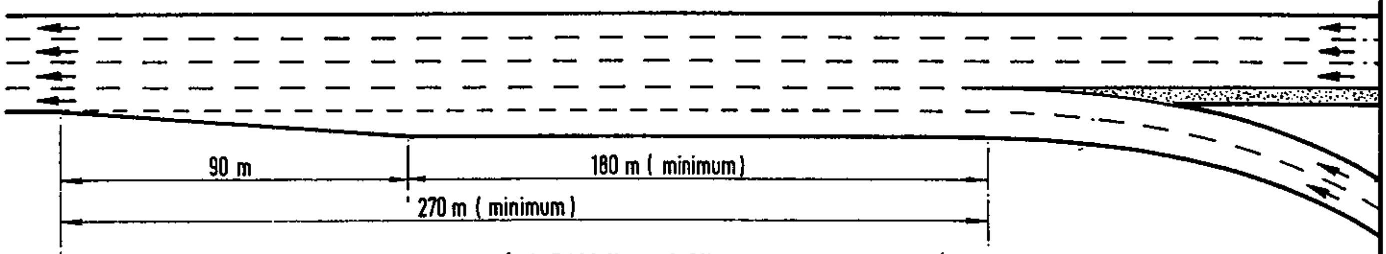
To satisfy lane balance requirements and not to reduce the basic number of through lanes, it is usually necessary to add an auxiliary lane upstream from the exit. A distance of approximately 450 m is required to develop the full capacity of a two-lane exit. Typical designs for two-lane exit terminals are shown in Figure 6-10; the taper type is illustrated in Figure 6-10(A) and the parallel type in Figure 6-10(B).

In cases where the basic number of lanes is to be reduced beyond a two-lane exit, the basic number of lanes should be carried beyond the exit before the outer lane is dropped. This design provides a recovery area for any through vehicles that remain in that lane.

With the parallel type of two-lane exit, as shown in Figure 6-10 (B) the operation is different from the taper type in that traffic in the outer through lane of the expressway must change lanes in order to exit.



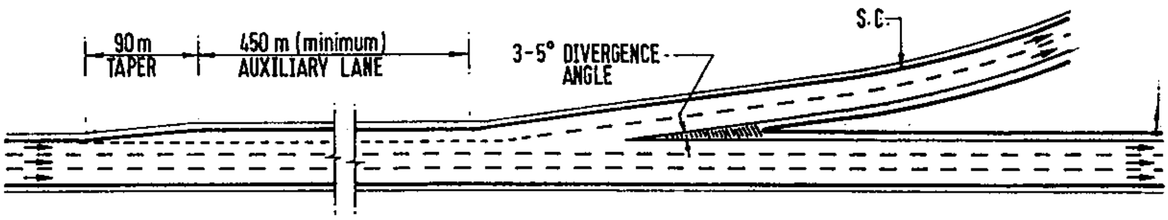
A) TAPERED DESIGN



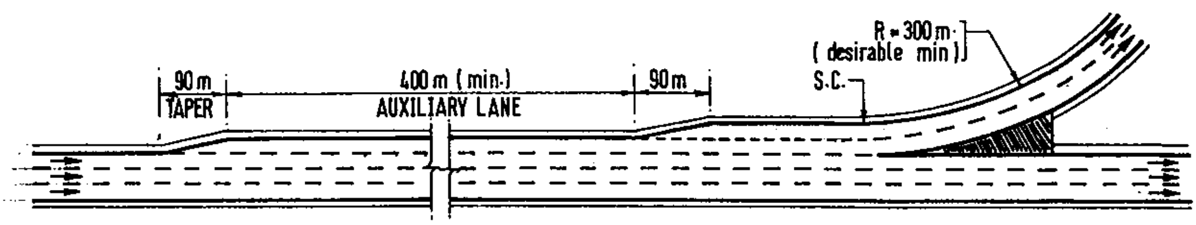
B) PARALLEL DESIGN

TYPICAL TWO - LANE ENTRANCE RAMPS

FIGURE 6 - 9



A) TAPERED DESIGN



B) PARALLEL DESIGN

TYPICAL TWO - LANE EXIT RAMPS

FIGURE 6 - 10

Considerable lane changing is necessary in order for the exit to operate efficiently. This entire operation requires a substantial length of highway, which is dependent in part on the total traffic volume on the expressway and especially on the volume using the exit ramp. The total length from the beginning of the first taper to the point of departure of the ramp travelled way from the left-hand through lane of the expressway will range from 760 m for turning volumes of 1,500 vph or less upward to 1060 m for turning volumes of 3,000 vph.

Major Forks and Branch Connections

A major fork is defined as the bifurcation of a directional roadway, of a terminating expressway route into two directional multilane ramps that connect to another expressway or as the diverging area created by the separation of an expressway route into two separate expressway routes of about equal importance.

The design of major forks is subject to the same principles of lane balance as any other diverging area. The total number of lanes in the two roadways beyond the divergence should exceed the number of lanes approaching the diverging area by at least one. Desirably, the number of lanes should be increased by only one. Operational difficulties invariably develop unless traffic in one of the interior lanes has an option of taking either of the diverging roadways. The nose should be placed in direct alignment with the centerline of one of the interior lanes, as illustrated in Figure 6-11.

This interior lane is continued as a full-width lane, both left and right of the gore. Thus, the width of this interior lane will be at least 7.0 m at the painted nose (prolongation of pavement-edge stripes) and preferably not over 8.5 m. The length over which the widening from 3.5 to 7m takes place should be within the range of 300 or 550 m.

