(11) Vertical Curves

1) Vertical Curves

1. Vertical curve should be provided on road sections where tangent grade changes.

2. Radii of vertical curvature should not be less than the values shown in Table 1.2.37, according to the design speed and the crest or sag types of curve. However, on a road of design speed of 60 km/h in an urban area, the radius of the crest type of vertical curve may be reduced to 1000m, when made necessary by a topographical or other reason.

3. The lengths of vertical curve should not be less than the values shown in Table 1.2.38.

Table 1.2.37 Radius of Vertical Curvature

Design Speed (km/h)	Radius of Vertical Curvature (m)	
	Crest Type	Sag Type
100	6,500	3,000
80	3,000	2,000
60	1,400	1,000
50	800	700
40	450	450
30	250	250
20	100	100

Table 1.2.38 Length of Vertical Curve

Design Speed (km/h)	Length of Vertical Curve
100	85
80	70
60	50
50	40
40	35
30	25
20	20

a) General

Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage. Minimum stopping sight distance should be provided in all cases. Longer curves are desired wherever feasible.

For simplicity the parabolic curve with an equivalent vertical axis centered on the vertical point of intersection is usually used in roadway profile design. There are two measures indicating the vertical curve. One is indicated by the length of curve, and another is indicated by the radius of curvature which is approximated to parabolic curve.

The relation between two measures are as follows;

$$L_r = \frac{R}{100} \cdot A$$

$$R = 100 \cdot \frac{L_r}{A}$$

where R = radius of vertical curvature, m;
Lr = length of vertical curve, m; and
A = algebraic difference in grades, percent

In the sag vertical curve of Figure 1.2.26, Io and I are tangent directions of vertical curve, where Io and I are vertical grades. The formula of parabolic curve of Figure 1.2.26 is as the following.

$$y = \frac{1}{2K} X^2 + I_o X$$

where: K = constant

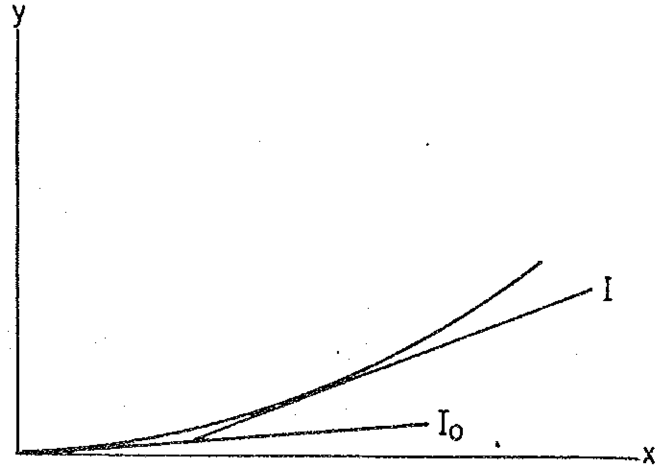


Figure 1.2.26 Sag Vertical Curve of Parabolic Line

The grade of I is the following:

$$I = \frac{X}{K} + I_o$$

If R is a radius of curvature at optional point,

$$R = \frac{\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{3/2}}{\frac{d^2y}{dx^2}} = K\,(1 + I^2)^{3/2}$$

When this curve is used as the vertical curve, I^2 is extremely small, hence;

$$R \fallingdotseq K$$

$$\frac{X}{I - I_o} = K \fallingdotseq R$$

From this relation, the value dividing the distance between two points by the algebraic difference in grades, is constant, and a constant of K is approximately a radius of curvature.

On the actual vertical curve, the above-mentioned value of X is the length of vertical curve of Lr, and I and Io are grades of both tangents. The radius of vertical curvature of R is as the following;

$$R \fallingdotseq \frac{L_r}{I - I_o}$$

$$R = \frac{100\ L_r}{A} \qquad .....(15)$$

In this guideline, the radius of vertical curvature is used as the regulatory parameter of vertical curve. However, the radius was determined based on the calculation for necessary curve length. It is easier to calculate the necessary curve length than the minimum radius of curvature, taking various factors into consideration. In practice, however, radius of curvature can be used more easily than curve length.

The length of curvature can be expressed as follows;

$$Lr = K^a \cdot A$$

Comparing the above equation with Equation (15)

$$K^a = R/100$$

$$R = 100 \cdot K^a$$

Thus, radius of curvature can be easily calculated from K^a , which are shown in Figure 1.2.27 and 1.2.28.

b) Calculation of the length of vertical curve

The necessary length of vertical curve is mainly determined from the followings:

- the necessary length for rider comfort
- the necessary length for securing the stopping sight distance
- the necessary length from the visual point of view.

The necessary length for rider comfort

Consideration of motorists' comfort requires that the rate of change of grade be kept within tolerable limits. It is necessary for motorists' comfort to insert the vertical curve into the changing point of tangent grades. This consideration is most important in sag vertical curves where gravitational and vertical centrifugal forces act in the same direction.

As the length for comfort, in general the following formula is known empirically.

$$L = \frac{A\ V^2}{360}$$

where, L = length of vertical curve, m;
V = design speed, Km/h; and
A = algebraic difference in grade, percent

Table 1.2.39 shows the lengths of vertical curve for comfort according to the design speed.

Table 1.2.39 Necessary Lengths of Vertical Curve for Rider Comfort

Design Speed (km/h)	120	100	80	60	50	40	30	20
Length of Verticle Curve (m)	40.0A	27.8A	17.8A	10.0A	7.0A	4.4A	2.5A	1.1A
Radius of Vertical Curvature (m)	4,000	2,780	1,780	1,000	700	440	250	110

The necessary length for securing the stopping sight distance

Minimum lengths of crest vertical curves as determined by sight distance requirements generally are satisfactory from the standpoint of safety, comfort, and appearance. On the sag vertical curves, securing the sight distance need not be considered. However, when the crossing structures, such as an overbridge, are constructed to the very limit of vertical clearance, there is a fear of obstructing sight distance. Therefore, the length of crest vertical curve is determined by securing the sight distance, while the length of sag vertical curve is determined by crossing structures.

In AASHTO standards, on sag vertical curves, headlight sight distance is recognized as one of the four criteria for determining lengths.

i) Crest vertical curve

The basic equation for length of a parabolic vertical curve in terms of algebraic difference in grade and sight distance are as follows:

When S is less than L,

$$L = \frac{AS^2}{100\ (\sqrt{2h_1} + \sqrt{2h_2})^2} \qquad \ldots(16)$$

When S is greater than L,

$$L = 2S - \frac{200\ (\sqrt{h_1} + \sqrt{h_2})^2}{A} \qquad \ldots(17)$$

where: L = length of vertical curve, m;
S = sight distance, m;
A = algebraic difference in grades, percent;
h_1= height of eye above roadway surface, m; and
h_2= height of object above roadway surface, m.

When the height of eye and the height of object are 1.2 m and 0.1 m, respectively, as used for stopping sight distance,

Where S is less than L,
$L = AS^2/398$ ...(18)

Where S is greater than L,
$L = 2S - 398/A$ ...(19)

The differences from AASHTO standard s are the height of eye and object of the stopping sight distance. The required lengths of vertical curves calculated from Equations (18) and (19) for different values of A for each design speed are shown in Figure 1.2.27. The solid lines give the required lengths, on basis of rounded values of K as determined from these equations. The chain line for K = 30.2 gives unrounded values for 80 Km/h for comparison.

The curve shown by a broken line, crossing lines of required curve length, indicates where S = L. Note that to the right of the S = L line, the value of K, or length of vertical curve change in A, is a simple and convenient expression of the design control. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a design value tabulation. The selection of design curves is facilitated because the required length of curve in meters is equal to K times the algebraic difference in grades in percent, L = KA. Conversely, the checking of plans is simplified by comparing all curves with the design K value.

Table 1.2.40 shows the computed K values for lengths of vertical curves as required for rider comfort and stopping sight distances, Table 1.2.28, for each design speed. For direct use in design, values of K are rounded as shown in the right column. The rounded values of K are plotted as the solid line in Figure 1.2.27.

Where S is greater than L (lower left in Figure 1.2.27), computed values plot as a curve (as shown by the broken line for 80 Km/h) that curves to the left, and for small values of A the required lengths are zero because the sight line passes over the apex. This relation does not represent desirable design practice.

The above values of K derived where S is less than L also can be used without significant error where S is greater than L. As shown in Figure 1.2.27, extension of the diagonal lines to meet the vertical lines for minimum lengths of vertical curves result in appreciable differences from the theoretical only where A is small.

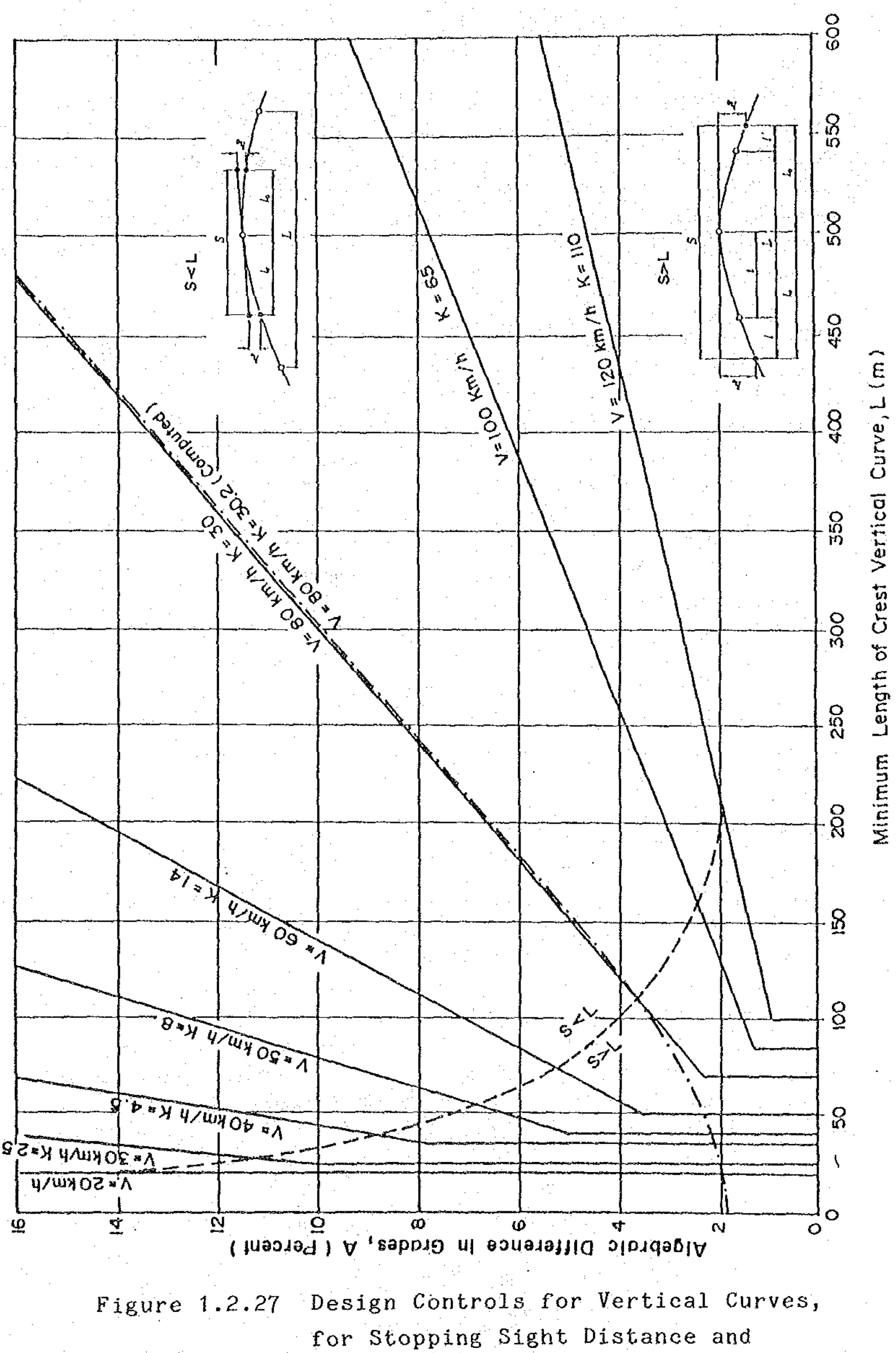


Figure 1.2.27 Design Controls for Vertical Curves, for Stopping Sight Distance and Open Road Conditions

Table 1.2.40 Length of Crest Vertical Curve

Design Speed (km/h)	Rider Comfort $L = \frac{AV^2}{360}$	Stopping Sight Distance $L = \frac{AS^2}{398}$	Length of Crest Vertical Curve (Rounded) K.A
120	40.0 A	111.0 A	110.0 A
100	27.8 A	64.5 A	65.0 A
80	17.8 A	30.2 A	30.0 A
60	10.0 A	14.1 A	14.0 A
50	7.0 A	7.6 A	8.0 A
40	4.4 A	4.1 A	4.5 A
30	2.5 A	2.3 A	2.5 A
20	1.1 A	1.0 A	1.0 A

A = algebraic difference in grades, percent;

S = sight distance, m;

V = design speed, Km/h.

ii) Sag vertical curve

When a crossing structure such as an overbridge is constructed to the very limit of vertical clearance, there is a fear of obstructing sight distance. The length of sag vertical curve is determined by taking the obstruction of sight distance into consideration.

The following equations show the relation between S, L and A :
Where S is less than L,

$$L = \frac{AS^2}{100\left\{\sqrt{2(c-h_e)} + \sqrt{2(c-h_o)}\right\}^2} \qquad \ldots(20)$$

Where S is greater than L,

$$L = 2S - \frac{400C}{A}\left\{1 - \frac{h_e + h_o}{2C} + \sqrt{\left(1 - \frac{h_e}{C}\right)\left(1 - \frac{h_o}{C}\right)}\right\} \qquad \ldots(21)$$

Where the height of eye, C, and the heights of objects, he and ho, are 4.5m, 1.5m and 0.75m, respectively, as used for stopping sight distance, Where S is less than L,

$$L = \frac{AS^2}{2692} \qquad ...(22)$$

Where S is greater than L,

$$L = 2S - \frac{2692}{A} \qquad ..(23)$$

On the sag vertical curve, also, the value of L obtained from the Equation (22) is larger than from the Equation (23), so that the length of sag vertical curve is determined by the Equation (22).

Table 1.2.41 shows K values for lengths of sag vertical curves. The design K values for lengths of sag vertical curves are determined by rider comfort. The rounded design values of K are plotted as the solid line in Figure 1.2.28.

Table 1.2.41 Length of Sag Vertial Curve

Design Speed (km/h)	Rider Comfort $L = \frac{AV^2}{360}$	Sight Distance Under Overbridge $L = \frac{AS^2}{2692}$	Length of Crest Vertical Curve (Rounded) K.A.
120	40.0 A	16.4 A	40 A
100	27.8 A	9.5 A	30 A
80	17.8 A	4.5 A	20 A
60	10.0 A	2.1 A	10 A
50	7.0 A	1.1 A	7 A
40	4.5 A	0.6 A	4.5 A
30	2.5 A	0.3 A	2.5 A
20	1.1 A	0.1 A	1.0 A

c) The necessary curve length from the visual point of view

When the difference in grades is small, the necessary lengths for rider comfort and sight distance are very short. However, as the profile of short vertical curves is seen as a broken gradeline by driver, it is necessary to provide the vertical curve of which the length is greater than a certain extent. This necessary length is determined from the visual point of view, and the length is considered to be in proportion to

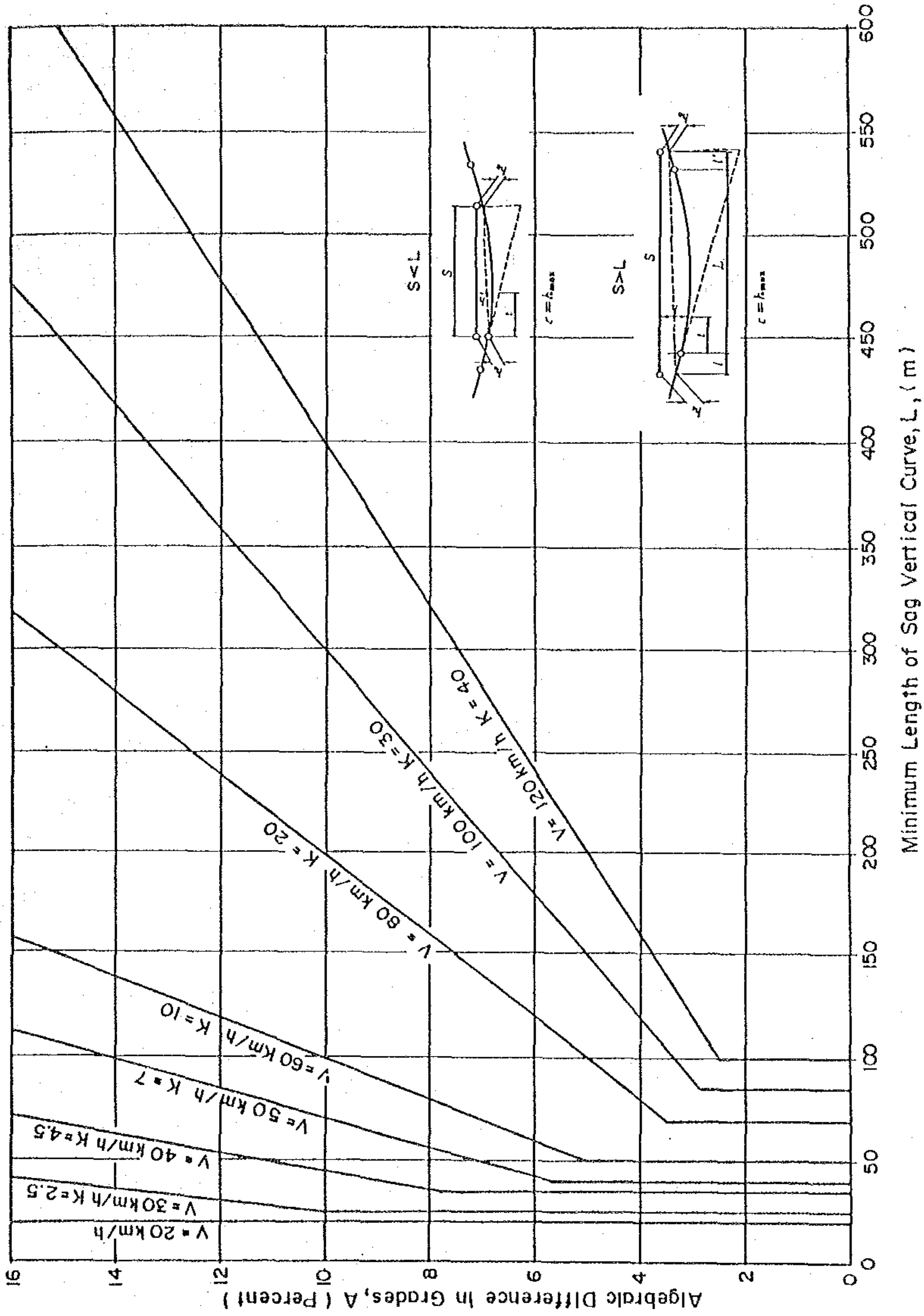


Figure 1.2.28 Design Controls for Sag Vertical Curves

the running speed. In the Japanese standard, this minimum length of vertical curve is empirically determined as the distance which a vehicle of design speed covers by running for 3 seconds.

2) Desirable value of radius of vertical curvature

1. It is desirable that the radii of vertical curvature are not less than the values shown in Table 1.2.42, according to the design speed.

Table 1.2.42 Desirable Value of Radius of Vertical Curvature

Design Speed (km/h)	Radius of Vertical Curvature (m)	
	Crest	Sag
100	10,000	4,500
80	4,500	3,000
60	2,000	1,500
50	1,200	1,000
40	700	700
30	400	400
20	200	200

The values shown in Table 1.2.37 are the minimum values of radius of vertical curvature. Those values should be used when conditions so demand. Therefore, in practical design, it is desirable that the radius of vertical curvature is larger than the value shown in Table 1.2.42 which is 1.5 to 2.0 times the minimum radius.

(12) Cross slope

1. On the carriageway and the shoulder which is set beside the carriageway, the cross slope should be provided as the standard of value shown in Table 1.2.43, according to the pavement surface types.

2. On the sidewalk, a 2 percent cross slope should be provided as the standard value.

Table 1.2.43 Cross Slope

Surface Type	Cross slope (%)
Cement Concrete Pavement and/or Asphalt Pavement	1.5 - 2.0
Others	3.0 - 5.0

The cross slope of pavement surface is necessary for draining rain water to gutters and drainages. Two-lane and wider undivided pavements on tangents or on flat curves have a crown or high point in the middle and slope downward toward both edges. The downward cross slope may be a plane or curved section or a combination of the two. With plane cross slopes there is a cross slope break at the crown line and a uniform slope on each side. Curved cross sections usually are parabolic, with a slightly rounded surface pavement edge. The advantage of the curved section lies in the fact that the cross slope is steeper toward the pavement edge, thereby facilitating drainage. The disadvantages are that curved sections are more difficult to construct, the cross slope of the outer lanes may be excessive, and warping of pavement areas at intersections may be awkward or difficult to construct.

In the Japanese standard, on multilane highways, the combination of two different plane cross slopes is proposed as shown in Figure 1.2.29. On the other hand, AASHTO prescribes that in some cases on multilane undivided highways, a rounded crown section be used for the central two lanes, with a plane section on the outer lanes having a cross slope rate of about 1.5 to 2 percent. Here the outer lane cross slope is made to be the same or slightly steeper than that at the end of the curved section.

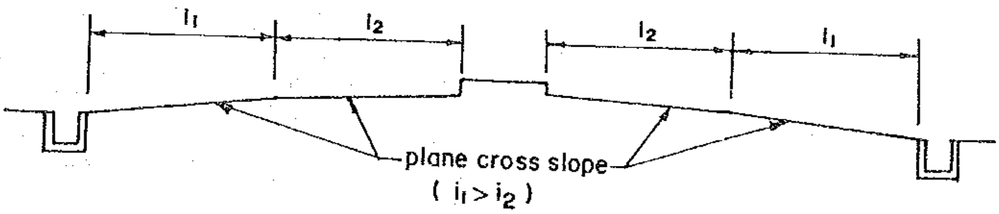


Figure 1.2.29 Combination of Two Plane Cross Slopes

Use of cross slopes steeper than 2 percent on high-type, high-speed pavements with a central crown line is not desirable. In passing maneuvers, drivers must cross and recross the crown line and negotiate a total rollover or cross-slope change of over 4 percent. The reverse curve path of travel of the passing vehicle causes a reversal in the direction of centrifugal force, which is further exaggerated by the effect of the reversing cross slope.

In areas of intense rainfall a somewhat steeper cross slope may be necessary to facilitate pavement drainage, even though undesirable from the operational point of view. In such areas the cross slope on high-type pavement may be increased to 2.5 percent.

When determining the cross slope, the following factors should be considered; types of vehicles, meteorology, alignments of road, grades, and surface types. The standard value of cross slope may be the value shown in Table 1.2.44 according to the surface type and the number of lanes which take a great effect on drainage.

Table 1.2.44 Standard Value of Cross Slope

Surface Type	Cross Slope (%)	
	two lane highway	more than two lane
Cement Concrete Pavement and/or Asphalt Pavement	1.5	2.0
Others	3.0 - 5.0	

On divided roads, two types of cross section are shown in Figure 1.2.30. Carriageways that slope in only one direction, as shown in Figure 1.2.30(a), are more comfortable to drivers because vehicles tend to be pulled in the same direction when changing lanes. Carriageways having a unidirectional slope may drain away from the median. Drainage away from the median may effect a savings in drainage structures and simplify treatment of intersecting streets.

A cross section with each carriageway crowned separately, as shown in Figure 1.2.30(b), has an advantage in rapidly draining the pavement during rainstorms. In addition, the difference between high and low points in the cross section is kept to a minimum. Disadvantages are that more inlets and underground drainage lines are required, and treatment of at-grade intersections is more difficult because of the several high and low points on the cross section. Use of such sections preferably should be limited to regions of high rainfall.

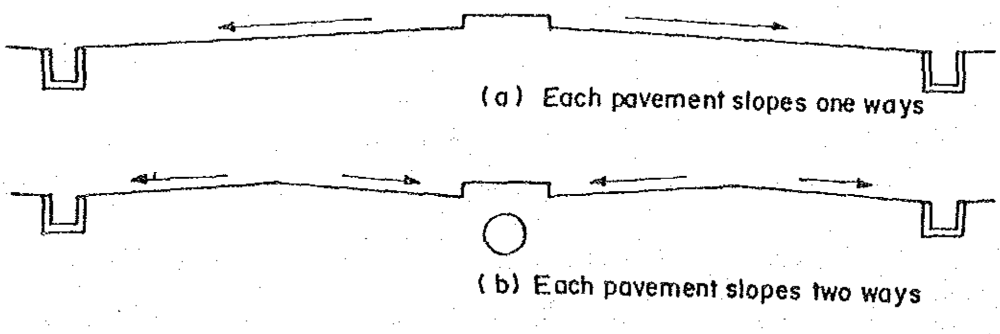


Figure 1.2.30 Basic Cross Slope Arrangements for Divided Road

1.2.6 At-Grade Intersection

(1) Introduction

1) Planning and design of at-grade intersection

a) Significance of planning and design of at-grade intersection

1. Planning of at-grade intersection must be done taking into account the following two aspects; i) the role of the intersection in the entire road network, and ii) the integrity of various conditions of road sections and related intersections.

An intersection is an important part of a highway because, to a great extent, the efficiency, safety speed, cost of operation, and capacity depend on its design.

Four basic elements enter into design considerations of at-grade intersections.

i) Human factors

These include driving habits, ability to make instant decisions,, driver expectancy, decision and reaction time, conformance to natural paths of movement, and pedestrian use and habits.

ii) Traffic considerations

These include design and actual capacities, design-hour turning movements, size and operating characteristics of vehicle, variety of movements (diverging, merging, weaving, and crossing), vehicle speeds, transit involvement, and accident experience.

iii) Physical elements

These include character and use of abutting property, vertical alignments at the intersection, sight distance, angle of the intersection, conflict area, speed-change lanes, geometric features, traffic control devices, lighting equipment, and safety features.

iv) Economic factors

These include costs of improvements, effects of controlling or limiting right-of-way on abutting residential or commercial properties where channelization restricts or prohibits vehicular movements, and energy consumption.

b) Design hourly volume and stage construction

1. As a general rule, the structural design of intersection should be made based on the design hourly volume of the roads crossing the intersection. If the original traffic volume at the intersection when construction starts is considerably less than the design hourly volume, the first stage construction can be done based on the estimated volume projected for five to ten years after the intersection is open to traffic. In this case, due attention shall be paid to construction steps, land acquisition, obstacle to construction, etc. for the phased construction from the second up to the last stage.

It is difficult to estimate the long-term traffic volume correctly. Therefore it is appropriate to plan by the predictable estimated traffic volume for five to ten years after. However, in this case, consideration of future requirements should be as reliable as possible, taking into account such factors as heavier volumes than the design hourly volume.

2) Geometric design and traffic control

1. Due attention shall be paid to the geometric design of at-grade intersections so that they will be fully coordinated with traffic signals and traffic control devices.

2. The traffic control as the basis of the design of an at-grade intersection shall be that the through traffic should not be controlled by a stop sign for the road with the design speed of more than 60 km/h.

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

Control of high - speed through traffic by stop signs is totally contrary to good driving practice, confuses the traffic flow, and may cause accidents. Therefore, on a road with the design speed of more than 60 Km/h, through traffic should not be controlled by stop signs.

3) Design vehicle, passage method, and design speed

a) Design vehicle and passage method

1. The combination of design vehicle and passage method shall be carefully determined by comprehensive judgment on the characteristics of road and traffic, their functions, areal characteristics, roadside conditions and pedestrians, etc.

The selection of any one of the designs depends on the types and sizes of vehicles that will be turning and to what extent they should be accommodated. In turn, these elements may depend on other factors such as the type, character and location of the intersecting roads and traffic volumes thereon, the number and frequency of the larger units involved in turning movements, and the effect of these larger vehicles on other traffic.

b) Design speed

1. The design speed of the approach legs of the intersection for through traffic shall be the same as that of the mid-block section. However, it may be reduced by 10 - 20 km/h under certain conditions.

The design speed of the approach legs of the intersection for through traffic should be the same as that of the mid-block section. However, when the relation of priority between major highway and minor highway is clear, it is recommended that the design speed of the approach legs of the minor highway be lower than that of the mid-block section. The terms "major highway" and "minor highway" are used here to indicate the relative importance of the highways through the intersection rather than the functional character of the highways. The terms "major highway" and "minor highway" are different from "Major Trunk Roads", "Major Roads" and "Minor Roads" in the classification of road.

Specially, it would be better to reduce the design speed of the approach egs, to insert a curved line into the approach and to change the intersecting angle nearly to a right angle, than to intersect at an acute angle with high design speed.

The speed for each point of the approach leg of the intersection can be obtained from the equation (24) substituting the value of acceleration or deceleration shown in Table 1.2.45 taking the speed change into consideration.

Table 1.2.45 Value of Acceleration and Deceleration Considering Change of Speed

(Unit : m/s^2)

		Acceleration	Deceleration
Urban Area		1.5	-3.0
Rural Area	Major highway	1.0	-2.5
	Minor highway (subordinate)	1.5	-3.0

Note : . The value of acceleration is decreased 0.1 m/s^2 per a upgrade of 1%.

. The value of deceleration is not changed on grades

$$L = \frac{1}{2 \times 3.6^2 \times \alpha}(V^2 - V_o^2) \quad .....(24)$$

where

L = running distance; in m,

α = acceleration or deceleration; in m/s^2,

V = ultimate speed; in Km/h

Vo = initial speed; in Km/h

(2) Configuration and interval of at-grade intersection

1) The number of legs and intersecting angle

1. An intersection shall be designed so as not to have more than five intersecting legs, except when it is located in a special place like in front of a station.

2. At an intersection where two or more roads join or intersect at grade, if necessary, a turning lane, speed change lane or traffic island is to be provided, and the corners of intersection shall be cut to ensure unobstructed sight.

a) The number of legs

It is recommended that an intersection has no more than four legs.

- It should not be planned so that a new road enters directly into an existing 4-leg at-grade intersection.
- When the newly planned road is forced to intersect with the existing at-grade intersection, it is necessary to improve the existing roads by replacement and/or realignment.

b) Intersecting angle

1. It must be planned that each intersecting angle at an at-grade intersection has a right and/or near right angle.

At a right or near - right angle intersection, the distance crossing the intersecting road is short, and the intersecting area is small. It is also desirable from the viewpoint of visibility. Therefore, the intersecting angle should be more than 75°. But, where variation from this is unavoidable, the angle may be more than 60°.

The practice of realigning a road by the following method, that is intersecting at acute angles, has proved to be effective.

- The realignment of the intersecting angle is made the target for the minor highway as shown in Figure 1.2.31.

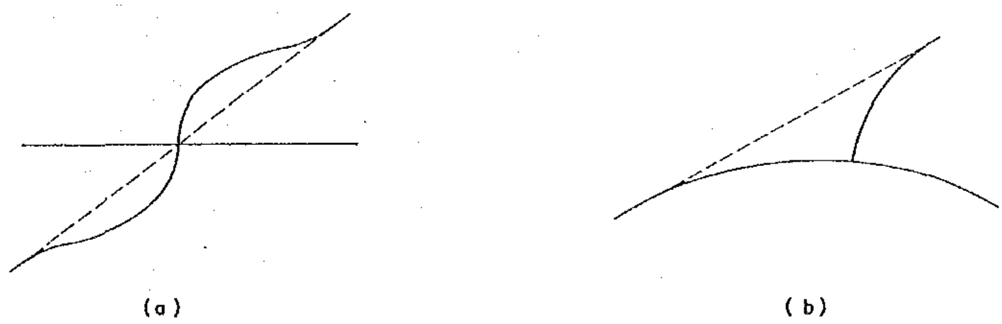


Figure 1.2.31 Realignment of Intersecting Angle

- Where a major highway is curving and subordinate highway is curving and a subordinate highway constitutes an extension of one tangent, realigning the subordinate highway is advantageous, as shown in Figure 1.2.32.

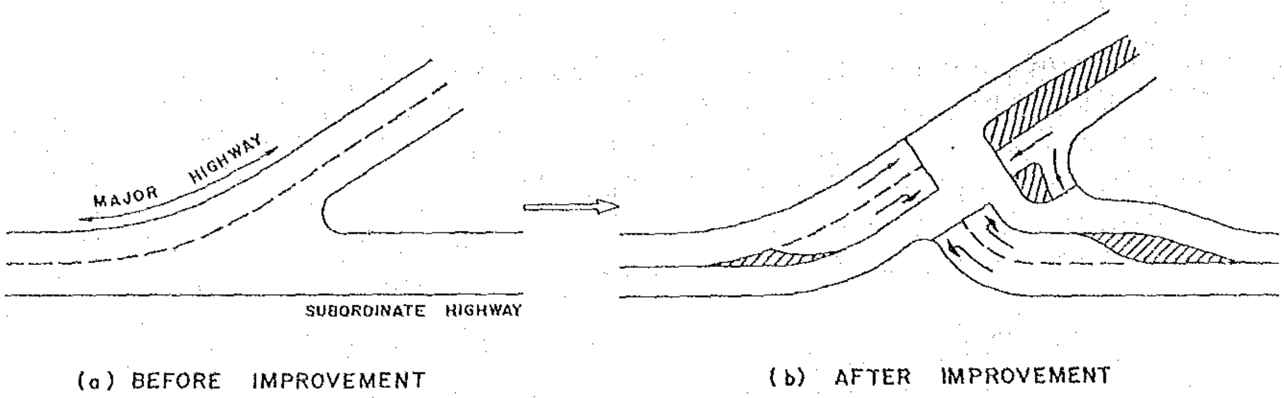


Figure 1.2.32 Connecting a Subordinate Highway at a Y Intersection

2) Configuration of at-grade intersection

1. As a general rule, planners should avoid creating offset intersections or dog-leg intersections.

2. The alignment of the main traffic flow at an intersection shall be so designed that it can secure a straight line as much as possible, and that it is not crossed by two or more intersecting legs.

3. Generally, a intersecting angle at an intersection should be a near right angle.

a) Irregular intersection

- When main traffic is right- or left- turn traffic, it is desirable for traffic management and traffic safety that the alignment of main traffic is improved as shown in Figure 1.2.33(b).

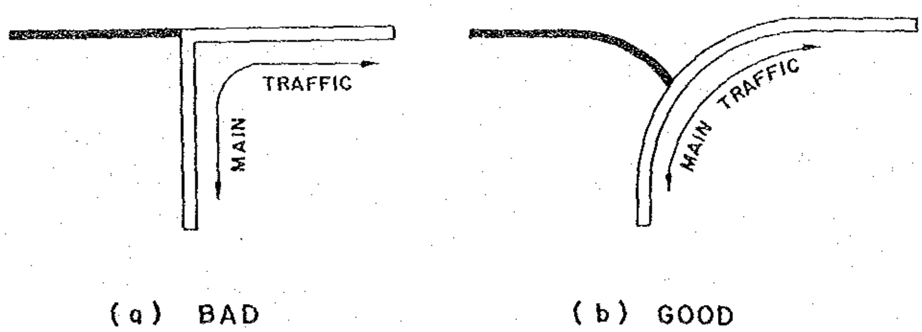
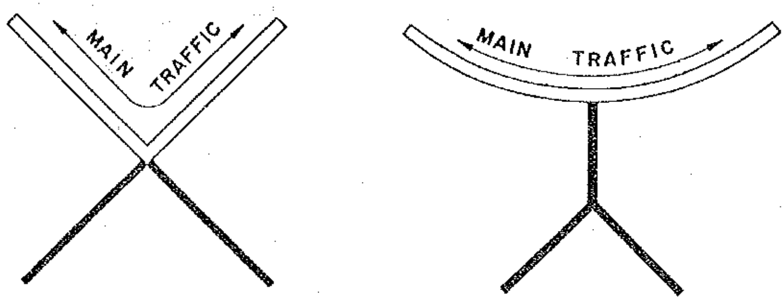


Figure 1.2.33 Improvement of T Intersection

- When more than two approach legs are intersecting on one side of the flow of main traffic, minor highways should be combined as shown in Figure 1.2.34(b).