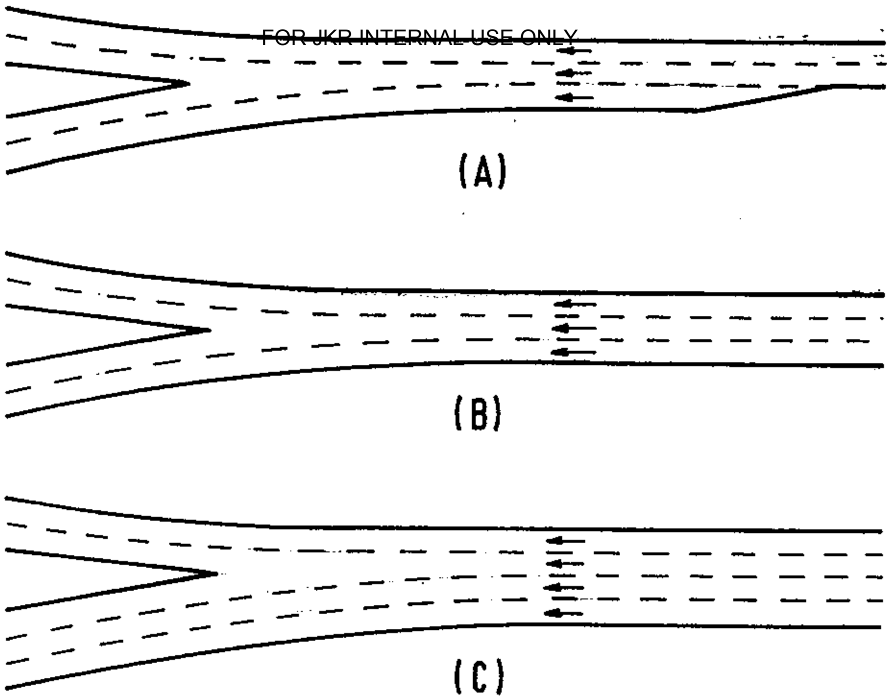
In case of a two-lane roadway separating into two, two-lane routes there is no interior lane. In such cases it is advisable to widen the approach roadway to three lanes, thus creating an interior lane. The lane is added on the side of the fork that serves the lesser traffic volume. In the illustration, Figure 6-11(A) the left (lower) fork would be the more lightly travelled of the two. The widening from 7m for the approach roadway to about 15 m at the painted nose should be accomplished in a continuous sweeping curve with no reverse curvature in the alignment of the pavement edges.

A branch connection is defined as the beginning of a directional roadway of an expressway formed by the convergence of two directional multilane ramps from another expressway or by the convergence of two expressway routes to form a single expressway route.

The number of lanes downstream from the point of convergence may be one lane fewer than the combined total on the two approach roadways. In some cases the traffic demand may require that the number of lanes going away from the merging area be equal to the sum of the number of lanes of the two roadways approaching it, and a design of this type will pose no operational problem. Such a design is illustrated in Figure 6-12(A).

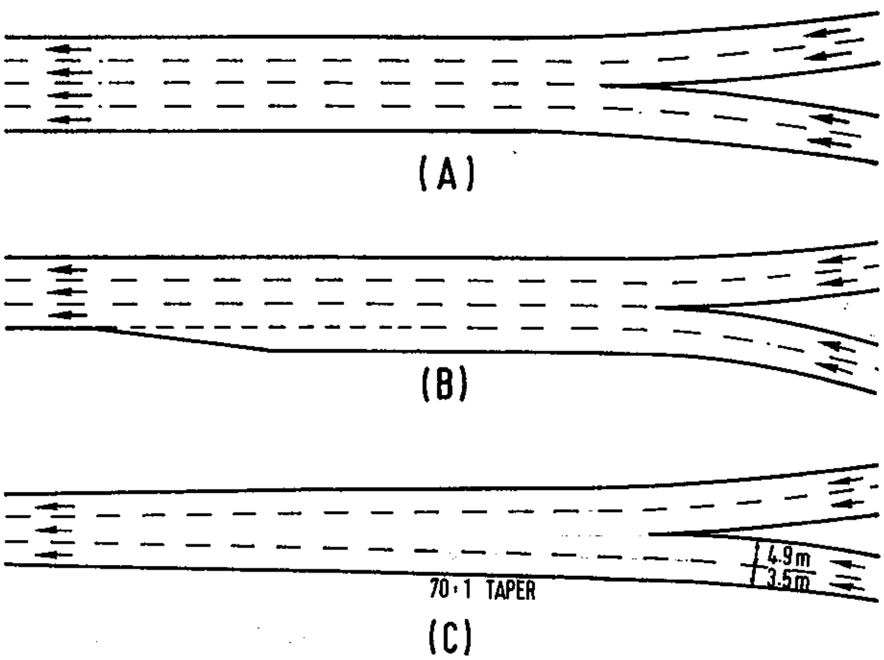
Where a lane is to be dropped, which is the more common case, a means for accomplishing the reduction is discussed in the section "Lane Reduction". The lane that is terminated will ordinarily be the exterior lane from the roadway serving the lowest volume per lane. However, some consideration should also be given to the fact that the outer lane from the roadway entering from the left is the slow-speed lane for that roadway; whereas the opposite is true for the roadway entering from the right. If the traffic volume per lane are about equal, it would be proper to terminate the lane on the left, as shown in Figure 6-9(B).

Another consideration is the possibility of a high-speed inside merge, as in Figure 6-12(C). This merge should be treated as any other high-speed merging situation by adding a lane on the left.



EXAMPLES OF MAJOR FORKS

FIGURE 6 - 11



NOTE :

FOR PROPER MERGE DESIGNS SEE TWO LANE ENTRANCES

EXAMPLES OF BRANCH CONNECTIONS

FIGURE 6 - 12

CHAPTER 7: INTERCHANGE CAPACITY

7.1 GENERAL

Traffic movement at interchanges is quite different from the uniform flow on an open road. Diverging, merging, and weaving movements frequently take place in the interchange area. Stopping may also occur at the minor crossroad terminals. Every facility provided to meet these various demand of movements must have sufficient capacity to maintain the designated level of service. The different facilities must also furnish the same level of service as that assumed for the highway crossing at the interchange. There must not be any bottleneck or spot congestion.

The capacity at the points of those special movements govern the performance of the whole interchange. Checking of the capacity thus must be done at the points of the special movements.

The capacity analysis for the ramp terminals and weaving section should follow that which is detailed in "Highway Capacity Manual" - Special Report 209, Transportation Research Board, 1985. (Chapter 4 and 5).

The following sections are taken from the above publication and gives a brief outline of the methodology and procedures that are adopted.

7.2 RAMP TERMINALS

7.2.1 Ramp Components

A ramp may consist of up to three geometric elements of interest i.e.:-

(a) Ramp terminal at the expressway

(b) Ramp roadway proper

(c) Ramp terminal at the minor crossroad.

A ramp terminal at the expressway is generally designed to permit high-speed merging or diverging movements to take place with a minimum of disruption to the adjacent expressway traffic stream. Geometric elements such as provision and length of acceleration/deceleration lanes, angle of convergence and divergence, relative grades and other aspects may impact ramp operations.

The ramp roadway itself may also vary widely from location to location. They can vary in the number of lanes, length, design speed, grades and horizontal curvature. They are rarely a source of operational difficulties unless a traffic accident causes a disruption along its length.