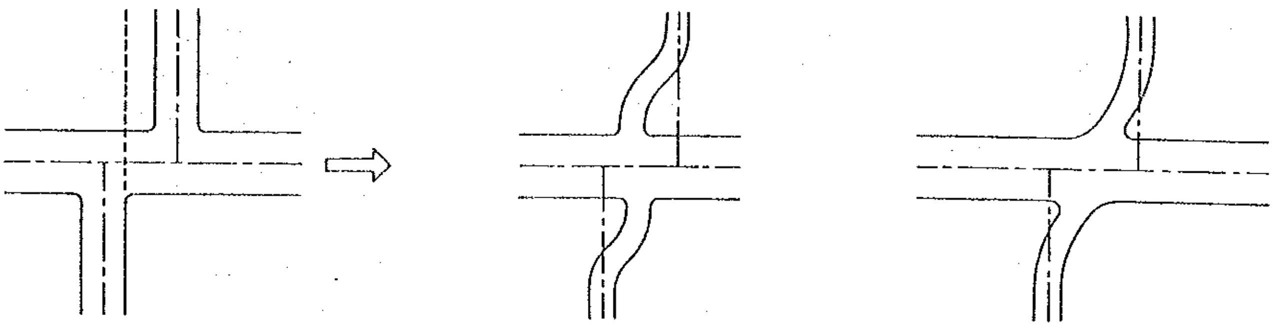
(a) BAD (b) GOOD

Figure 1.2.34 Improvement of Crossroads Intersection

b) Deformed intersection

Offset intersections might be regarded as two T intersections located very close to each other. At such an intersection, the intersection area is very large, and running path of each traffic flow is complicated. In many cases, vehicular traffic flow so conflicts with pedestrian crosswalking that it is necessary to take a signal arrangement into consideration because not to do so may cause undesirable results in traffic safety and operational ability.

- It is desirable that the configuration of an offset intersection be improved as shown in Figure 1.2.35.



(a) BEFORE IMPROVEMENT (b) AFTER IMPROVEMENT (c) AFTER IMPROVEMENT

Figure 1.2.35 Improvement of Offset Intersection

- In an unavoidable case of offset intersection, only left turning traffic should be permitted or right turning and right crossing of main traffic should be regulated physically by providing medians. In this case, by providing an opening in the median, right turns can be made for U-turns as shown in Figure 1.2.36.

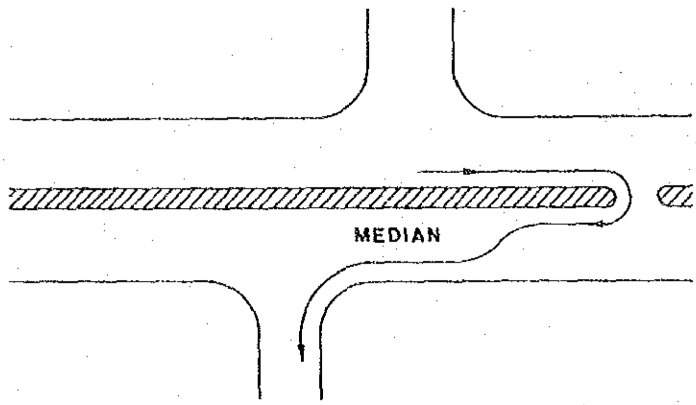


Figure 1.2.36 Regulation by Medians.

c) Widening of crossroad

Examination of the number of lanes, including provision of a right-turn lane, is apt to be made the target for major highways. However, when there are so many cases that the green time of a subordinate road is high ratio in signal phase, this green time can be reduced by adding one entrance lane or more on the subordinate road (Figure 1.2.37). In this case, the following should be taken into consideration regarding the green time of subordinate road.

- The green time should not be less than 15 seconds.
- The volume of the pedestrians and time they require to cross the major highway, who use the same green time, should be taken into account.

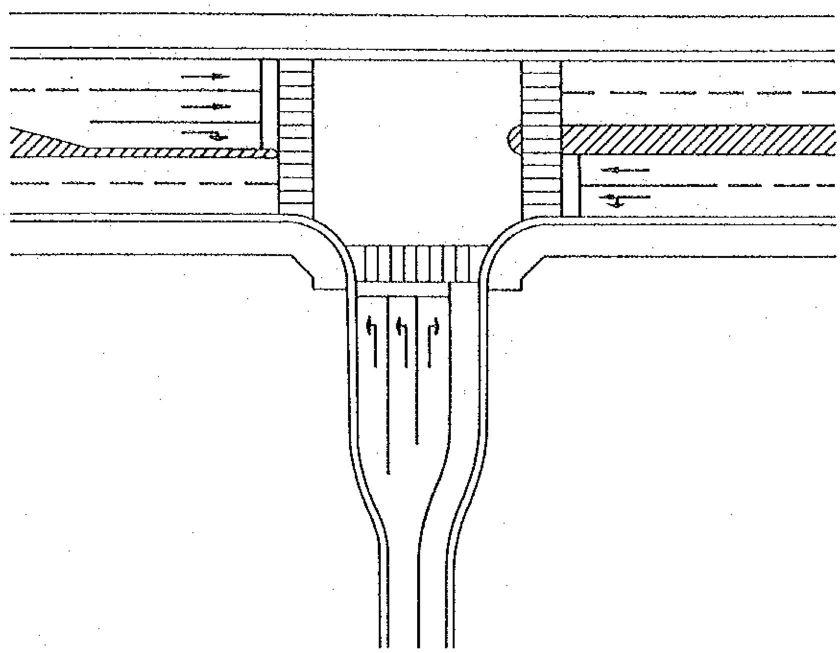


Figure 1.2.37 Widening of Crossroad

3) Interval between intersections

1. Intervals between intersections are recommended to be as long as needed in order to ensure proper traffic control.

a) Restriction on interval between intersections

The minimum interval between intersections is restricted by the following.

- Weaving length
- Storage length of signal control
- Length of right turn lane

i) Restriction by weaving length

The interval between intersections is restricted by the weaving length, in all the cases where the weaving flows occur as shown in Figure 1.2.38. When one of the weaving flows is a main traffic flow, there are many cases that cause problems from aspects of both traffic safety and operational ability. No single generally method of calculating the required interval between intersections has been established. The following equation is one of the standards that includes a high safe margin, obtained by estimating the maximum weaving traffic volume.

Required interval of intersection (inside measurement) (m)
= design speed (Km/h) x the number of lanes for one direction x 2 ...(25)

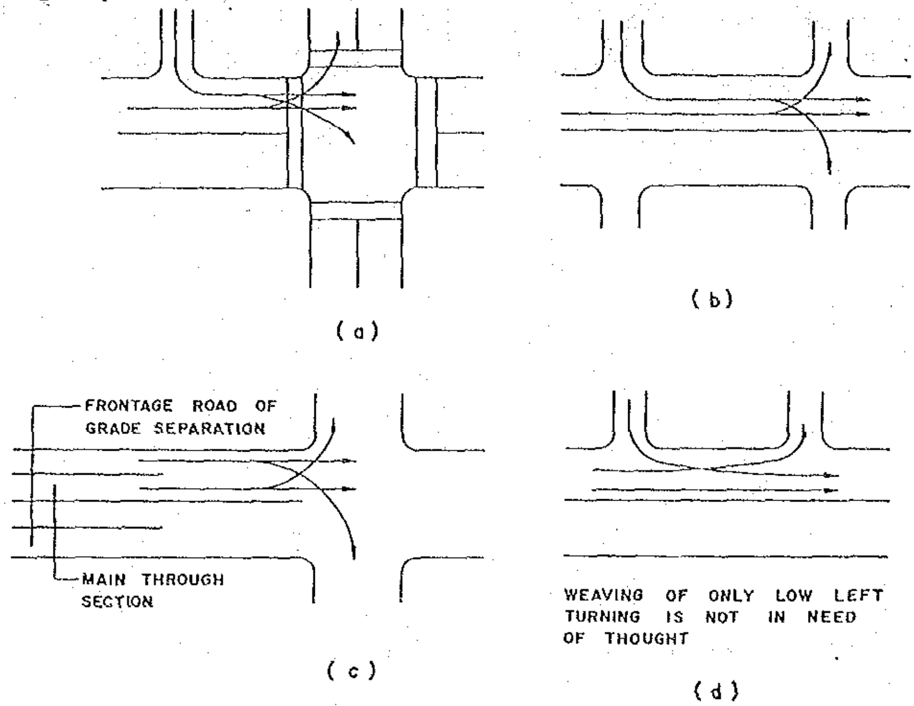


Figure 1.2.38 Examples of Weaving between Sequential Intersections

ii) Restriction by storage length of signal control

It is necessary to make the interval between intersections long enough so that the vehicles stopped at an intersection by a signal control do not block the intersection behind.

Generally, there are few cases where the interval between intersections is restricted by the storage length of main traffic flow on the assumption that the traffic signals of two neighboring intersections are controlled systematically at the nearly same time. However, the interval between intersections is restricted frequently by the storage length of the minor traffic flow merging into the main flow after turning right or left.

iii) Restriction by length of right turn lane

Figure 1.2.39 shows an example where the interval between intersections is restricted by the length of right turn lane. The minimum interval between intersections is determined by the design right turning volume per one cycle of signal. However, a uniform minimum interval can not be stipulated.

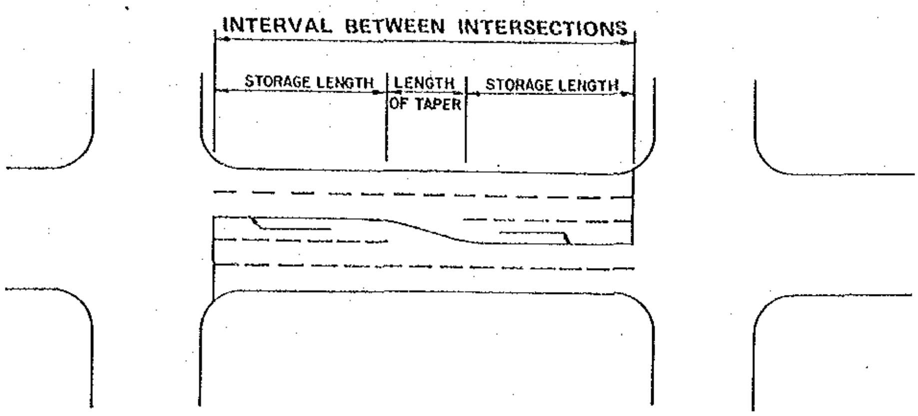


Figure 1.2.39 Restriction by Length of Right Turn Lane

b) Treatment of small roads (Soi) crossing a major highway

One problem related to the interval between intersections in planning a major highway is that of the many small intersections resulting when a major highway is to cross existing small roads. As for this point, it is desirable to arrange the following treatment.

- Small roads are first to be connected to minor highways or a road which collects some of small roads and then crosses the major highway. To be avoided is an intersection plan in which not only small roads connect with major highway directly but also collection and distribution of traffic are managed independently. (Figure 1.2.40)

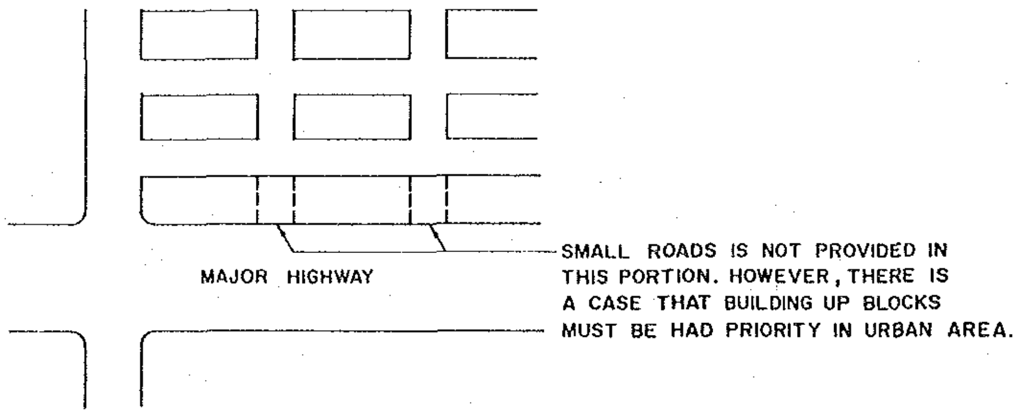


Figure 1.2.40 Treatment of Small Roads

- In the unavoidable case when a small road is to connect in the neighborhood of a main at-grade intersection, the exit or entrance of the small road should be restricted to only a left turn, while right turning from major road and right turning from small road should be regulated physically by providing a median, or by other measures . (Figure 1.2.41)

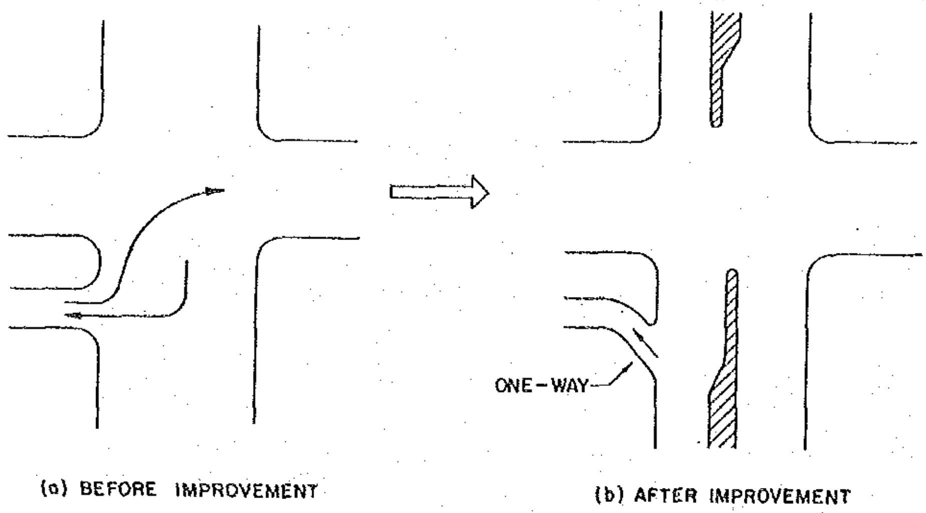


Figure 1.2.41 Example of Treatment of Neighboring Small Intersection

- When planning a major highway, to at-grade intersections where the major highway is to cross the existing road network, not only configurations but also intervals between intersections should be examined. If necessary, realignment and rearrangement of existing roads, and traffic regulation, may be examined.

(3) Alignment in the neighborhood of an intersection

1) Sight distance and visible distance of intersection

1. In order that vehicles can pass through an at-grade intersection safely and easily, intersection, signal, road sign, etc., must be clearly seen. The distances at which either a signal at a signalized intersection or a stop sign at a stop controlled intersection is visible, should not less than the values shown in Table 1.2.46, according to the classification and design speed of road.

Table 1.2.46 Visible Distance at an At-grade Intersection

Design Speed (Km/h) \ Minimum Visible Distance	Signal Control		Stop Control
	Suburban Rural	Urban	
80	350	-	-
60	240	170	105
50	190	130	80
40	140	100	55
30	100	70	35
20	60	40	20

The following must be considered from the viewpoint of sight distance, in order that vehicles approaching an intersection can pass through it safely and smoothly.

- The signal and road sign at the intersection be clearly seen at the distance from the intersection as shown above.
- In the intersection, the necessary visibility is assured.

In this guideline, use of the former criterion is prescribed.

a) Signalized intersection

The minimum sight distance of signal sight is the sum of two distances; the distance traversed by the vehicle from the instant the driver sights a stop signal to the instant the brakes are applied, and the distance required to stop the vehicle without feeling uncomfortable from the instant brakes are applied. In this guideline, two kinds of total reaction times, at urban areas and at rural areas are determined reffering to existing reports. They are 6 sec. in an urban area, and 10 sec. in a rural area. The total reaction time in an urban area is shorter than in the rural area because in urban areas there are many intersections and drivers tend to know the existence of the signals. The deceleration at which there is no feeling of discomfort is 0.2 g.

The minimum visible distance can be obtained from the following equation (26) according to the design speed.

$$S = \frac{vt}{3.6} + \frac{1}{2\alpha}\left(\frac{v}{3.6}\right)^2 \qquad .....(26)$$

Where;

S = minimum visible distance in m;
V = design speed, in Km/h;
t = total reaction time, in sec;
α = deceleration, in m/s^2;

By substituting, $t=10s$, $\alpha=1.96m/s^2(=0.2g)$ in a rural area, and $t=6s$, $\alpha=1.96m/s^2$ in an urban area into the above equation, the minimum visible distances can be obtained as shown in Table 1.2.46 according to the design speed.

b) Stop controlled intersection

A Stop controlled intersection shall be designed so that the minor roadway be posted with stop signs as its legs intersect the major roadway. This stop signs must be visible from the distance required to stop the vehicle without feeling discomfort, from the instant the driver sights a stop sign. In the AASHTO standard, the amount of time necessary to start deceleration is the driver's perception and reaction time, which, for intersection design, can be assumed to be 2.0 sec. The deceleration for which there is no discomfort is 0.2g, the same as for a signalized intersection noted immediately above. The minimum visible distance for a stop sign is obtained according to the design speed by substituting, $t = 2$ sec, $\alpha = 1.96\ m/s^2(= 0.2\ g)$ into the equation (26). (Table 1.2.46)

When the minimum distance can not be ensured, it is necessary to provide warning signs.

2) Radius of curvature

1. The curvature radius of a carriageway's center line at the approach of an intersection should not be less than the value shown in Table 1.2.47, according to the method of control used at the intersection and the design speed of the road concerned.

Table 1.2.47 Minimum Radius of Curvature at the Approach of Intersection

Design Speed (Km/h) \ Minimum Radius of Curvature (m)	Major highway of signalized and stop controlled intersection		Minor highway of stop controlled intersection
	Standard Value	Exceptional Value	
80	280	230	-
60	150	120	60
50	100	80	40
40	60	50	30
30	30	-	15
20	15	-	15

In order to secure visibility at an intersection, the alignment of the intersection is more restricted than that of a mid-block section. The prescriptions of the minimum radius of curvature and the grade of the mid-block section must be kept at the lowest levels for the intersection. When the nearly minimum values of these levels are adopted at an intersection approach, it is desirable to post signals and warning signs announcing previously the existence of an at-grade intersection.

3) Vertical alignment

1. The grade at the approach of an intersection and its extension should be designed as flat as practically possible.
2. It is desirable that an intersection shall be designed so as not to be located in the neighborhood of the top or bottom of a vertical curve.

The grades of intersecting highways should be as flat as practical on those sections that are to be occupied by stopped vehicles, sometimes referred to as storage platforms. Grades in excess of 2.5 percent should be avoided on intersecting highways. In the AASHTO standard, this value is prescribed as 3.0 percent. The minimum section length of gentle grade should be ensured. This is obtained from the product of the number of stopped-vehicles per lane per cycle multiplied by the average space headway. When the minimum length can not be ensured due to certain restrictions, such as topographical reasons, the lengths should not be less than the values shown in Table 1.2.48.

Table 1.2.48 Minimum Length of Gentle Grade Section in the Neighborhood of an Intersection

Classification of Road		Minimum Section Length (m)
Rural, Suburban	Urban	
Major Trunk Roads Major Roads	Major Trunk Roads	40
Minor Roads	Major Roads	35
-	Minor Roads	15
Access Roads	-	10
-	Access Roads	6

(4) Cross section in the neighborhood of an intersection

1. When turning lane and/or speed change lane are provided at an intersection, the width of a lane other than a turning or speed change lane may be reduced to 3.0 m on Major Trunk Roads and Major Roads in urban areas. On other roads the width may be reduced to 2.75 m.

2. The standard width of a turning lane and speed change lane is 3.0 m.

3. When a turning lane and/or speed change lane is/are provided at an intersection, transition runoff should be provided as appropriate, according to the design speed.

1) Lane width and the number of lanes

1. Lane width should have the value shown in Table 1.2.49, according to the classification of road.

Table 1.2.49 Lane Width at an At-grade Intersection (Unit:m)

Type (Area)	Class	Lane width of mid-block section	Width of Through Lane in the Section Provided Added Lane	Width of Added Lane
Urban	1. Major Trunk Roads	3.5	3.5 or 3.25	3.25, 3.0,
	2. Major Roads	3.25 [3.5]	3.25 or 3.0	or 2.75
	3. Minor Roads	3.0	3.0 or 2.75	(2.5)
Suburban Rural	1. Major Trunk Roads	3.5	3.5	
	2. Major Roads	3.25 [3.5]	3.25 [3.5]	
	3. Minor Roads	3.0	3.0	
	4. Access Roads	3.0	2.75	

[ ] : Only when necessary

( ) : In the unavoidable case, reduced on the right turn lane in the urban area

a) Lane width

When the added lane or the through lane is added at an at-grade intersection, if it is very difficult to acquire land in the neighborhood of urban streets, the width of the through lane may be reduced by 0.25 m provided that the design speed of the approach legs of the intersection may be reduced by 20 km/h. In such case, it is desirable to provide an added lane by reducing the widths of through lanes, stopping lanes, and medians at an intersection. The standard width of an added lane is 3.0 m. However the width of a right turn lane in an urban area may be reduced to 2.5 m, when the large-sized vehicle ratio is low and the abovementioned width can not be ensured by reducing the width of sidewalk, etc.

b) Widening for right turn lane

When the width of a right turn lane cannot be ensured on the existing roads by several restrictions, separation of right turning vehicles plays so important a part in traffic management at an intersection, that it is adequate to widen merely 1.5 m or more, not marking a boundary line with through lane for the right turn lane. (Figure 1.2.42)

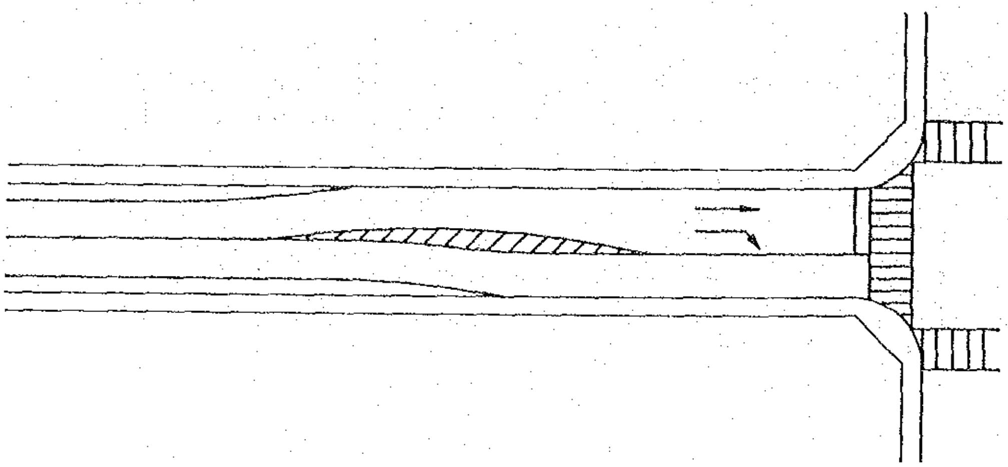


Figure 1.2.42 Widening for Right Turn Lane

c) The number of lanes

As a general rule, the number of lanes at an exit of an intersection must be the same as the number of through lanes at its approach. This number of through lanes is equal to total number of lanes minus the number of right- and left- turn lanes.

When it is necessary to reduce the number of through lanes at an intersection exit to be less than at its approach, the number of through lanes at an approach should have been previously decreased. In this case, extra through lanes at the approach may be used for the turning lanes.

Shift of lane within an intersection must be avoided by laying out the lanes at the exit section on an extension line of through lanes at the approach section. In case the through lane at the approach is shifted as shown in Figure 1.2.43 in order to provide the right turn lane, the lane at the exit should be laid out as an extension of the shifted through lane at the approach.

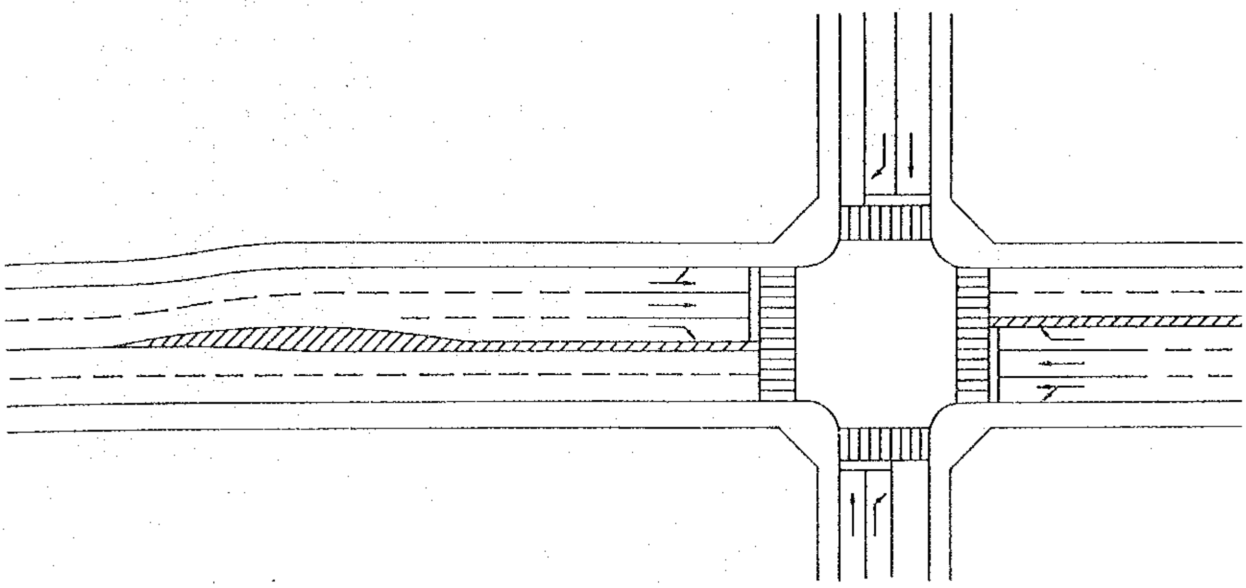


Figure 1.2.43 Through Lane at an Exit Section Correspoing to Shifted Lane at an Approach

2) Lane shift

The section length of lane shift for providing the mid-block added lane at an at-grade intersection is to be determined according to the design speed, the urban or rural area, and the horizontal alignment.

On the mid-block section the standard section length of transition runoff for addition or reduction in number of lanes, see the section 1.2.5, (8), 2).

a) Tangent section

When lane shift is adopted at the approach of an intersection in order to provide the added lane, the standard section length of lane shift at its tangent section should have the larger of either the value calculated by the equation or the minimum value shown in Table 1.2.50.

b) Curved section

The standard value of a tangent section may be a good guide to the lengths of lane shift at a curved section.

Table 1.2.50 Section Length of Main Through Lane Shift

Design speed (Km/h) \ Area	Suburban, Rural		Urban	
	Calculation Formula	Minimum Value	Calculation Formula	Minimum Value
80	$\frac{V - \Delta W}{2}$	85	-	-
60		60		40
50		40		35
40		35	$\frac{V - \Delta W}{3}$	30
30	$\frac{V - \Delta W}{3}$	30		25
20		25		20

(unit : m)

ΔW : Lateral value of lane shift, m
V : Design speed, km/h

3) Right turn lane

a) Providing the right turn lane

1. At an at-grade intersection, a right turn lane should normally be provided except in the following cases.
   - Right turn prohibition
   - On Minor Roads and Access Roads, where there is enough traffic capacity of the road and the crossing road at the peak time.
   - On a two-lane road with design speed of 40 Km/h or less, when the design traffic volume is extremely low.

The right turn lane must be added independently to the through lane except for the special case where the right turn flow is the main traffic flow. None of the through lanes on the mid-block section may be used for a right turn lane.

The right turn lane should be provided at all intersections as a general rule, however in the above-mentioned cases, this rule may be waived. The above-mentioned case that "the design traffic volume is extremely low" is specifically, that the design traffic volume is less than 200 vehicles per hour and the right turn ratio is less than 20 percent. However, on rural roads, when the above-mentioned items are applicable, it is desirable to provide the right turn lane in as many instances as possible to insure traffic safety.

b) Length of right turn lane

1.	The length of the right turn lane shall be determined according to the design speed and the number of vehicles stored on the lane.

The length of right turn lane is composed of the length of taper and the storage length. (Figure 1.2.44)

$$L = l_d + l_s \qquad .....(27)$$

where;
- L = length of right turn lane, in m;
- l_d = length of taper, in m;
- l_s = storage length, in m.

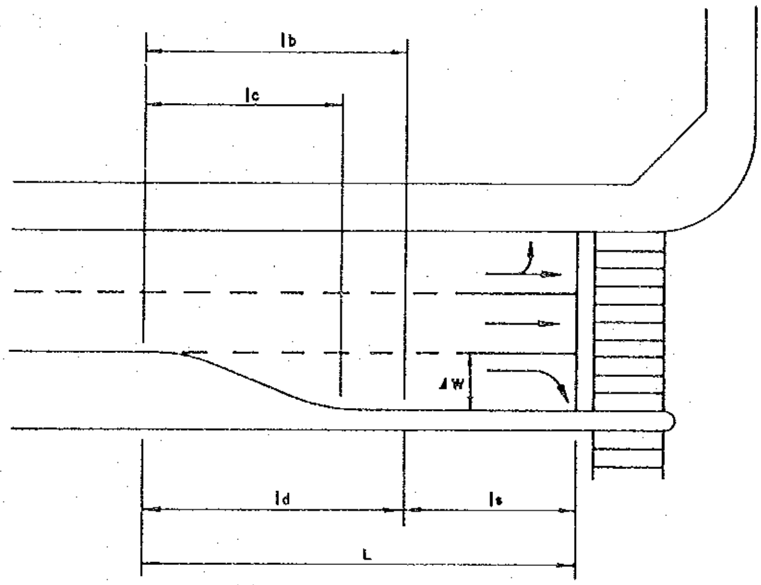


Figure 1.2.44 Length of Right Turn Lane (lb > lc)

The length of taper, l_d, is the necessary section for deceleration, and has the function of shifting the right turning vehicle smoothly from the through lane to the right turn lane. Therefore the length of taper, l_d, should not be less than both the necessary length for deceleration and the necessary length for a shift to the right turn lane.

The minimum length, l_b, for deceleration at an at-grade intersection is shown in Table 1,2.51. The minimum length, l_c, for shift from the through lane to the right turn lane, is obtained from the following equation.

$$l_c = \frac{V \times \Delta W}{6} \qquad .....(28)$$

where;

l_c = minimum length for shift, in m;

V = design speed, in Km/h;

ΔW = lateral width for shift, in m. (width of added lane)

Therefore, the minimum length of taper, l_d, must be the larger of either the value of l_b shown in Table 1.2.51 or the value of l_c obtained by the equation (28).

Table 1.2.51 Minimum Length for Deceleration (lb)

(unit : m)

Type (Area) / Design Speed (Km/h)	Major Highway in Rural Area	Urban Highway/and Minor Highway in Rural Area
80	60	45
60	40	30
50	30	20
40	20	15
30	10	10
20	10	10

$$l_d = \max(l_b, l_c) \qquad .....(29)$$

The necessary length of storage is obtained from the following equation.

$$l_s = 1.5 \times N \times S \qquad .....(30)$$

where;

l_s = storage length, in m;

N = average number of right turn vehicle per cycle, in veh/cycle;

S = average space headway, in m. (when, passenger car, S = 6 m large-sized vehicle, S = 12 m)

The average space headway, S, is adjusted by the large-sized vehicle ratio. When this ratio is unknown, S may be 7 m.

At an unsignalized intersection, l_s is obtained from the following equation taking the fluctuation of volume into account.

$$l_s = 2 \times M \times S \qquad .....(31)$$

where;
M = average number of right turn vehicles per minute, in veh/min

When the storage length cannot be calculated at either an intersection with a signal or one without a signal, the length should be at least 30 m.

Three equations, (29), (30), (31) take into account extremely severe running conditions, so that a sufficient length of right turn lane should be provided as often as possible on rural roads where alignment is good and high-speed driving is expected. In this case, it is desirable that the above-mentioned length, l_d, is approximately two times that of the equation (29).

In an urban area having many restrictions, there are not a few cases where the minimum length of right turn lane cannot be ensured. In these cases, the reduction of the length of the right turn lane should be done at the taper section (l_d), so that as much storage length (l_s) can be ensured as is possible.

c) Right turn lane at newly constructed at-grade intersections

Generally, the estimation of traffic volume at new planned at-grade intersection may not be precise, it is desirable to secure room for improvement of the length of right turn lane, if possible.

d) Plural right turn lanes

When right turning volume is heavy and plural right turn lanes are planned, the length of storage lane is obtained by dividing the necessary length of storage ls, at singular right turn lane by the number of plural right turn lanes. When the plural number of right turn lanes are provided, it is necessary to pay attention to such matters as separation of right turning traffic, opposing through traffic and the method of providing a median. Right turn traffic must depart and traverse at the independent phase of signal separated from through traffic. The number of lanes at the exit section must be more than the number of right turn lanes at the approach section.

4) <u>Left turn lane</u>

1.	In the following cases, a left turn lane or left turn roadway should be provided. - An intersecting angle is not more than 60 degree and left turn traffic is heavy. - Left turn traffic is to be specially distinguished. - The speed of left turning vehicles is high. - Many left turning vehicles and pedestrians. - Special cases where the necessity of left turn lane is warranted.
2.	The length of left turn lane should be determined according to the design speed and the number of storage vehicles.

The left turn lane must be added independently to the main flow lane (through lane) as in the case of right turn lane. "Left turn roadway" shown in Figure 1.2.46 is defined as the left turn channel provided and separated by a traffic island. The left turn roadway may be used not only by itself independently but also for approach - end treatment of the left turn lane or deceleration lane.

The length of the left turn lane, L, shown in Figure 1.2.45, is composed of the length of taper, l_d, and the storage length, l_s. The length of the left turn is determined by the same method as the length of right turn lane. (See 1.2.6, (4), 3), b))

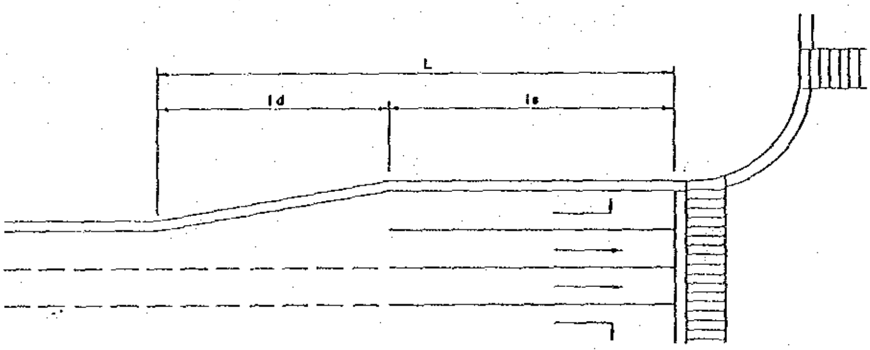


Figure 1.2.45 Left Turn Lane

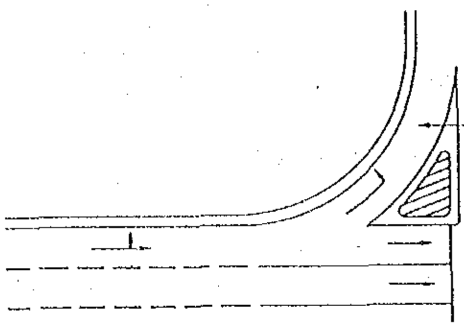


Figure 1.2.46 Left Turn Roadway

5) Speed change lane

1. In the following cases, a deceleration lane should be provided.
   - There is decelerating and diverging traffic from the Major Trunk Roads of full or partial access control.
   - The necessity is warranted.

2. In the following cases, an acceleration lane should be provided.
   - There is accelerating and merging traffic into the Major Trunk Roads having full or partial access control.
   - The necessity is warranted.

3. The lengths of speed change lane vary according to the characteristics of roads, the difference between design speed of the through lane and that of the speed change lane, the method of traffic control, etc. The standard lengths of speed change lanes are shown in Table 1.2.52.