The ramp terminal at the minor crossroad can be of a type permitting uncontrolled merging or diverging movements or it can take the form of an at-grade intersection, in which case the requirements as in Arahan Teknik (Jalan) 11/87 "A Guide to the Design of At-Grade Intersections" should be met.

7.2.2 Operational Characteristics

A ramp terminal at the expressway is an area of competing traffic demands for space.

In the merge area, entrance-ramp vehicles try to find openings or "gaps" in the adjacent expressway lane traffic stream. As most ramps are on the left side of the facility, the expressway lane most directly impacted is the shoulder lane, which is designated as lane 1.

At exit-ramps, the basic maneouver is to diverge Existing vehicles must occupy the lane adjacent to the ramp so that there is a nett effect of other drivers redistributing themselves amongst the other lanes.

A ramp will operate efficiently only if all of its elements, the terminals with expressways, and minor crossroads and the ramp roadway have been properly designed. It is critical to note that a breakdown on any one of these elements will adversely affect the operation of the entire ramp, which may also extend to the facilities it connects.

7.2.3 Computational Procedure For Ramp Terminals at the Expressway

The step-by-step computational procedure for the capacity analysis of ramp terminals as detailed from the Highway Capacity Manual is as follows:-

<u>Step 1</u> - Establish Ramp Geometry and Volumes

The establishment of a geometric configuration includes the type, location and volumes on the adjacent ramps.

<u>Step 2</u> - Compute Lane 1 volume

This is computed using one of the nomographs or the approximation procedure. The Lane 1 volume is dependent on the ramp volume, the total expressway volume upstream of the ramp, the distance and volume of the adjacent upstream and/or downstream ramps and the type of ramp in question.

<u>Step 3</u> - Convert all volumes to Passenger Cars Per Hour

<u>Step 4</u> - Compute Checkpoint Volumes

For each ramp analysis, there are up to three checkpoint volumes for each ramp or pair of ramps i.e.: - merging volume, diverging volume or total expressway volume.

<u>Step 5</u> - Convert checkpoint volumes to Peak Flow Rates

<u>Step 6</u> - Find Relevant Levels of Service

The level of service for a given analysis is found by comparing the checkpoint flow rates for merging, diverging and total expressway volume with the given criteria.

In many cases, the various operational elements (merges, diverges, expressway flows) will not be in balance i.e. have the same level of services (LOS). In such cases, the worst resultant LOS is assumed to govern and would then be candidates for improvement if the resulting LOS is unacceptable.

The LOS for the point locations should be in balanced with the expressway as a whole. It is desirable in fact to have the LOS of the merging and diverging points better than the LOS for the total expressway volume.

7.3 WEAVING SECTIONS

7.3.1 General

Weaving is defined as the crossing of two or more traffic streams travelling in the same general direction along a significant length of highway, without the aid of traffic control devices. Weaving areas are formed when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane.

Weaving areas require intense lane-changing maneouvers as drivers must access lanes appropriate to their desired exit point. Thus, traffic in a weaving area is subject to turbulence in excess of that normally are present on basic highway sections. This turbulence presents special operational problems and design requirements.

Figure 7-1 shows the formation of a weaving area. If entry and exit roadways are referred to as "legs," vehicles travelling from leg A to leg D must cross the path of vehicles travelling from leg B to leg C. Flows A-D and B-C are, therefore, referred to as weaving flows. Flows A-C and B-D may also exist in the section, but these need not cross the path of other flows, and are referred to as non-weaving flows. Figure 7-1 shows a simple weaving area, formed by a single merge point followed by a single diverge point. Multiple weaving-areas are formed by one merge followed by two diverges or two merges followed by a single diverge.

7.3.2 Weaving Length

The weaving area length measurement is as shown in Figure 7-2 and is the length from the merge gore area at a point where the left edge of the expressway shoulder lane and the right edge of the merging lane are 0.6m apart to a point at the diverge gore area where the two edges are 3.6 m apart.

All lane changes to complete the many weaving movements must be within the weaving length. This thus constraints the time and space in which the driver must make all the required lane changes which increases the intensity of lane-changing and the level of turbulence.

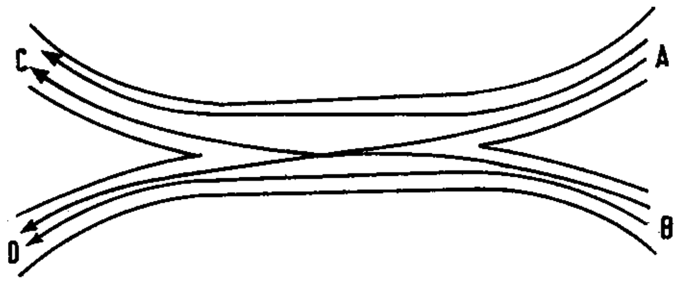


FIGURE. 7.1: FORMATION OF A WEAVING SECTION

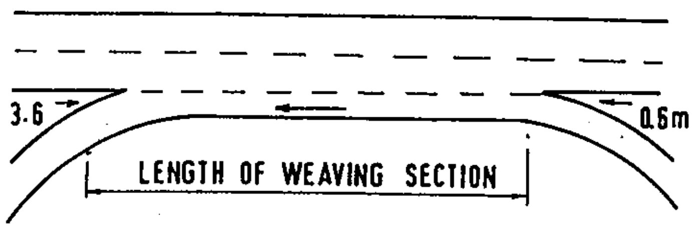


FIGURE 7-2: LENGTH OF WEAVING SECTION

7.3.3 Configuration

Configuration refers to the relative placement and number of entry and exit lanes for the section and can have a major impact on the operational feature of weaving areas.

There are three primary types of weaving configuration, defined in terms of the minimum number of lane changes which must be made by weaving vehicles as they travel through the section. These are referred to as Types A, B and C as shown in Figures 7-3, 7-4 and 7-5 respectively

(a) Type A Weaving Areas

Type A weaving areas require that each weaving vehicle make one lane change in order to execute the desired movement. Figure 7-3 show two examples of Type A weaving areas.

Because weaving vehicles in a Type A weaving area must cross the crown line, weaving vehicles are usually confined to occupying the two lanes adjacent to the crown line. Lanes adjacent to the crown line are, therefore, generally shared by weaving and non-weaving vehicles. One of the most significant effects of configuration on operations is to limit the maximum number of lanes which weaving vehicles may occupy while traversing the section.