Table 1.2.52 Standard Length of Speed Change Lane (excluding taper)

Type (Area) / Speed (km/h) / Design Speed (Km/h)	Length of Deceleration Lane						Length of Acceleration Lane					
	Major Highway in Rural Area			Urban Highway and Minor Highway in Rural Area			Major Highway in Rural Area			Urban Highway and Minor Highway in Rural Area		
	To Stop	To 20km/h	To 40km/h	To Stop	To 20km/h	To 40km/h	From Stop	From 20km/h	From 40km/h	From Stop	From 20km/h	From 40km/h
80	60	50	30	45	40	25	140	120	80	90	80	50
60	40	30	20	30	20	10	100	80	40	65	55	25
50	30	20	-	20	15	-	60	50	-	40	30	-
40	20	10	-	15	10	-	40	20	-	25	15	-
30	10	-	-	10	-	-	20	-	-	10	-	-

The items prescribed in this section should mainly be applied to the speed change lane provided at an at-grade intersection, but should not be applied at a grade separated intersection.

The standard length of speed change lane should not be less than the value shown in Table 1.2.52. However, it is possible to think of deceleration at a taper section also on the deceleration lane. In this case, the length of deceleration lane including taper section may not be less than the value shown in Table 1.2.52.

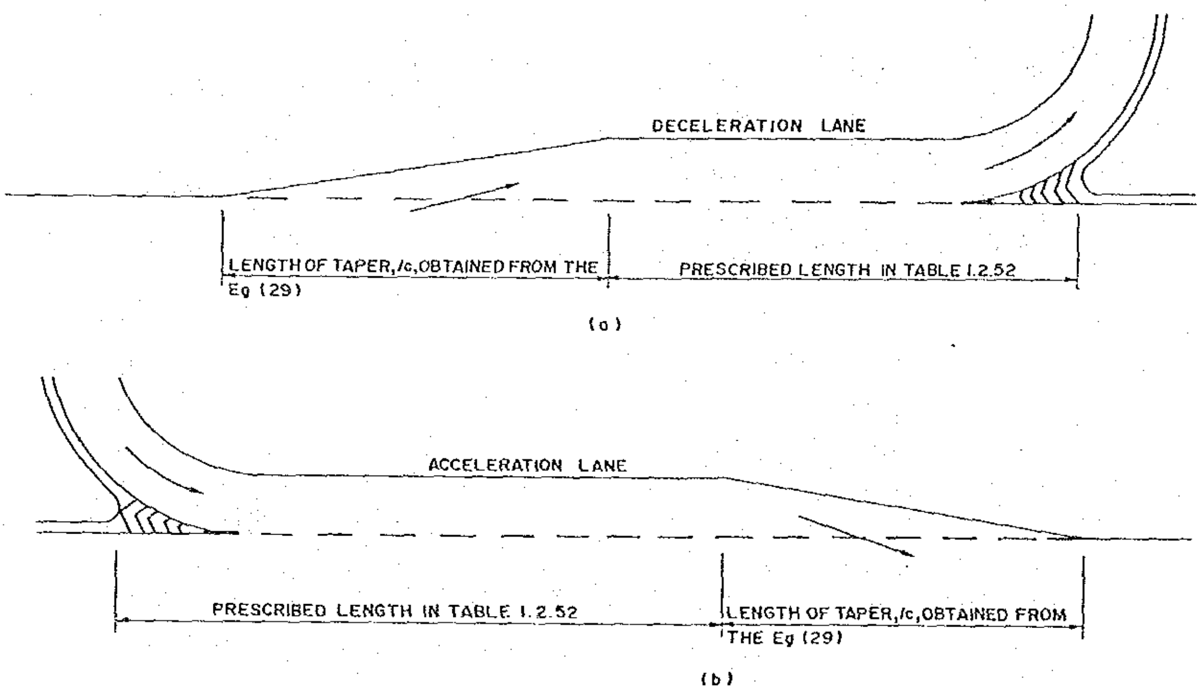


Figure 1.2.47 Speed Change Lane

(5) Channel, traffic island, and corner cut

1) Channel

1. When designing a channel, the speed of the assumed vehicle and various other conditions must be taken into account.

2. Arrangement of channel must take into account the traffic volume, method of traffic control, and pedestrian, to present disturbance of traffic flow.

3. The widths of channel corresponding to the design vehicles are

shown in Table 1.2.53.
4. The widths shall be determined according to the design vehicle, the radius of curvature, and the turning angle. The widths must not be too wide or too narrow.

Table 1.2.53 Widths of channel

(Unit : m)

Outer Radius of Channel \ Design Vehicle	Semi-trailer (Major Truck Roads)	Ordinary Motor Vehicle (Other Roads)
13 ≦ R < 14	8.5	5.5
14 ≦ R < 15	8.0	
15 ≦ R < 16	7.5	5.0
16 ≦ R < 17	7.0	
17 ≦ R < 19	6.5	
19 ≦ R < 21	6.0	4.5
21 ≦ R < 25	5.5	
25 ≦ R < 30	5.0	4.0
30 ≦ R < 40	4.5	
40 ≦ R < 60	4.0	3.5
60 ≦	3.5	

It is correct proctice to concentrate a channel as much as possible in order to make the size of intersection small and guide traffic flow regularly. If channel width is too great, traffic flow gets disorderly when traffic volume is heavy. Therefore the reasonable width of channel should be provided.

a) <u>Radius of curvature of channel</u>

As for the right turn channel, the radius of curvature may be from 15 to 30 m when a intersecting angle is nearly a right angle, considering that vehicles are turning at very slow speed from a full stop. In this case, the inner curve can begin approximately 5 m before, so that the tangent length of outer curve is longer than that of inner curve. (Figure 1.2.48)

On urban streets, when the length of a corner cut had been determined previously, the possible maximum radius may be determined by ensuring the width of sidewalk. (Figure 1.2.49)

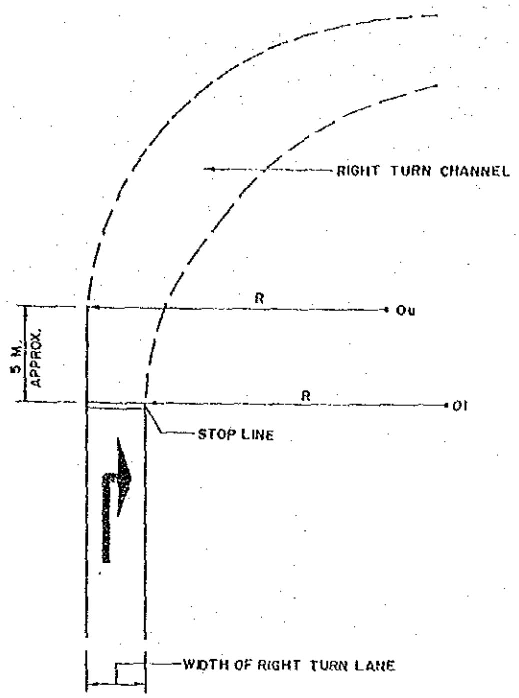


Figure 1.2.48 Design of Right Turn Channel

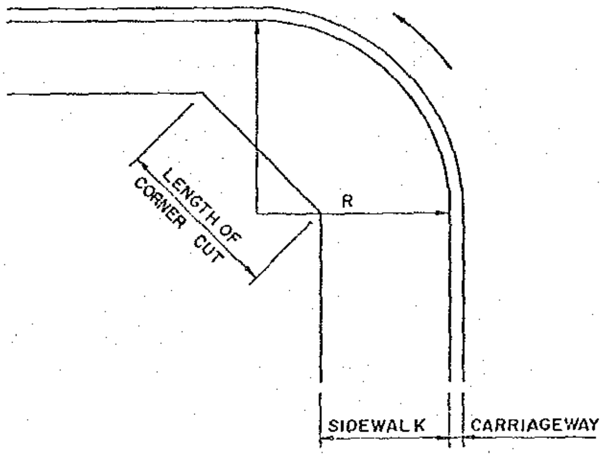


Figure 1.2.49 Radius of Corner-curb and Length of Corner Unit

b) Width of channel

The widths of channel shall be the values shown in Table 1.2.53 according to the design vehicle and the radius of curvature. When the channel is separated by the islands, the lateral clearance allowance of 0.5 m should be provided on both sides of the channel, which is paved just like a carriageway. This lateral clearance allowance may be included in the necessary width for shoulder, gutter, and setback of channel.

Transition taper of widening is generally done on the inner side. In this case, a clothoid curve or circular curve is used for the transition curve. For a circular curve, the radius of curvature may be 3 to 4 times the inner radius of channel.

c) Treatment for channel of wide width

When a channel of comparative small radius is provided and designed using a semi-trailer as design vehicle, the channel must be made wide. As a result, there may be cases when two or three small-sized cars are turning side by side and traffic flow will be confused, which may result in accidents such as collisions between large-sized vehicles and motorcycles.

Proper channelization of traffic flow in a wide channel should be ensured by providing sufficient width so that a passenger car can pass in the center of the channel and by marking diagonal lines on both sides as shown in Figure 1.2.50.

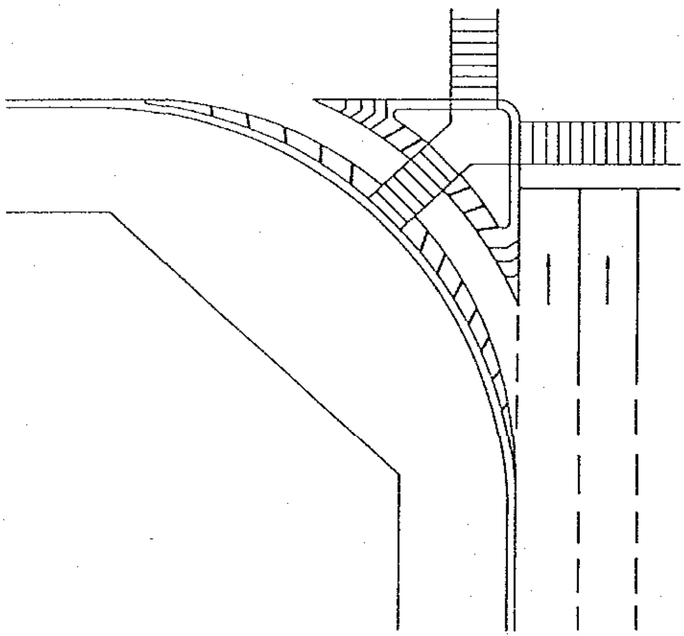


Figure 1.2.50 Channel of Wide Width

2) Traffic island and separator

1. A traffic island and separator should be provided to guide traffic flow safely and properly, taking channelization of intersection into

consideration.

2. Traffic island and separator may be provided with curbs where deemed necessary or desirable.

3. In the following cases, it is desirable to provide medians at the approach of an intersection, even when a two-way highway is not separated by medians.
   - Roads with design speed of 60 km/h or higher intersect each other.
   - Many pedestrians are crossing and the distance they cover to cross is long.

4. Design sequence is that the channel is designed first and then the traffic island and separator are planned at the remaining portion. In this case, an appropriate nose offset and setback should be provided.

5. The appropriate width, length, and area of the traffic island or separator should be determined taking its function into full consideration.

6. As a general rule, the alignment is the combination of a straight line and a curve.

7. It is important to give indication of approaching to the traffic island or separator by pavement markings, etc.

An at-grade intersection where traffic is directed into definite paths by islands is termed a channelized intersection.

An island is a defined area between traffic lanes for control of vehicle movements. Islands also provide an area for pedestrian refuge and traffic control devices. Within an intersection a median or an outer separation is considered an island. This definition makes it clear that an island has no single physical type--it may range from an area delineated by a curb to a pavement area marked by paint.

Islands are generally included in intersection design (channelization) for one or more of the following purposes:
- Separation of traffic conflicts.
- Control of angle of traffic conflict.
- Reduction in excessive pavement areas.
- Regulation of traffic and indication of proper use of intersection.

- Arrangements to give advantage to a predominant turning movement.
- Protection of pedestrians.
- Protection and storage of turning and crossing vehicles.
- Location of traffic control devices.

Islands serve three primary functions: (i) channelization -- to control and direct traffic movement, usually turning, (ii) division -- to divide opposing or same direction traffic streams, usually through movements, and (iii) refuge -- to provide refuge for pedestrians. Most islands combine two or all of these functions.

The minimum radius of curvature of approach nose of island may generally be 0.5 m. The nose offset (O_1, O_2) and the setback (S_1, S_2, S_3) shown in Figure 1.2.51 may be provided. These values differ according to the speeds of vehicle, the sizes of island, the urban or rural area, and the classifications of road. The standard values are shown in Table 1.2.54 and Table 1.2.55.

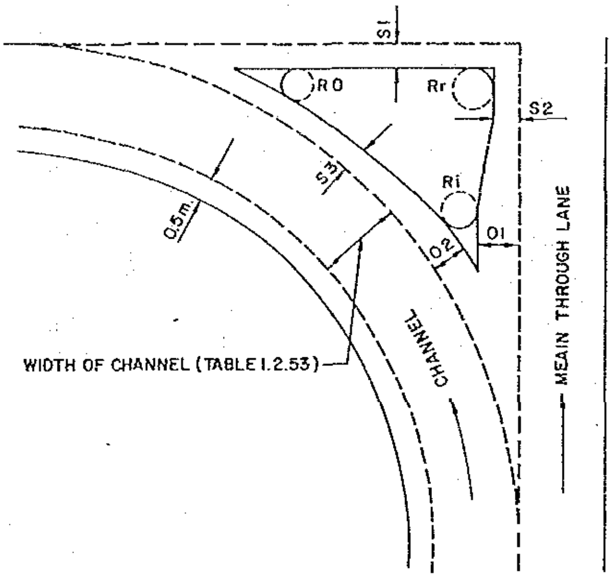


Figure 1.2.51 Setback and Nose Offset

Table 1.2.54 Values of Setback and Nose Offset

(Unit : m)

Design Speed (Km/h)	S_1,S_2	S_3	O_1	O_2
80	1.0	0.5	1.5	1.0
60	0.75	0.5	1.0	0.75
50	0.5	0.5	0.5	0.5

Table 1.2.55 Radius of Island Nose

(Unit : m)

R_i	R_o	R_r
0.50 ~ 1.00	0.50	0.50 ~ 1.50

The transition of the nose offset is provided gradually at the whole island on both sides of the through lane and channel. In the case of a large island, the rate of transition is 1/10 to 1/20 on the side of the through lane, and 1/5 to 1/10 on the side of the channel.

Very small islands are troublesome for drivers and difficult to see in a rainy night, which is dangerous because there is possibility that a vehicle collides with a small island. The minimum dimensions of island and separator are shown in Table 1.2.56 and Figure 1.2.52 according to the type. When the prescribed size of island or separator can not be provided in spite of its necessity, the pavement markings should be provided instead of the island or separator.

Table 1.2.56 Minimum Dimensions of Island and Separator

	Element	Urban	Rural
(a)	W_a	1.0 m	1.5 m
	l_a	3.0 m	5.0 m
	R_a	0.5 m	0.5 m
(b)	W_b	1.5 m	2.0 m
	l_b	$(W_p + 1.0)$m	$(W_p + 1.0)$m
	R_b	0.5 m	0.5 m
	Area	5.0 m^2	7.0 m^2
(c)	W_c	(D + 1.0)m	(D + 1.5)m
	l_c	5 m	5 m
(d)	W_d	1.0 m	1.5 m

D : Width of Road Appurtenance (m)
W_p : Width of Crosswalk
$W_{a \sim d}$: See Figure 1.2.52

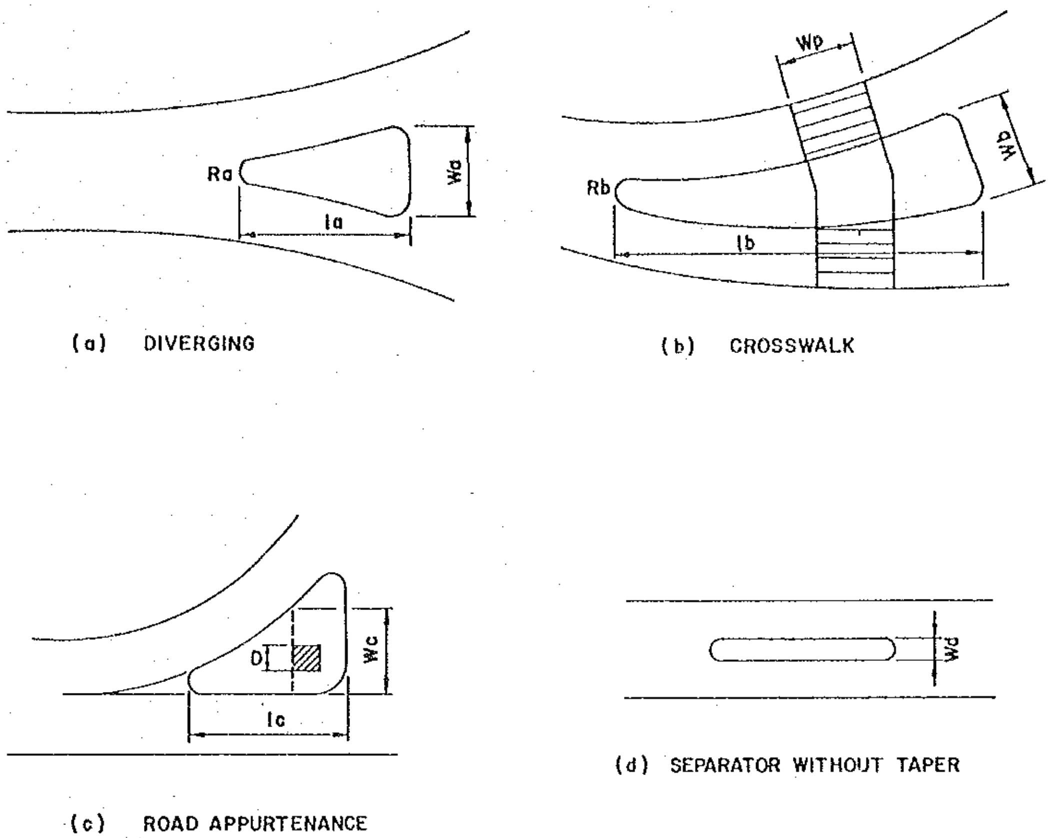


Figure 1.2.52 Islands and Separators

The marking of the approach to the island is important for traffic safety. The transition length of marking needs at least the value obtained from the following equation, as shown in Figure 1.2.53.

$$l_a = \frac{1}{3} V R \qquad \text{.....(32)}$$

$$l_b = \frac{2}{3} V R \qquad \text{.....(33)}$$

where,

l_a, l_b = transition length of marking, in m,
V = design speed, in km/h,
R = radius of approach nose of island, in m.