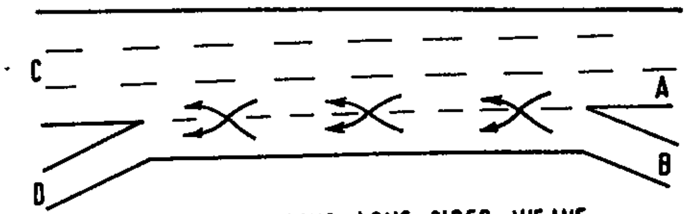
(b) Type B Weaving Areas

Type B weaving areas involve multilane entry and/or exit legs and are referred to as major weaving sections. They are characterised by:-

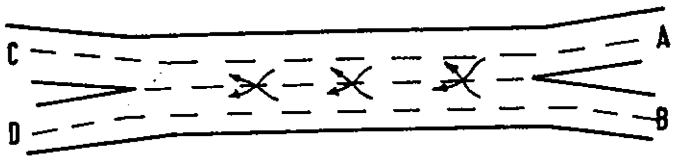
i) one weaving movement may be accomplished without making any lane change.

ii) the other weaving movement requires at most one lane change.

Type B weaving areas are extremely efficient in carrying large weaving volumes, primarily because of the provision of a "through lane" for one of the weaving movements. Weaving maneouvers can be accomplished with a single lane change from the lane or lanes adjacent to this "through lane". Thus, weaving vehicles can occupy a substantial number of lanes in

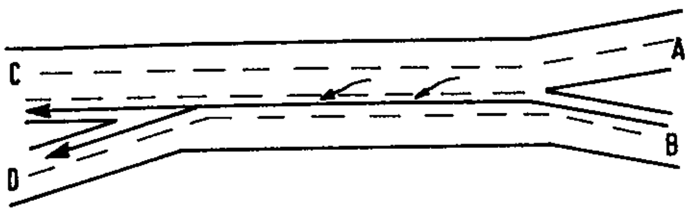


(A) RAMP - WEAVE / ONE SIDED WEAVE

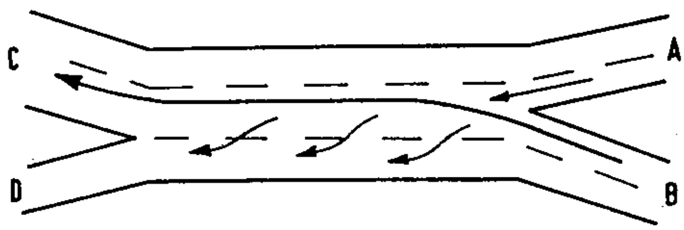


(B) MAJOR WEAVE WITH CROWN LINE

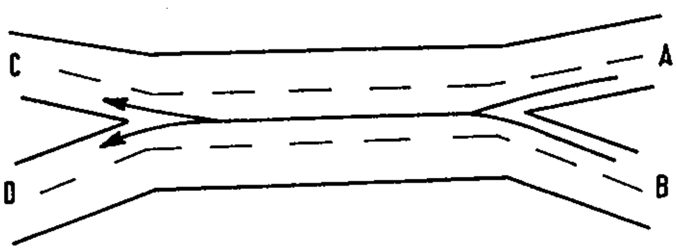
FIGURE 7-3: TYPE A WEAVING AREAS



(A) MAJOR WEAVE WITH LANE BALANCE AT EXIT GORE.

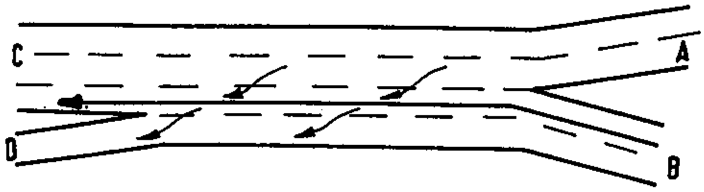


(B) MAJOR WEAVE WITH MERGING AT ENTRANCE GORE

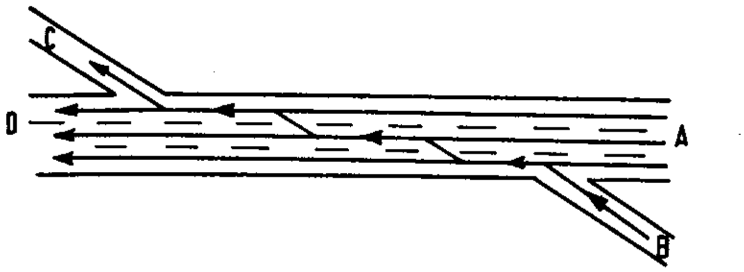


(C) MAJOR WEAVE WITH MERGING AT ENTRANCE GORE AND LANE BALANCE AT EXIT GORE

FIGURE 7-4: TYPE B WEAVING AREAS



(A) MAJOR WEAVE WITHOUT LANE BALANCE OR MERGING



(B) TWO - SIDED WEAVE

FIGURE 7-5 : TYPE C WEAVING AREAS

the weaving section, and are not restricted in this regard as in Type A sections.

(c) Type C Weaving Areas

Type C weaving areas are similar to Type B sections in that one or more "through lanes" are provided for one of the weaving movements. The distinguishing feature between Type B and C is the number of lane changes required. A Type C weaving area is characterised by:-

i) one weaving movement may be accomplished without making a lane change

ii) the other weaving movement requires two or more lane changes.

7.3.4 Weaving width and Type of Operation

Another geometric characteristic with a significant impact on weaving area operations is the width of the weaving area, measured as the number of lanes in the section. It is, however, not only the total number of lanes that impacts weaving area operations, but the proportional use of those lanes by weaving and non-weaving vehicles.

The nature of weaving movements create traffic stream turbulence, and results in a weaving vehicle consuming more of the available roadway space than a non-weaving vehicle. The exact nature of the relative space use depends on the relative weaving and non-weaving volumes using the weaving area and the number of lane changes weaving vehicles must make. The latter is, as discussed, dependent on the configuration of the weaving section. Thus, the proportional use of space is dependent not only on relative volumes, but on the configuration of the weaving area.

Configuration has a further impact on proportional use of available lanes. The configuration can limit the ability of weaving vehicles to use outer lanes in the section. This limitation is most severe in Type A sections, in which all weaving vehicles must cross a crown line, and is least severe in Type B sections.

In general, vehicles in a weaving area will make use of available lanes in such a way that all component flows achieve approximately the same average running speed, with weaving flows somewhat slower than non-weaving flows. Occasionally, the configuration limits the ability of weaving vehicles to occupy the proportion of available lanes required to achieve this equivalent or balanced operation. In such cases, weaving vehicles occupy a smaller proportion of the available lanes than desired, while non-weaving vehicles occupy a larger proportion of lanes than for balanced operation. When this occurs, the operation of the weaving area is classified as'constrained' by the configuration. The result of constrained operation is that non-weaving vehicles will operate at significantly higher speeds than weaving vehicles.

Where configuration does not restrain weaving vehicles from occupying a balanced proportion of available lanes, the operation is classified as 'unconstrained'.

7.3.5 Computational Procedure For Simple Weaving Areas

The step by step computational procedure for the evaluation of the level of service in an existing or projected simple weaving area as detailed in the Highway Capacity Manual is as follows:-

<u>Step 1</u> - Establish Roadway and Traffic Conditions

All existing or projected roadway and traffic conditions are to be specified. Roadway condition include the length, number of lanes, and type of configuration for the weaving area under study as well as the lane widths and the general terrain.

Traffic conditions include the distribution of vehicle types in the traffic stream, and the peak hour factor where the component flows have differing peaking characteristics.

<u>Step 2</u> - Convert All Traffic Volumes to Peak Flow under Ideal Conditions

<u>Step 3</u> - Construct Weaving Diagram

<u>Step 4</u> - Compute Unconstrained Weaving and Nonweaving speeds

<u>Step 5</u> - Check for Constrained Operation.

<u>Step 6</u> - Check Weaving Area Limitations

<u>Step 7</u> - Determine the Level of Service (LOS)

Levels of service in weaving areas are directly related to the average running speeds of weaving and non-weaving vehicles. The LOS for a given analysis is found by comparing the calculated average weaving and non-weaving speeds with the given criteria.

The analysis of multiple weaving areas is similar except that it involves the construction of the appropriate weaving diagrams for each sub-segment of the area in question, which is then analysed as a simple weaving area.

CHAPTER 8: INTERCHANGE SIGNING

8.1 GENERAL

The development of a signing system for an interchange must be approached on the premise that the signing is primarily for the benefit and direction of drivers who are not familiar with the route or area. The signing thus must provide drivers with clear instructions for an orderly progress to their destinations.

The installation of the signs must be taken as an integral part of the interchange and expressway facility and as such must be planned concurrently with the geometric design.

Drivers should be confronted with consistent signing on the approaches to interchanges as they drive from one State to another and when driving through rural or urban areas. Geographical, geometric and operating factors create significant differences between urban and rural conditions and the signing must take these into account.

8.2 TYPES OF INTERCHANGE SIGNING

8.2.1 Standard Traffic Signs

Standard traffic signs are normally installed at ramp terminals i.e. exits, entrances and gore areas. The full complement of standard traffic signs (in accordance with Arahan Teknik (Jalan) 2A/85) must be installed to ensure the proper operation of the interchange.

Examples of standard traffic signs installed at ramp terminals are:-

(a) at gore areas - a combination double arrow (WD 36) and an obstruction marker sign (WD 23).

(b) at ramp exit terminals - stop (RP 1), giveway (RP 13) or traffic signal signs (WD 23) sign together with no-entry (RP 4) and/or advanced signs (WD 18/WD 19)

(c) at ramp entrance terminals - give way signs (RP 13) together with its advanced sign (WD 19).

8.2.2 Guide Signs

The layout design and application of the various types of guide signs must follow the relevant and the latest JKR standards. The following general guidelines should also be followed:-

(a) Destination Signs - On the expressway approaches a minimum of two destination signs (i.e. 500 m and 1 km away) should be installed while on the minor crossroad approaches one destination sign (at 500 m away) is sufficient.

(b) Directional Signs - On the expressway, a minimum of two directional signs (one at the gore area if feasible) and a complementary overhead gantry sign at the beginning of the taper section of the exit should be installed. On a minor crossroad, the overhead gantry sign is usually not required.

(c) Distance signs - these are required on all the roadways leading away from the interchange area.

Figures 8-1 and 8-2 give the typical guide signing requirements for a diamond interchange and a full cloverleaf interchange (without collector - distributor roads) respectively. Figures 8-1 and 8-2 also only show the signing requirements for the north bound approach of the main expressway facility and the east bound approach of the crossroad. Full signing of the interchange should cover all approaches and ramps.

Figures 8-1 and 8-2 should be taken only as typical examples as the signing arrangements at every interchange may be different due to physical or operational constraints, in which case the necessary modifications should be made. For example, if the gore area is physically restrictive, the directional sign can be installed as a butterfly sign (ie. with only one post).

8.2.3 Gantry Signs

Gantry signs are very effective at interchanges and should be provided at all ramp exits on the expressway approaches before the interchange, especially where the physical constraints of the site makes it difficult to provide adequately for the normal directional signs. This will be more apparent on