When the shift direction is not a main traffic flow in the case of (b), the transition length may be l_a.

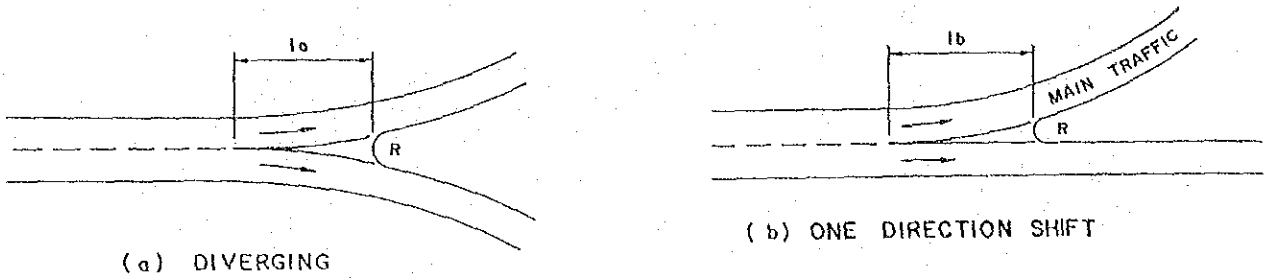


Figure 1.2.53 Transition Length of Marking of Approach to Island

3) Passage method and corner cut

a) Passage method

The passage methods at an intersection are shown in Table 1.2.57. They differ according to the classification of roads and whether intersection is furniched with a signal. The elements of intersection alignment depend on the passage method, so that it is necessary for intersection design to assume the passage method of the intersection. Table 1.2.57 shows the standard of passage method of right- and left- turning vehicle.

Table 1.2.57 Passage Method of Right- and Left-turning Vehicle at an At-grade Intersection

Condition \ Classification of Road			Rural, Suburban				Urban			
			Major Trunk	Major	Minor	Access	Major Trunk	Major	Minor	Access
Stop Control	Approach		S4*	T4	T4	T3, T2 or T1	S4*	T3	T2	T1
	Exit	Major highway	S4*	T4	T3	T2 or T1	S4*	T3	T2	T1
		Subordinate highway		T3	T3	T2 or T1		T2	T2	T1
Signal Control	Approach		S4*	T4	T4	T3, T2 or T1	S4*	T3	T2	T1
	Exit		S3*	T3	T2	T2 or T1	S3*	T2	T2	T1

S : Semi - trailer

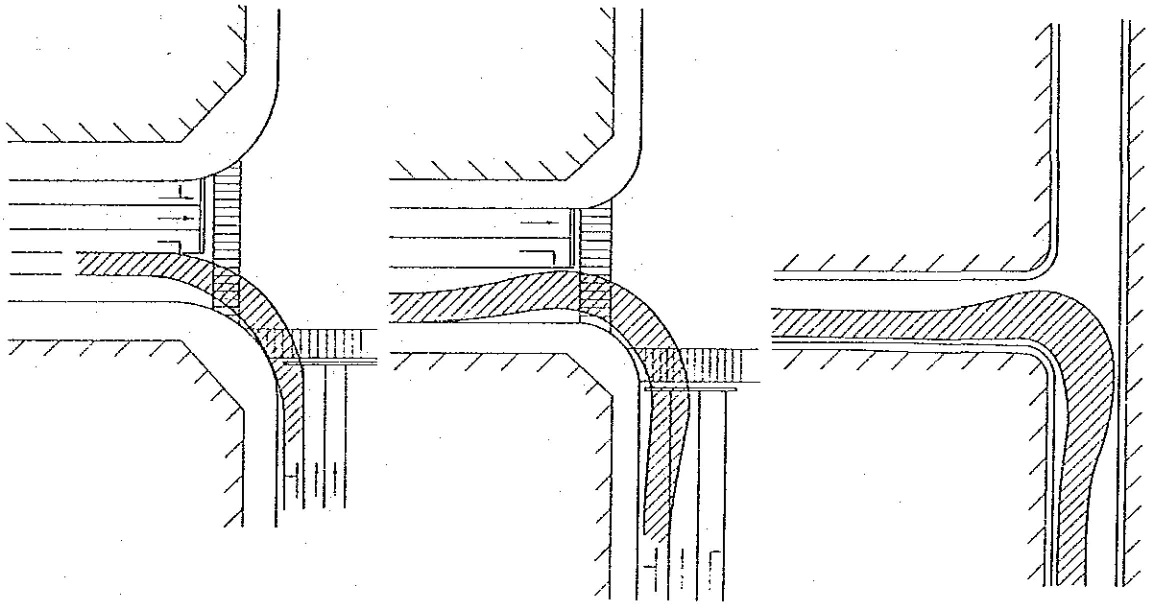
T : Ordinary Motor Vehicle

The numbers "1" to "4" written after S and T in Table 1.2.57 represent the following passage methods.

1. Occupy the whole width of carriageway.
2. Occupy the left side of carriageway using the opposite lane.
3. Occupy the turning lane and the contiguous lane without using the opposite lane. Instead of the turning lane, the first right lane may be used at the right turn or the first left lane at the left turn.
4. Occupy only the turning lane or the first right lane at the right turn, and the first left lane at the left turn.

* : When there is difference in the design vehicle used for a main road and a subordinate road, the design vehicle of the subordinate road is to be used.

Figure 1.2.54 shows examples of passage method at an intersection.



(a) S4 — S3 (b) T3 — T2 (c) T1 — T1

Figure 1.2.54 Examples of Passage Method at an At-grade Intersection

It is necessary to take the following into consideration.

- At an approach of a road on the exit side where passage method "1" is adopted, it is necessary to push back the stop line to the possible point for this passage method.
- At an approach of a multilane road adopting methods "1" to "3", when the vehicle corresponding to the design vehicle is turning right or left, the contiguous lane is occupied. When the division of passage at approach to the intersection is specified by traffic sign or other means, passing to invade the contiguous lane is prohibited.

b) <u>Corner cut</u>

For arterial street design, adequate radii for vehicles must reconcile the needs of pedestrians and the difficulty in acquiring additional right-of-way or corner setbacks. Because the corner radius often is a compromise, its effect on pedestrians in combination with vehicular movements should be examined.

It is desirable that the length of a corner cut to be determined by examination of the various factors at each intersection, based on the necessary values for the smooth passage of vehicle depending on the intersecting angle, the width of sidewalk, the design vehicle, and the passage method. (See Figure 1.2.55) The various factors are as follows; the storage space of pedestrians and bicycles, visibility, and space for landscaping. In urban areas, it is not necessarily practical to calculate the length of corner cut at each intersection. (See Figure 1.2.56) The general standard lengths of corner cut at urban intersections are shown in Table 1.2.58.

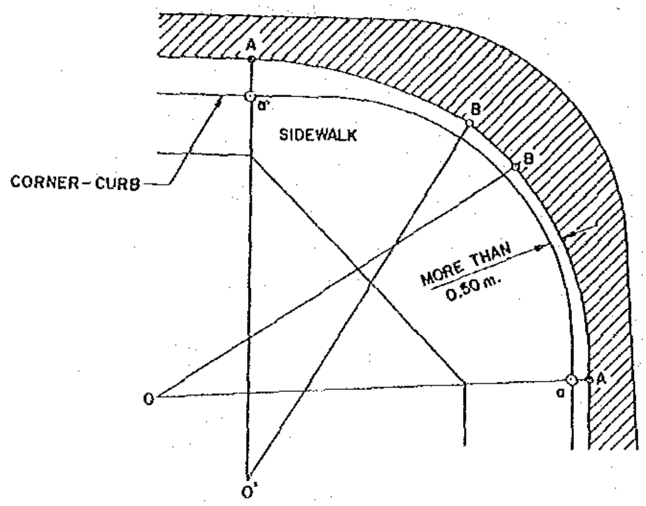


Figure 1.2.55 Occupied Width by Vehicle and Corner-curb

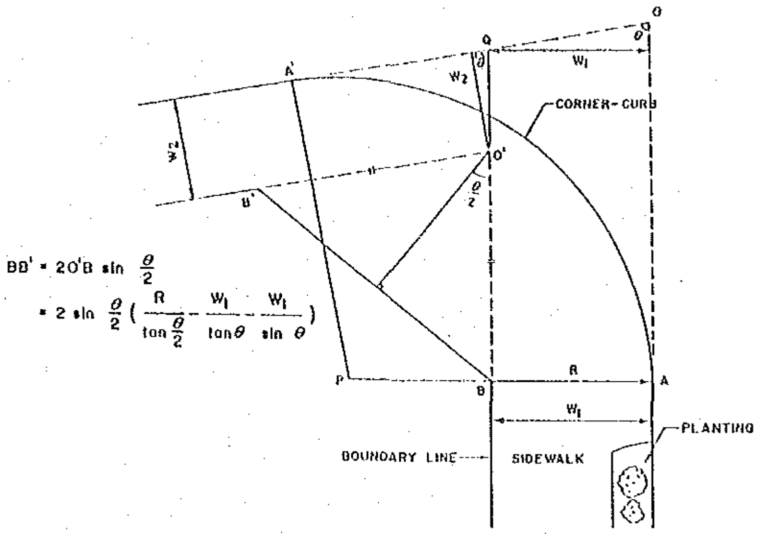


Figure 1.2.56 Corner Curb and Corner Cut

Table 1.2.58 Standard Length of Corner Cut

Classification	Major Trunk Roads	Major Roads	Minor Road	Access Roads
Major Trunk Roads	12	10	5	3
Major Roads	-	10	5	3
Minor Roads	-	-	5	3
Access Roads	-	-	-	3

1.2.7 Grade Separations

(1) General

1. In planning grade separation, the propriety of grade separation and its structural type shall be determined by the comprehensive examination not only of the standard and function of the road, and various problems of traffic control in the area including that in the vicinity of intersections, but also on the roadside conditions including the surrounding land use and other environmental conditions.

The ability to accommodate a high volume of traffic safely and efficiently through intersections depends largely on what arrangement is provided for handling intersecting traffic. The greatest efficiency, safety, and capacity are attained when the intersecting through-traffic lanes are separated in grades. An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels.

The type of grade separation and interchange, along with its design, is influenced by many factors, such as highway classification, character and composition of traffic, design speed, and degree of access control. These controls plus sign requirements, economics, terrain, and right-of-way are of great importance in designing facilities with adequate capacity to safely accommodate traffic demands.

In this guideline, only grade separated intersections are described. As for the interchange, please refer to the other books, such as AASHTO's standard.

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. It may be necessary to make several fairly complete preliminary layout plans before a decision regarding the most desirable general plan can be reached.

In many instances the existing street approaching the major roadway requires some improvements to facilitate traffic and increase capacity to ensure free movement to and from the major roadway. Typical improvements are pavement widening, control of parking and pedestrians, improvements of intersections with traffic signals, marking, channelization, and one-way operation where appropriate.

(2) Planning standard for grade separations

1. When two roads having four and more lanes excluding climbing lane, turning lane, and speed change lane are intersecting mutually, the inter section should be separated in grades as a general rule. However, when the grade separation is unsuitable due to traffic conditions, or in an unavoidable case, such as due to a topographical reason, this rule may be waived. 2. Even at the intersection where grade separation has been planned for, if the traffic flow can be managed properly by at-grade intersection for a certain period of time, in terms of traffic volume and safety, this location may be made to be an at-grade intersection as one of the phases of stage construction. However, necessary land acquisition should be made in advance based on the design of grade separation for future implementation.

When two roads each having four and more lanes intersect mutually, the intersection should be separated in grades as a general rule. However, when an at-grade intersection is allowable judging from the traffic volume of intersection, the traffic safety, the composition of road network, and the interval between intersections, or in an unavoidable case, such as due to a topographical reason, this rule may be waived.

When either of the intersecting roads is a two-lane road, an at-grade intersection is generally recommended. However, on the basis of the traffic volume at the intersection, traffic safety, and the function of road, this rule may be waived when a grade separation is desirable.

When a two-lane road intersects a two- or one- lane road, and one-lane roads intersect mutually, the intersection may be at-grade.

1) Concrete application of planning standard

a) Examination by traffic volume

When the traffic volumes of intersecting roads exceed the capacity of a signalized at-grade intersection, this intersection should be separated in grades as a general rule.

When grade separation is desirable on the basis of the classification of road, the composition of road network or traffic control, even the intersection within the capacity of signal control should be separated in grades.

b) Design traffic volume and stage construction

It is a general rule to determine the necessity of grade separation, by judging whether the desired level of service, corresponding to the design traffic volumes of roads crossing the intersection, can be attained.

When grade separation is expected to be needed in a certain target year, measures for necessary land acquisition must be taken and examination of the construction site and the road structure should be made on the basis of the assumption that there will be a grade separation. However, from the economic viewpoint, at times it is difficult to separate in grades at a first stage construction and it is not necessarily a wise policy that excessive facilities be constructed at the first stage.

In the case of stage construction, it is normally adequate to take the following measures according to the adopted standard and the necessary completion time of grade separation.

On the basis of estimation of the future traffic volume of the intersection, when the intersection is to be at-grade for the time being, it is still desirable to acquire the necessary land for grade separation from the first. The time to start constructing the grade separation should be determined from the preinvestment, the cost of modification of earlier construction, and the estimated time when the traffic volume will exceed the capacity of an at-grade intersection.

When an intersection is temporarily opened as at-grade intersection where the land has been acquired for the complete configuration of grade separation from the first, the following are recommended to be taken into consideration.

In case that the mid-block sections shall be opened at a completed configuration and the intersection shall be temporarily opened as an at-grade intersection, the general configuration shown in Figure 1.2.57 should usually be employed. This configuration shows that the frontage road of grade separation is used for main traffic and reconstruction at the time of grade separation shall be minimized. In this case, the width of frontage road of grade separation is generally narrower than the width of one side of main traffic lane, so that the following measures shall be taken.

- Sidewalk is partly reduced to widen the carriageway. After construction of grade separation, the sidewalk is restored to the regular width.
- A carriageway is provided to make inroads partly on the site of grade separation structure and a sidewalk is provided with the same width as that of mid-block section. At the time of construction of a grade separation structure, temporarily the width of carriageway shall be secured by narrowing the sidewalk.

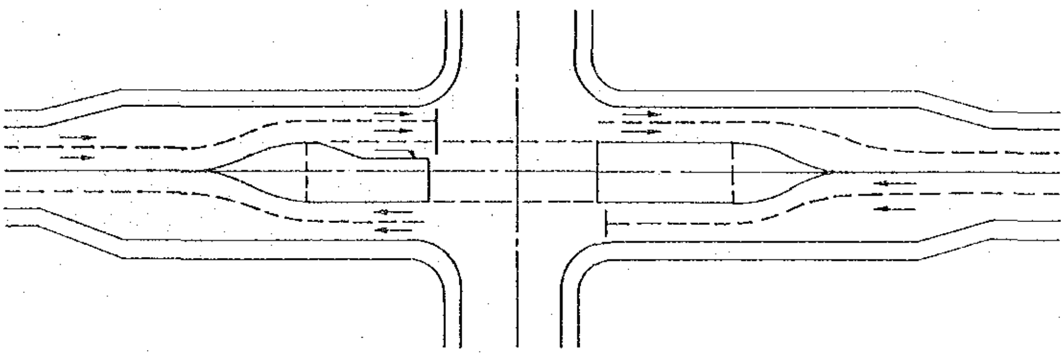


Figure 1.2.57 Temporary Open of Intersection

The choice between the two measures should be made on the basis of the time required for constructing grade separation, economic efficiency, and the width of sidewalk.

Generally, the latter measure, by which the width of sidewalk can be ensured, is suitable for urban areas, and the former measure is economical and suited for rural areas.

2) General rule of grade separation structure

1. When a intersection is separated in grades, if necessary, a road connecting with the intersecting through traffic lanes shall be provided.

2. When choosing the structural type of grade separation, adequate attention should be given to insure that the dominating traffic flow can be managed smoothly, and due attention is to be paid to the possible impact on roadside conditions.

3. The number of lanes at a grade separated intersection and connecting road section shall be determined based on review of the number of lanes of the extended road sections to and from the intersection as well as on the traffic flow conditions including collection and distribution of vehicles.

4. Grade separation must be designed based on the design conditions of the intersecting roads separated in grades. In addition, due attention shall be paid to ensure the safe and smooth traffic flow on the frontage road of grade separation. In this case, it is necessary to take measures to separate the pedestrians and bicycles from vehicles.

Grade separation is ruled by various design factors depending on the purpose. At a simple grade separation, both roads have geometric structures satisfying each running condition. The type of grade separation is determined mainly by topographical factors. In many cases, an at-grade intersection is juxtaposed with a grade separation. In this case, the road which should have priority should be planned to carry over or under the structure.

The effects on traffic flow by grade separation are felt not only at the site of grade separation but also the extended road section. Special attention should be paid to the balance between the configulation of grade separation and the extended road section from the viewpoint of the traffic capacity of surrounding road network and the lebel of service.

In an unavoidable case when the number of main through lanes at a grade separation is fewer than that at a surface street section, it is necessary not only to ensure a sufficient length of transition shift section but also to guide vehicles appropriately by signs, not to cause the confusion of traffic.

In case an at-grade intersection of low capacity is located in the neighborhood of a grade separated intersection, the at-grade intersection may be separated in grades continuously, because the effect of grade separation is decreased and it is undesirable from the viewpoint of securing traffic safety to keep such a case.

Since the road network in an urban area compared to one of a rural area is densely composed of various roads, from Major Trunk Roads to Access Roads, the interval between intersections is short. When an intersection is separated in grades, it will impose various restrictions on the roadside area between the grade separated intersection including the approach section and the next intersection. It will also put a great effect on the traffic management of the next intersection.

When the necessary interval of main intersections is calculated in the case of grade separation, each section shown in Figure 1.2.58 must be considered. Therefore, where there is a heavy traffic intersection within the section obtained from Figure 1.2.58, it is necessary to consider the next grade separation.

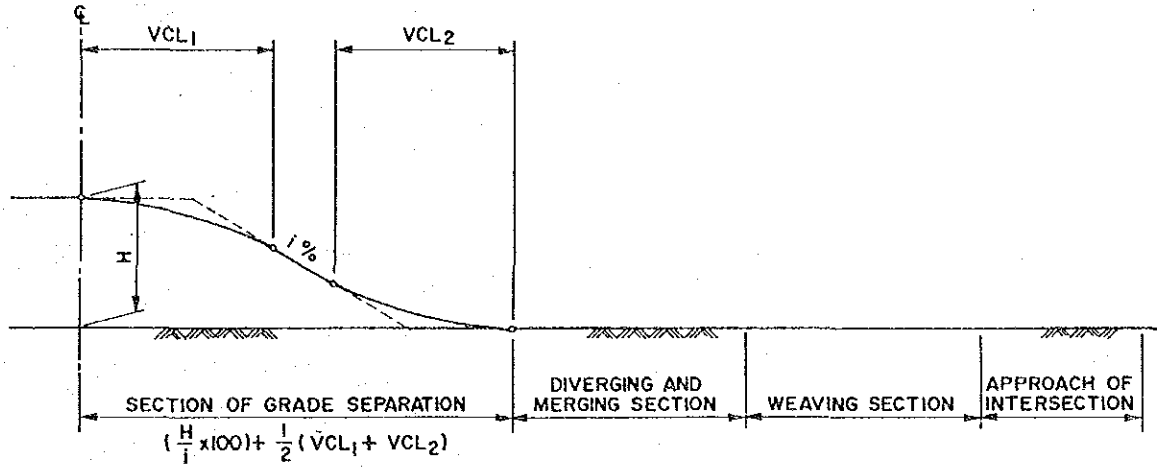


Figure 1.2.58 Necessary Interval of Main Intersections

The choice of overpass versus underpass is influenced by the various conditions, such as, topography, geology, cost of construction work and harmony with surrounding view. Generally, the advantages and disadvantages are as follows.

- From the point of construction work, in the case of an underpass, there are tendencies for construction term to be long and for construction cost to be high because of need for retaining wall work, abutment work, remoral of encumbrances for excavation, and breasting work.
- From the point of maintenance and administration, in the case of an underpass, troublesome drainage problems may exist, so that it is necessary to manage the planning carefully and provide adequately for the cost of maintenance and administration.
- An underpass is more advantageous from the standpoint of aesthetic sense and road-user convenience.

Vertical clearance must be determined so as not to interfere with the right turning vehicles, and to allow room for pedestrian crossing. In the case of an overpass, the length of retaining wall at an approach section of overpass is determined by judgement with consideration given to aesthetics, utilization of the space under overpass structure, and economy. Since pedestrians and bicycles can cross the surface street and the crossing distance is made shorter, there is a little necessity for providing a sidewalk and cycle path on the grade separated structure.

If only motor vehicle traffic is separated in grades, frequently great impact on traffic operation is to be expected. As in designing the frontage road of grade separation left under at-grade condition, to provide sidewalk and stopping lane for roadside service must be taken into consideration.

(3) Planning and design of grade separated intersection

1) General rule for planning

1. At a grade separated intersection, the traffic flow separated in grades shall be in general the flow with the heaviest traffic volume. However, the determination shall be made based on the comprehensive examination of various factors such as management of smooth traffic flow, topography of the location, land use situation in the neighborhood of the intersection, type of roads, other facilities and construction cost, etc.

There is almost no case that the cloverleaf-type or the trumpet-type are adopted for a grade separated intersection on urban roads, for economical reason. Usually, the diamond-type or its variation shown in Figure 1.2.59 are planned because they need a smaller site area. Through traffic section, frontage road of grade separation, and exit and entrance section of grade separation are defined as shown in Figure 1.2.59. The frontage road of grade separation is provided on both sides of through traffic section of grade separation. The direction separated in grade must be determined based on the comprehensive examination on the various factors such as management of smooth traffic flow, topography of the location, land use situation in the neighborhood of the intersection, type of road network, railway, river, other facilities and construction cost.

It is a general rule to pass main traffic through freely, which is the direction of heaviest traffic volume, in order to operate prominent traffic flow smoothly. When main traffic can not be passed through freely, sometimes the advantage occurs where the traffic direction crossing the main traffic flow is passed through freely. Therefore sufficient examination is required. (Figure 1.2.59(C)).

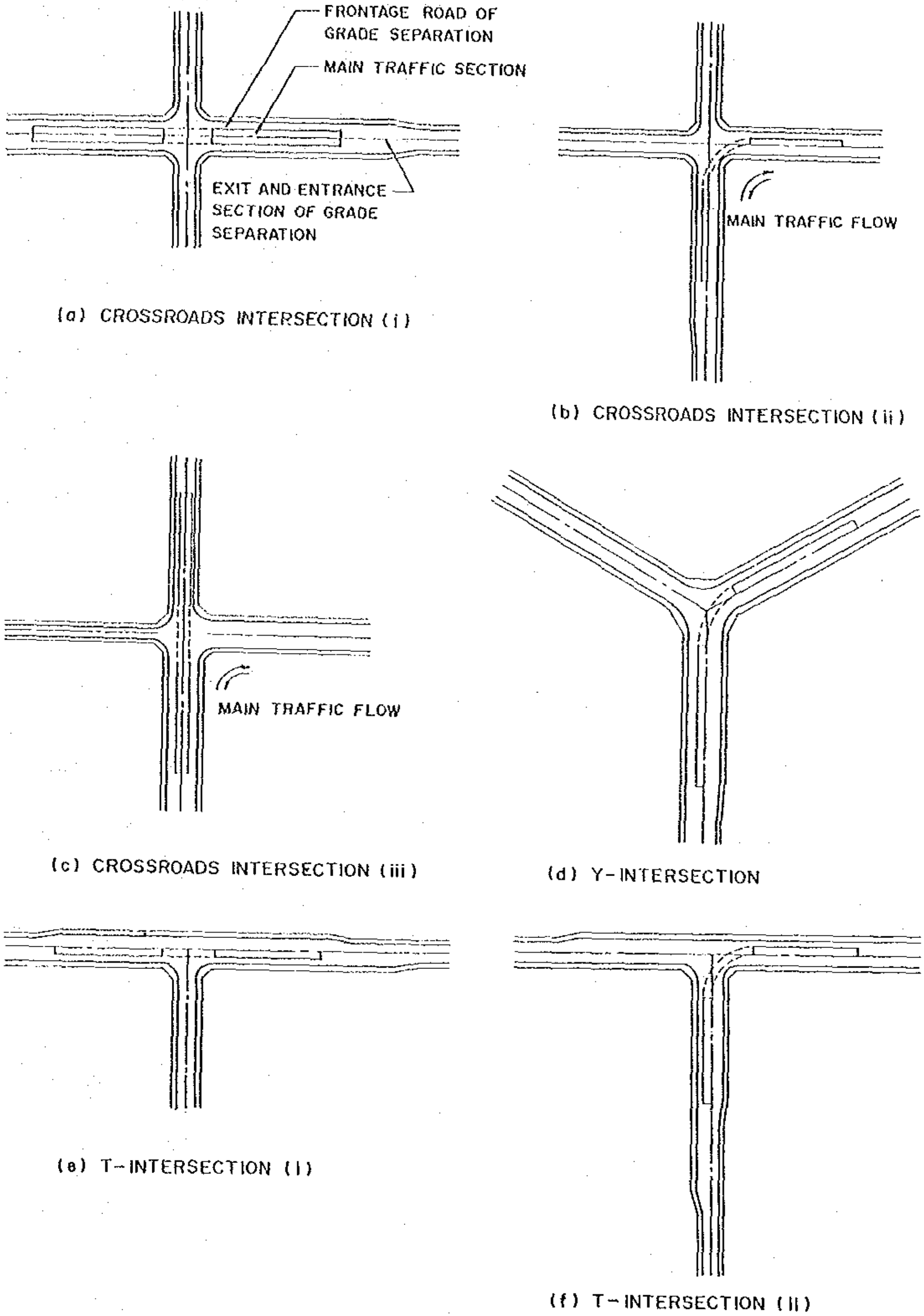


Figure 1.2.59 Type of Grade Separated Intersections

In urban areas, plans are often drafted wherein main intersections are separated in grades continuously in order to systematically increase the capacity of traffic. In this case, since weaving flow occurs between one grade separated intersection and the next, planning must ensure the proper weaving length.

When several intersections in a short section are separated in grade all together, since right- and left- turn traffic is collected, it is necessary to take the traffic management of frontage road at grade separated intersections into consideration.

2) Design of grade separated intersection

1. The geometric design of through traffic section at a grade separated intersection shall be based on the standard of ordinary road section, as a general rule.

2. When the through traffic section is a two-way two-lane roadway, its cross section must be designed to have a proper space for disabled vehicles, if deemed necessary.

3. Walkways shall be provided for maintenance and inspection of roadway, if deemed necessary.

- The standard of geometric design at a grade separated intersection shall be based on the standard of ordinary road section. However, as for the choice of design speed, in the unavoidable case, such as due to the topography or the interval of contiguous intersections, the design speed at a grade separated intersection may be reduced by 10 - 20 km/h below that on the mid-block section.
- It is desirable that the number of through lanes shall be more than two lanes for one direction from the viewpoint of traffic safety of vehicles in operation. If the number of through lanes is one lane for one direction, it is desirable to provide the width added to the shoulder where a disabled vehicle can be turned out.
- It is desirable to provide a walkway of 0.75 m in width in order to secure safety of maintenance and administrative personnel.

3) Frontage road of a grade separated intersection

1. The geometric design of a frontage road of a grade separated intersection shall be based on the standard of ordinary road section as a general rule.

2. The width of a frontage road of grade separated intersection must have at least the width of one lane and stopping lane added.

3. At the intersection where the frontage road of grade separation and the crossing road intersect at -grade, handling of the traffic must be done smoothly. The geometric design of such intersections is based on the standard of "1.2.6 At-grade intersection".

Various elements of frontage road at grade separated intersections, such as cross section, horizontal alignment, and vertical alignment, shall be based on the ordinary standard prescribed in the abovementioned chapters. The design speed may be determined by taking the design speed of the main road and that of subordinate roads into consideration. The width must be determined according to the volume of right- and left- turns at an intersection. However, considering the stopping of vehicles, the width should not be less than the value of one-lane width plus stopping lane width at least. In this case, it should be better to use all the width as the left turn lane instead of providing the stopping lane in the neighborhood of intersection.

There is usually an at-grade intersection crossing the subordinate road and the frontage road under or over the grade separated structure. In this case, heavy traffic volumes of right- and left-turning vehicles occur at the at-grade intersection. It is necessary to pay attention to designing at-grade intersections in light of traffic management requirements. If the traffic management of this intersection is poor, the effect of grade separation is negated.

To give an example of the diamond type used usually at a grade separated intersection, the treatment of right turn traffic is classified into two methods; a) turning right on inner lane b) turning right on outer lane, shown in Figure 1.2.60. Generally, the method of turning right on an inner lane is a desirable method because the number of signal phases is small and the capacity of traffic management is increased. However, as the span length of overpass structure is increased, the construction cost is higher than in the case of the treatment by outer lane. The method of turning right on the outer lane is limited to the case where not only the traffic volume of crossing road is low but also the volume of right- and left- turn traffic is low.

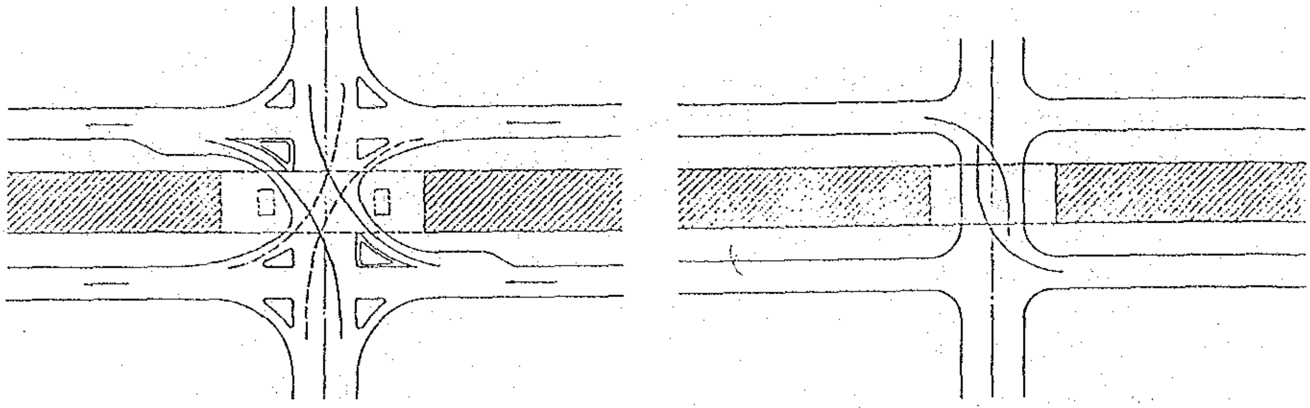


Figure 1.2.60 Treatment of Right Turn Traffic

In the case of turning right on an inner lane, there is an arrangement whereby the right turn lane is provided under overpass structure. This makes the traffic management of at-grade intersection easy, but on the other hand, the vertical clearance of right turn lane must be ensured and the vertical alignment of main traffic lane is restricted and the effect of grade separation extends to a wide area. Therefore, adequate examination of the advantages and disadvantages of this form is needed.

4) Exit and entrance section of grade separation

1. Transition runoff at the exit and entrance section of grade separation should be designed in a continuous smooth curved line, for safe and smooth traffic flow. The rate of transition shall be in accordance with the design standard specified in the section 1.2.5 (8), 2) "Transition runoff in addition or reduction of number of lane".

2. The exit and entrance sections of grade separation shall be designed to ensure the smooth traffic flow.

Exit and entrance sections of grade separation are defined as the neighborhood parts connecting with the main traffic section and the frontage road. In these sections, traffic diverges and merges. To prevent confused traffic flow, it is necessary to pay the closest attention to these matters in the design work.

As the number of lanes at a grade separation is larger than that on the roadway segment, it is a general rule to widen the carriageway. The transition rate of widening shall be in accordance with the design standard specified in the section 1.2.5 (8), 2) "Transition run-off in addition or reduction of number of lane". Furthermore, the length, L, of parallel section of main traffic section with frontage road as shown in Figure 1.2.61 must be provided with the proper length for the safety of diverging, and for the management of smooth merging traffic. For example, it is desirable that the length is approximately 20 m, when the design speed of main traffic section is 60 Km/h.

It is easy for transition to be done by using an S-shape curve of proper radius. However, it is not always desirable to draw a boundary line of a road and private property by a curved line. It will be adequate that the boundary line shown in Figure 1.2.61 is drawn in short sections of straight lines.

As traffic accidents are apt to occur in the exit and entrance section, grade separation must be distinguished early by driver and measures for visual guidance must be taken in order to secure safety in vehicle operation. For that purpose, the following measures are to be considered.

- Installation of a guide sign of high visibility.
- Provision of a separator of distinguishable structure, and the delineator and road marking having enough length in order to separate traffic flow.
- In the case of an underpass, adjust the heights of lighting equipment in proportion to the level of road surface in order to get higher visual guidance in a vertical direction.

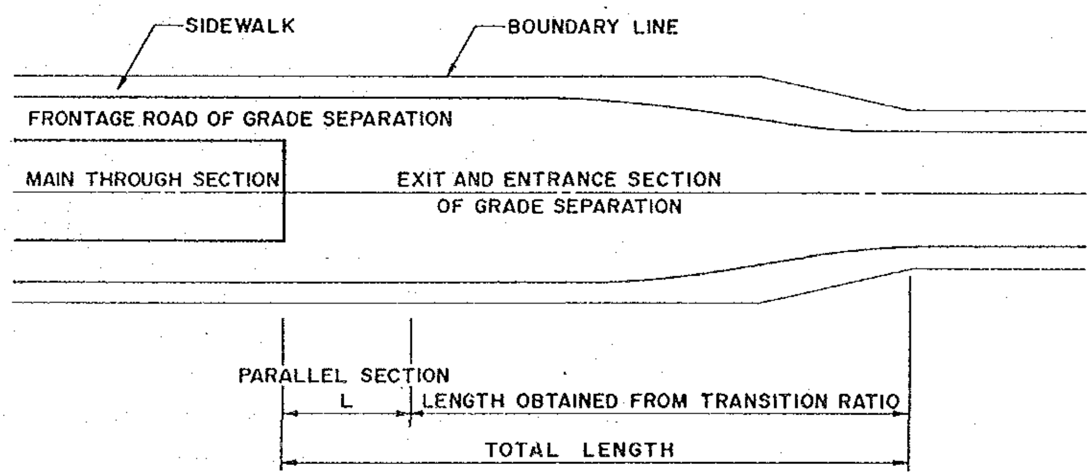


Figure 1.2.61 Transition Runoff at the Exit and Entrance Section of Grade Separation

1.3 Bridge Design

1.3.1 Scope of application

The guideline is to be applied for ordinary bridges on BMA roads, where spans are not longer than 150m. Supplemental specifications may be required for unusual types and for bridges with spans longer than 150m.

1.3.2 Loads

(1) General

1. The structure shall be designed to carry the following loads and forces.
   a) Dead load (D)
   b) Live load (L)
   c) Impact or dynamic effect of live load (I)
   d) Wind loads (W)
   e) Other forces, when they are expected to exist, such as longitudinal forces, centrifugal force, thermal forces, earth pressure, buoyancy shrinkage stresses, rib shortening, erection stresses, current pressure, and earthquake stresses.

The Bangkok Metropolitan Administration adopts AASHTO's standard (Standard Specification for Highway Bridges; the American Associations of State Highway and Transportation Officials) in principle.