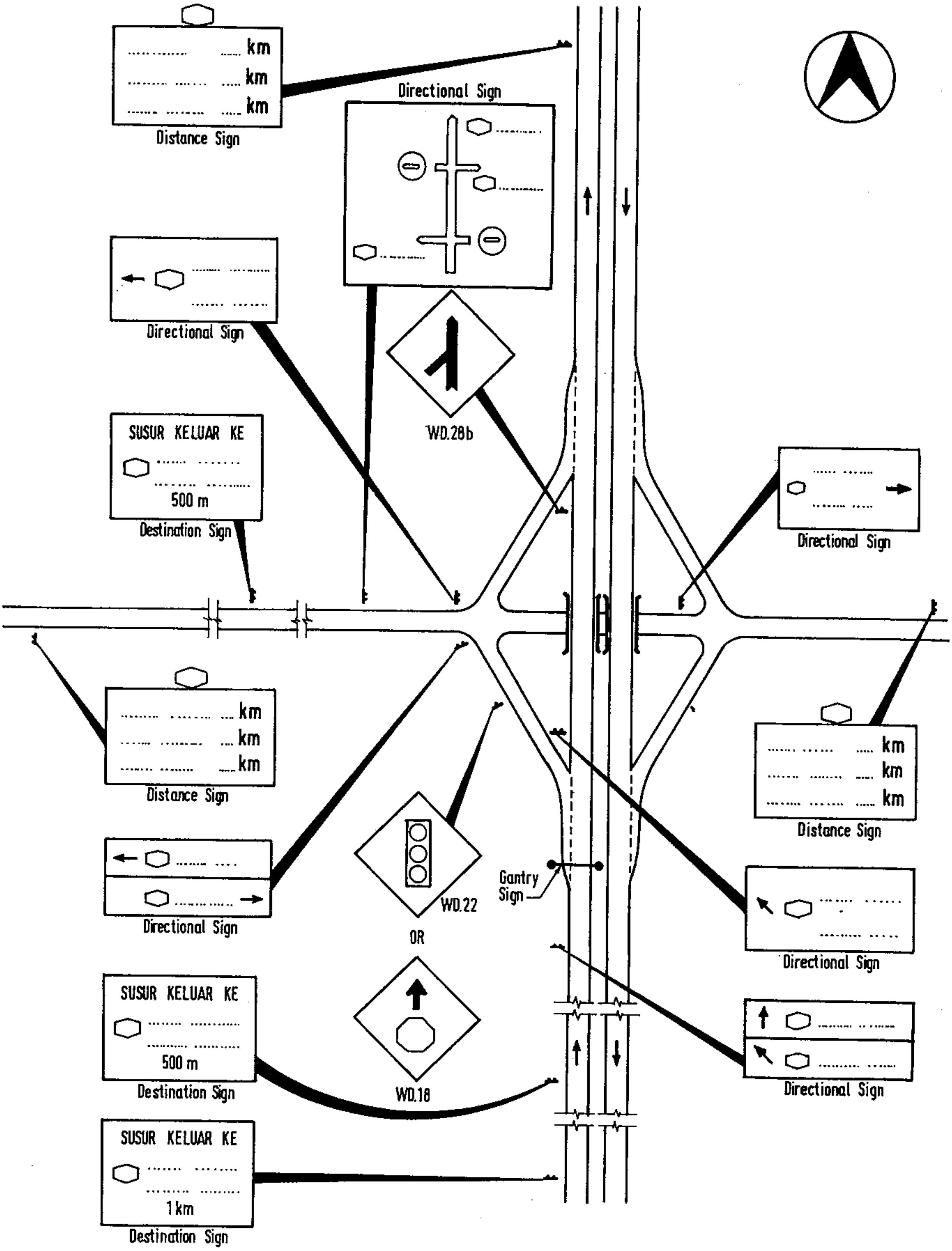


FIGURE 8-1: TYPICAL SIGNING REQUIREMENTS OF DIAMOND INTERCHANGE

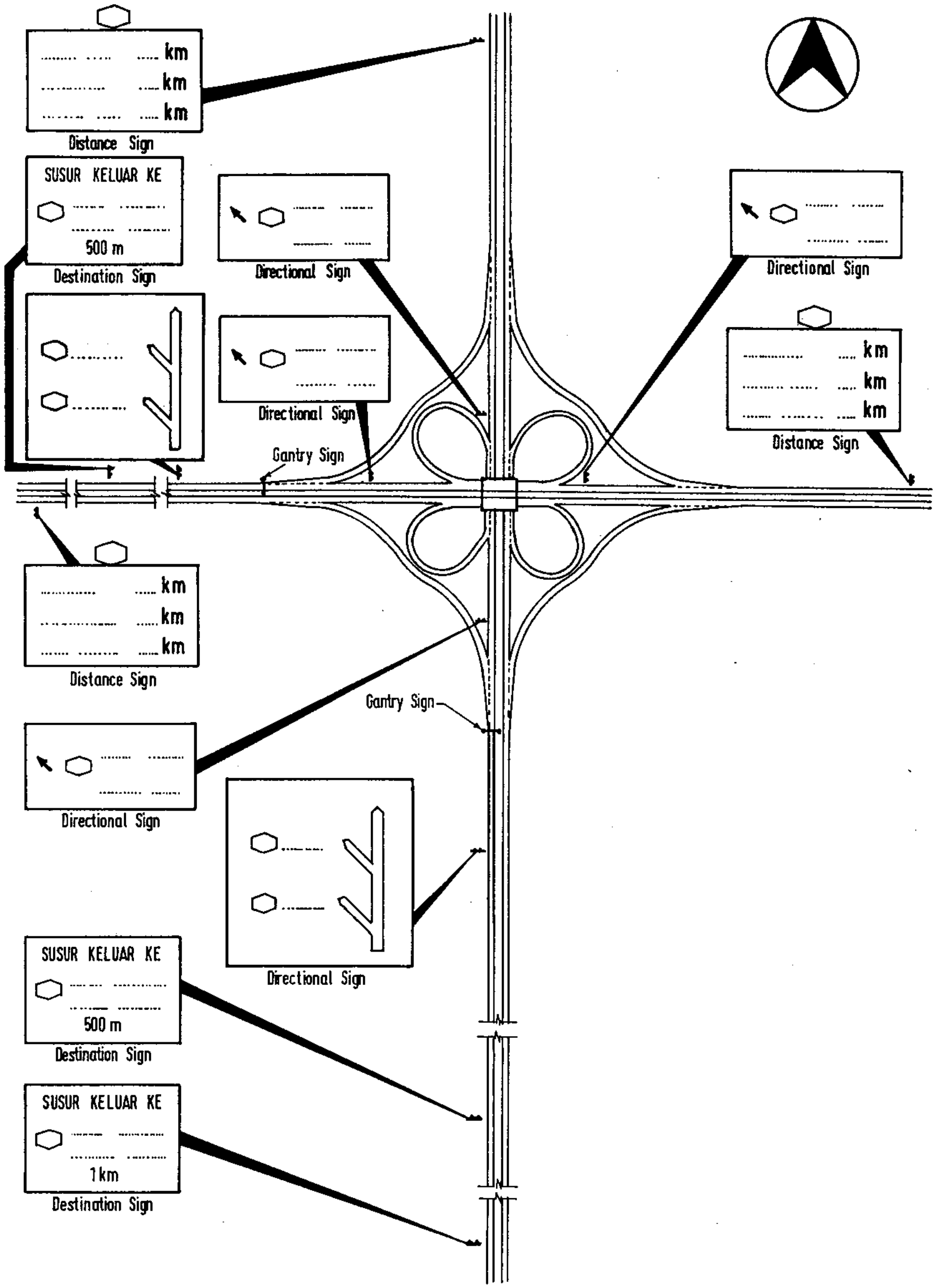


FIGURE 8-2: TYPICAL SIGNING REQUIREMENTS OF FULL CLOVERLEAF INTERCHANGE

interchanges in urban than rural areas.

The layout, structural design and other requirements of the gantry signs should follow the latest JKR standards.

CHAPTER 9 OTHER DESIGN FEATURES

9.1 TESTING FOR EASE OF OPERATION

Each section of expressway that includes a series of interchanges or a succession of exits and entrances should be tested for operational characteristics of the route after the preliminary design, including adaptability, capacity, and operational features. The test is an evaluation of the section for ease of operation and for route continuity from a driver's point of view, both of which are affected by the location, proximity, and sequence of exits and entrances, the merging, diverging, and weaving movements necessary, practicability of signing, and clarity of paths to be followed.

A route may be tested by isolating that part of the plan, for each path, and examining it only with regard to other parts of the layout that will affect a driver on the path being tested. Certain weaknesses of operation not evident on the overall plan may be revealed in testing a single path of travel.

The plan should be tested by drawing of tracing separately the path of each principal origin and destination and studying thereon those physical features that will be encountered by a driver. The test can also be made on an overall plan on which the path to be studied and the stubs of connecting roads are colored or shaded.

The plan should show the peak-hour volumes, number of traffic lanes, and peak-hour and off peak-hour running speed. Thus, the designer can visualize exactly what the driver sees - only the road being travelled, with the various points of entrances and exits and the signing along it - and have a sense of the accompanying traffic.

Such an analysis indicates whether or not confusion is likely because of exits and entrances too close together or whether interference is likely because of successive weaving sections. It would show also whether or not the path is clearly defined, if it is feasible to sign the facility properly, and if major or overhead signs are required and where they may be placed. The test may show that the path is easy to travel, direct in character, and free from sections that might confuse drivers, or it may show that the path is sufficiently complex and confronted with disturbing elements to require adjustment in design. As a result it may be necessary to move or eliminate certain ramps. In an extreme case the best may show that it is necessary to change the overall pattern by eliminating an interchange, or to introduce collector-distributor roads in order to prevent interference with through traffic, or to make some other radical changes in design.

9.2 GRADING, AESTHETICS AND LANDSCAPE DEVELOPMENT

Grading at an interchange is determined chiefly by the alignments, profiles, cross sections, and drainage requirements for the intersecting highways and ramps. Each through roadway or ramp should not be treated as a separate unit and graded to standard cross sections without regard to its relation with adjacent roads and to the surrounding topography. Instead, the whole construction area should be designed as a single unit to keep construction and maintenance costs to a minimum, to obtain maximum visibility, and enhance the appearance of the area.

It is also important that the aesthetic aspects of all structures be taken into consideration to ensure that it harmonises with the surrounding areas and the subsequent interchanges.

Landscape development and design of the entire interchange area should also be considered as part of the overall facility and should be included during the geometric design stage.

9.3 ALIGNMENT DESIGN

An important and early step in the design of interchanges is the initial bridge control study in which the preliminry alignment and profiles of the intersecting roads are developed to determine the controls for bridge design. Such elements as clearances, kerbs, and position and extent of walls should be examined with regard to general grading before conclusions are drawn for the bridge design, particularly for lengths of wing walls. Minor modifications in alignment and profile, in abutments and walls, and in related earthworks may produce a more desirable solution as a whole.

At interchanges, and elsewhere as feasible, steep roadside earth slopes should be avoided for all roads and ramps. Flat slopes should be used where feasible, for economical construction and maintenance, to increase safety, and to enhance the appearance of the area. Broad rounded drainageways or swalelike depressions should be used, where feasible, to encourage good turf and easy mower maintenance. V ditches and small ditches with steep side slopes should be avoided. Drainage channels and related structures should be as inconspicuous and maintenance-free as feasible. They should not be an eyesore, or a hazard to traffic.

Where good trees and other desirable landscape features exist, the contour grading and drainage plan should be designed to protect and preserve these features, as far as possible.

9.4 TREATMENT OF PEDESTRIAN TRAFFIC

Where ramps are connected to a crossroad which is not operated exclusively for motor vehicles and the volume of pedestrians, bicycles, and other slow moving vehicles on the road is considerable, pedestrian walks or cycle tracks should be provided. It should be grade separated from the ramps or detoured outside of the ramps if possible.

If an at-grade crossing is inevitable, it should be designed so that pedestrian traffic and vehicular traffic will cross at nearly right angle. Sufficient sight distances should also be provided. With proper arrangement of guard rails, tree planting, and traffic islands, the passage of pedestrians should be channelized; crossings other than those at designated areas is discouraged.

9.5 LIGHTING

Lighting is desirable, and sometimes necessary at interchanges where a series of serious judgements and decisions are required in diverging, merging, and lane changes. Drivers should be able to see not only the road ahead, but also the entire turning roadway area to discern properly the paths to be followed. They should also see all other vehicles that may influence their own behaviour.

The designer should consider lighting the entire interchange area when information value justifies it. This particular point has contributed substantially to the intensive use of high-mast lighting of interchanges, which is recommended.

The principal objective in the application of high-mast lighting to highway interchanges is to synthesize the visual advantage provided to the driver by daylight. Thus, the driver can see all things pertinent to the decision-making process in time to assimilate the information and then plan and execute his maneouvers effectively. He can also distinguish roadway geometry, obstructions, terrain, and other roadways, each in its proper perspective.

Additional advantages of high-mast lighting are related to safety and aesthetics. When there are fewer poles, there are fewer opportunities for collision. The masts can be located farther from the roadway so that the possibility of a collision with the luminaire support is virtually eliminated. Daytime aesthetics are greatly improved by removing the "forest" of poles generally necessary to light complex interchanges and intersections with continous lighting.

Without lighting there may be noticeable decrease in the usefulness of the interchange at night, when there would be more cars slowing down and moving with uncertainty than during daylight hours. Consideration should be given to making visible at night by roadway lighting (or reflectorizing devices) the parts of grade separation structures that may be hazardous such as kerbs, piers and abutments. The greater the volume of traffic, particularly turning traffic, the more important the fixed-source lighting at interchanges becomes.

9.6 DRAINAGE

The provision of proper drainage facilities is of utmost importance at interchanges and careful attention must be given to the requirements for an adequate drainage system and protection of the interchange facility from flooding. The design of the drainage system must be considered as an integral part of the design of the interchange.

The current JKR design standards and requirements are to be followed.

9.7 PUBLIC UTILITIES

The location and size of any underground and overhead public utility installations which are close enough to the interchange to be affected should be determined in the preliminary stages of the design. The design for its relocation should also be carried out.

Service tunnels/culverts or ducts should also be provided for future services to avoid the digging of the carriageway pavement.

APPENDIX A

PROCEDURES FOR THE

DESIGN OF INTERCHANGES

Procedures For The Design of Interchanges

A brief step by step procedure for the design of interchanges is as follows:

(a) Step 1 - Forecasting of traffic

For existing roads, the forecasting of traffic can be based on Highway Planning Unit's traffic census or from a manual classified counts carried out separately.

For new roads, the traffic forecast will have to be based on traffic studies carried out following established procedures.

The design period to be taken is 20 years after completion of the interchange. While stage construction is allowed, it should be restricted only to the ramps and pavement.

(b) Step 2 - Warrants for Grade Separation

With the traffic forecasted and other relevant information such as site conditions, accident records, road-user benefits etc., a warrant for the grade separation must be establised.

If grade separation is not warranted, the junction or intersection can be designed as an at-grade facility in accordance with Arahan Teknik (Jalan) 11/87.

(c) Step 3 - Determine Type of Interchange

The type of interchange that is required is then determined. A more detailed traffic study may be necessary to determined the traffic movements, composition of traffic etc before the configuration of the interchange can be decided upon. Other factors such as physical constraints on the site may also affect the type of configuration that can be selected, and may need to be taken into consideration. Where different configurations are possible, the most cost-effective one should be selected.

(d) Step 4 - Preliminary Geometric Design

With the selected interchange configuration, a preliminary layout design of the interchange is carried out based on the requirement of traffic volumes and geometric elements.

(e) Step 5 - Capacity Analysis

Capacity analysis is then carried out for the various checkpoints of the preliminary layout design. If the level of service determined is unacceptable, the layout design is then reviewed and revised until the required level of service is achieved. If the level of service is still unsatisfactory, the type of interchange configuration selected may have to be revised.

(f) Step 6 - Detailed Design

The detailed design of the various elements of the interchange is then carried out i.e.:-

(a) geometric elements such as horizontal and vertical alignments, etc which should be optimised for an economic design.

(b) bridge structures

(c) pavement structure

(d) drainage elements

(e) geotechnical elements

(f) signing and pavement markings

(g) grading and landscaping

(h) lighting

(i) other miscellaneous elements

Figure A-1 is a flow chart showing the flow of the procedures described above.

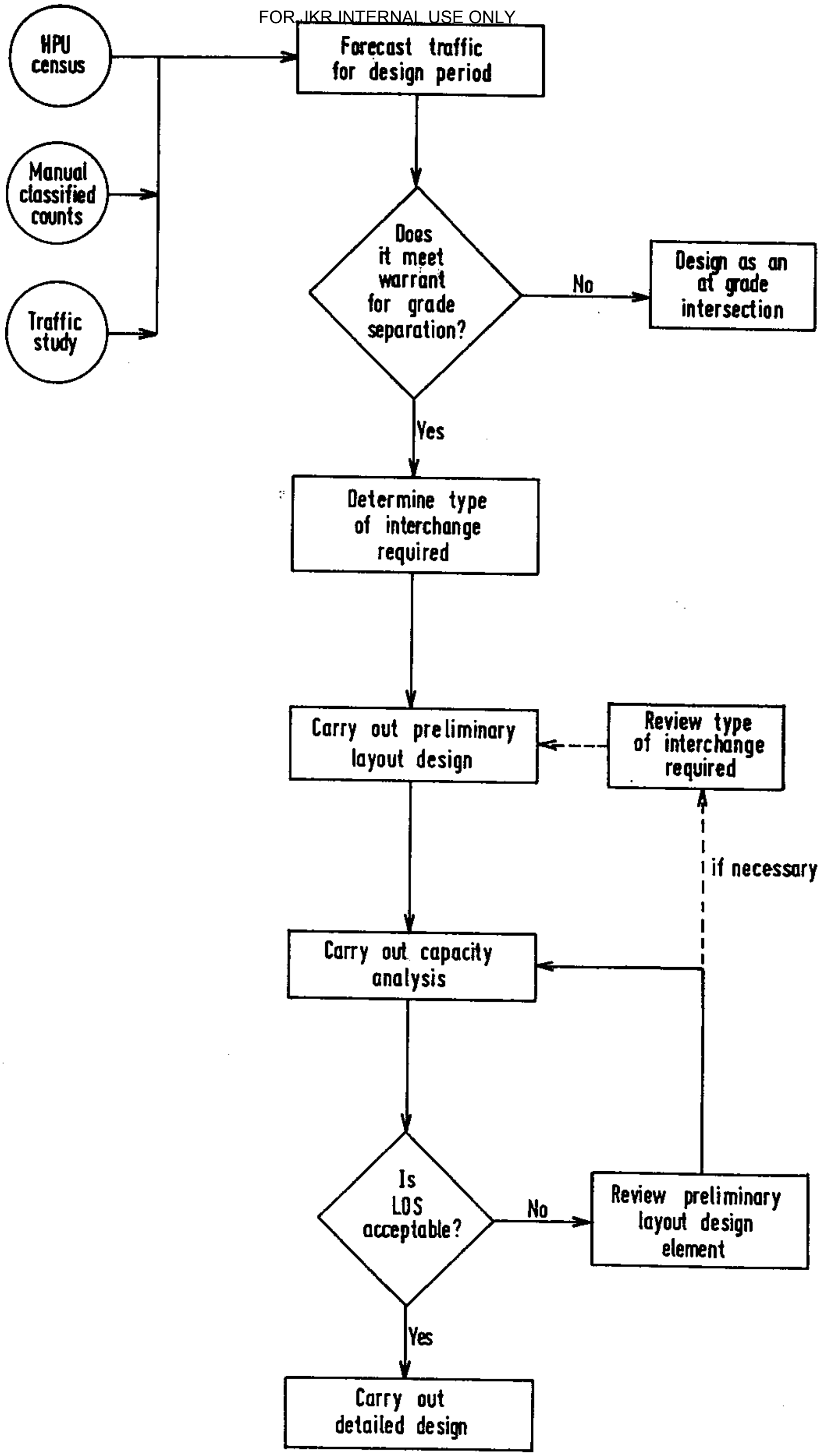


FIGURE A-1: FLOW CHART OF PROCEDURES FOR DESIGN OF INTERCHANGES

APPENDIX B

LIST OF REFERENCES

APPENDIX B

LIST OF REFERENCES

1. "A Policy on Geometric Design of Highways and Streets - 1984", American Association of State Highway and Transportation Officials, Washington D.C.

2. "Highway Capacity Manual" - Special Report 209, Transportation Research Board, National Research Council, Washington D.C. 1985.

3. "Manual of Geometric Design Standards for the Interurban Toll Expressway System of Malaysia" - Malaysian Highway Authority (LLM)