As prescribed in the section 1.1.1 "Collection and Review of Relevant Reference Materials in Thailand", BMA and other agencies mainly have been using the AASHTO's standard. The engineers of BMA are familiar with the AASHTO's standard. For these reasons, the design loads of bridges are determined based on AASHTO's standard. However, in the section 1.2 "Road Planning", the guideline proposed is mainly based on Japanese standards due to choice of the "Design Vehicle" to be used. As for the loads, the difference between AASHTO's standard and Japanese standard is small and, thus, has little effect on structures' size. It is not recommended to change the design practice of bridges to the one's similar to those of Japan immediately. However, it may be desirable, from viewpoint of uniformity, that design loads similar to Japanese standard be adopted in the future, as far as dimensions of design vehicles are determined based on those of Japanese cars.

(2) Dead load

1. The unit weight as given in Table 1.3.1 may be used for calculation of the dead load. However, actual weight shall be used if it is known.

Table 1.3.1 Unit Weights of Materials

Material	Unit Weight	
	Kg/m^3	(#/cu.ft)
Steel or case steel	7,850	(490)
Cast iron	7,200	(450)
Aluminum alloys	2,800	(175)
Timber (treated or untreated)	800	(50)
Concrete, plain or reinforced	2,400	(150)
Compacted sand, earth, gravel or ballast	1,900	(120)
Loose sand, earth, and gravel	1,600	(100)
Macadam or gravel, rolled	2,250	(140)
Cindor filling	950	(60)
Pavement, other than wood block	2,400	(150)
Railway rails, guard rails and fastenings (per linear foot of track)	3,200	(200)
Stone masonry	2,700	(170)
Asphalt plank	1,750	9 lb.sq.ft./lin.thick

For reference, the dead load prescribed in the Japanese standard is shown in Appendix 1.3.1.

(3) Live load

1) Live load on the carriageways of bridges

1. Live loadings on the carriageways of bridges or incidental structures shall consist of standard trucks or lane loads that are equivalent to truck trains. Two systems of loadings are provided, the H loading and the HS loadings. The HS loadings are heavier than the corresponding H loadings.

2. Four classes of loading should be used according to the classification of the road;
H20, H15, HS20 and HS15 (AASHTO standard) are shown in Appendices 1.3.2, 1.3.3, and 1.3.4.

On the carriageway of bridges of three-classification Roads except Access Roads, H20 loading shall be placed.

On Access Roads, H15 loading shall be placed.

On the bridges of specific routes expected to carry heavy traffic volumes of semi-trailers, in Major Frunk Roads and/or Major Roads, HS loading must be used.

2) Live load on the sidewalk of bridges

1. Sidewalk floors, stringers and their immediate supports shall be designed for a live load of 415 Kg/m^2 of sidewalk area.

2. Girders, trusses, arches and other members shall be designed for the following sidewalk live loads:
Spans 0 to 7.5m in length ... 415 Kg/m^2
Spans 7.5 to 30.0m in length ... 300 Kg/m^2
Spans over 30.0m in length ..... according to the following equation.

$$P = (1575 + \frac{48000}{L}) (\frac{16.8 - W}{164})$$

where ; P = live load per square meters, max. 300 kg/m^2;
L = load length of sidewalk in meters;
W = width of sidewalk in meters

1.3.3 Cross Section

1. In an urban area, the standard widths of road elements of bridge cross sections should be the same values as those at ordinary road section. However, in unfavorable places, the widths may be reduced to the values in parentheses shown in Table 1.2.5.

2. In suburban and rural areas, the widths of each element may be reduced to the values in parentheses shown in Table 1.2.5. However, it is desirable that the widths shall be the same values as those at ordinary road section, if possible.

3. When the widths and combination of cross section elements of flyover and underpass are designed, they should be determined in consideration of standard widths shown in Table 1.2.5. The standard cross sections of flyover on Major Trunk Roads and Major Roads are shown in Figure 1.3.1 and 1.3.2.

In urban areas where traffic volume is heavy, it is not desirable to reduce bridge widths because to do so will create disturbance in traffic flow. Hence, widths of cross section elements of bridge should be the same to those at ordinary road section. In the Japanese standard, the reduced values are restricted to a bridge with spans longer than 50 m, and a tunnel with length longer than 100 m. Therefore, in the BMA area also, the reduced values may be limited to the locations of structures with above-mentioned dimensions or at unfavorable places.

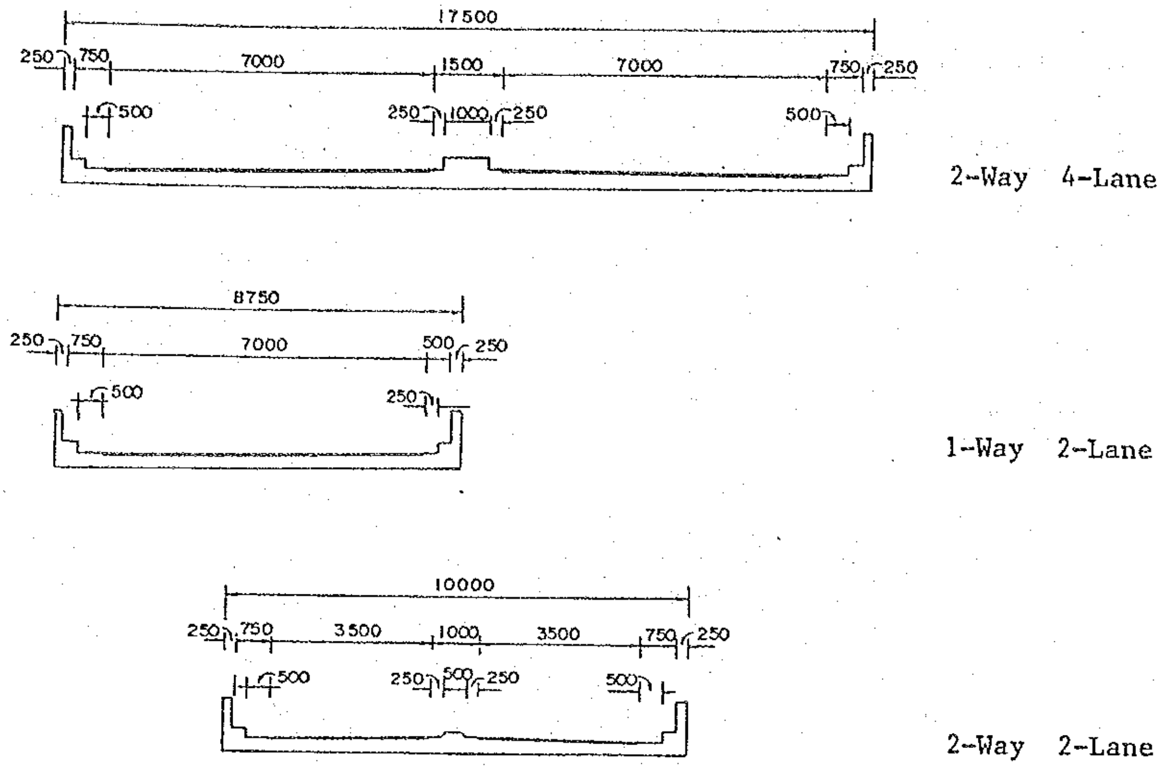


Figure 1.3.1 Standard Cross Section (FlyOver)
(Major Trunk Roads) Design Speed = 80km/h

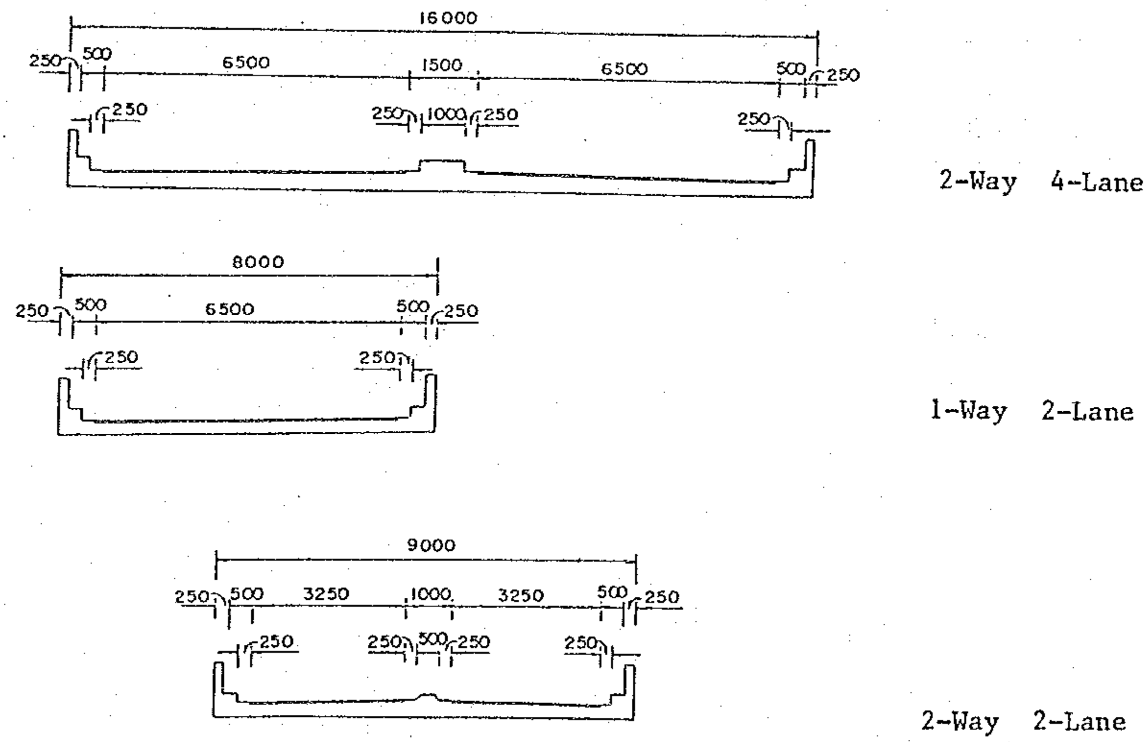


Figure 1.3.2 Standard Cross Section (Flyover)
(Major Roads) Design Speed = 60km/h

1.4 Traffic Capacity

1.4.1 Introduction

(1) Concept of capacity

This guideline defines capacity as "...the maximum rate of flow that can reasonably be expected to pass a point or uniform section of a lane or roadway under prevailing roadway, traffic, and control conditions."

(2) Level of service concept

The word "level of service" is frequently used in this guideline, and is defined as "....a qualitative measure describing operational conditions within a traffic stream, or their perception by motorists and/or passengers...," with these conditions generally described by "...such measures as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience, and safety".

(3) Unit of traffic capacity

It is fundamental that traffic capacity is described by "passenger car unit per hour" (pcu/hr). The passenger car equivalents (pce) for each vehicle quoted from studies by OCMRT and STTR are shown in Table 1.4.1.

Table 1.4.1 Passenger Car Equivalents for Each Vehicle

Type of Vehicle	Passenger-car equivalent (pce)
Car	1.0
Taxi	1.0
Mini Bus	1.5
Heavy Bus	2.1
Pick-up	1.0
Truck	2.5
Motorcycle	0.175

1.4.2 Basic Freeway Segments

(1) Level of service

1. Levels of service criteria for basic freeway segments are given in Table 1.4.2 for design speed element. Level of service is indicated by the ratio of the average travel speed to the design speed. The volume-to-capacity ratios and maximum service flow rates indicated in the table are expected to exist under ideal conditions.

Table 1.4.2 Levels of Service for Basic Freeway Segments

Level of Service	$V_D \gtrsim 80$ Km/h			$V_D \leq 60$ Km/h		
	V_A/V_D	v/c	MSF (PCPHL)	V_A/V_D	v/c	MSF (PCPHPL)
I	≧ 0.80	0.75	1,650	≧ 0.85	0.65	1,450
II	≧ 0.70	0.85	1,850	≧ 0.75	0.85	1,850
III	≧ 0.55	1.0	2,200	≧ 0.60	1.0	2,200
IV	< 0.55	*	*	< 0.60	*	*

V_A = Average travel speed (Km/h)

V_D = Design speed (Km/h)

MSF = Maximum service flow rate per lane under ideal conditions for multilane road

v/c = Maximum volume - to - capacity ratio associated with LOS

* = Highly variable, unstable

NOTE : All values of MSF rounded to the nearest 50 pcph

(2) Maximum service flow rate per lane

1. Table 1.4.2 presents criteria for maximum service flow rate, MSF, under ideal conditions.

2. These values are computed from the volume-to-capacity ratios, v/c, as follows, then rounded to the nearest 50 passenger car (unit) per hour per lane (pcphpl).

$$MSF_i = c_j \times (v/c)_i \quad .....(1)$$

where:

MSF_i = maximum service flow rate per lane for LOSi (Level of Service) under ideal conditions, in pcphpl;

$(v/c)_i$ = maximum volume-to-capacity ratio associated with LOSi;

c_j = capacity under ideal conditions for freeway element of the number of lanes j; 2200 (pcphpl) for multilane road, 2500 (pcph/two-way) for two-lane two-way road; the value of c_j

is synonymous with the maximum service flow rate for LOS III in Table 1.4.2.

(3) Service flow rate

1. Service flow rate for LOSi is obtained from the following equation;

$$SF_i = MSF_i \times N \times f_L \times f_c \times f_R \quad .....(2)$$

where :

SF_i = service flow rate for LOSi under prevailing roadway and traffic conditions for N lanes in one direction, in vph

$MSF_i = c_j \text{ X } (v/c)_i \quad .....(1)$

MSF_i = maximum service flow rate per lane for LOSi under ideal condition, in pcphpl;

N = number of lanes in one direction of the freeway

f_L = factor to adjust for the effect of restricted lane width

f_c = factor for lateral clearance

f_R = factor for roadside condition

Equations (1) and (2) can be combined as follows. The combined form is useful when a computation of SF is desired using v/c values directly, rather than MSF values.

$$SF_i = c_j \times (v/c)_i \times N \times f_L \times f_c \times f_R \quad ....(3)$$

These three basic relationship form the basis of all capacity analysis applications for basic freeway segments.

(4) Adjustments to maximum service flow rate

1) Adjustment for restricted lane width

1. The MSF for any freeway segment with lane width narrower than 3.25 m is adjusted to reflect this prevailing condition using the factor f_L.

2. When the lane width is narrower than 3.25 m, the factor f_L is calculated from the following equation;

$$f_L = 0.24\, W_L + 0.22 \quad .....(4)$$

where:

f_L = factor to adjust for the effect of restricted lane width W_L = lane width (m) $(2.5m \leqq W_L < 3.25m)$

Table 1.4.3 shows the values of the adjustment factor for lane width, f_L.

Table 1.4.3 Adjustment Factor for Lane Width, f_L

Lane Width W_L(m)	f_L
≧ 3.25	1.00
3.00	0.94
2.75	0.88
2.50	0.82

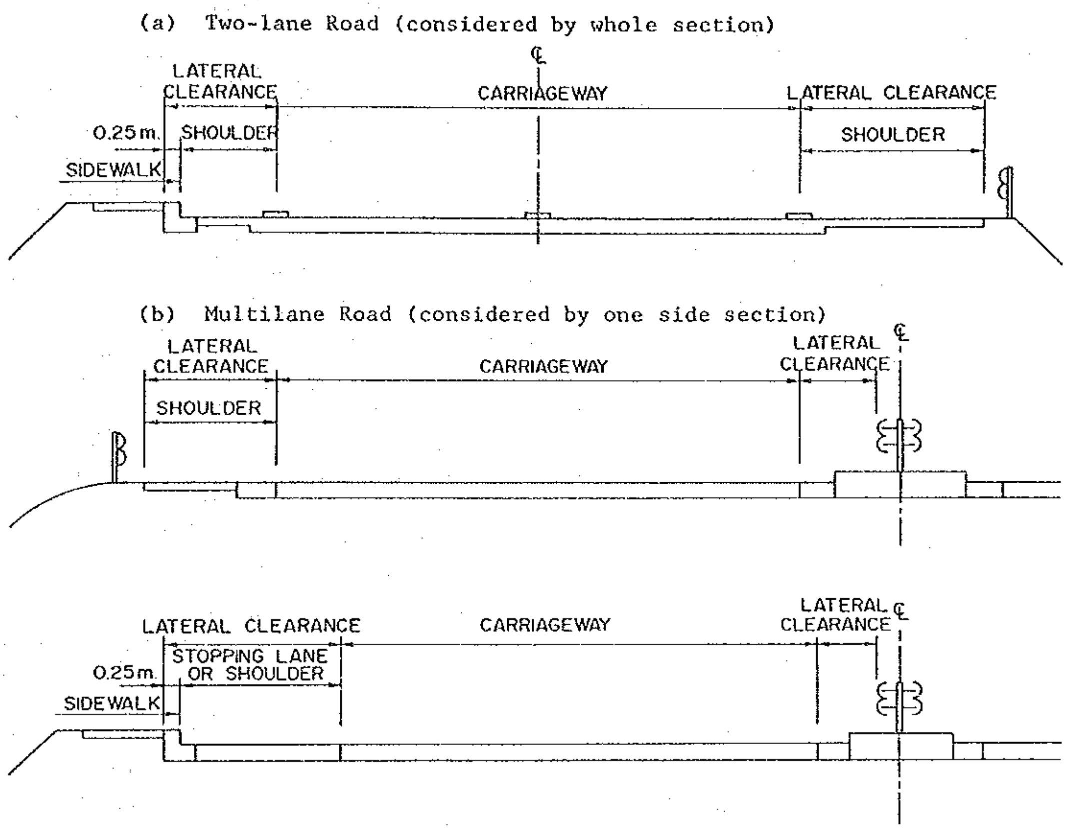
2) Adjustment for lateral clearance (f_c)

1. The MSF for any freeway segment with objects closer to the edge of the travel lanes than 0.75m (at the roadside and/or in the median) is adjusted to reflect this prevailing condition using the factor f_c. 2. The values of the factor for lateral clearance, f_c, are given in Table 1.4.4.

Figure 1.4.1 shows the method of measuring the width of lateral clearance.

Table 1.4.4 Adjustment Factor for Lateral Clearance, f_c

Lateral Clearance Wc(m)	f_c	
	Obstruction on One Side of the Carriageway	Obstruction on Both Sides of the Carriageway
≧ 0.75	1.00	1.00
0.50	0.98	0.95
0.25	0.95	0.91
0.00	0.93	0.86



Note : 1. In the case that there are obstructions on both sides of the carriageway, the average width of lateral clearance on both sides is adopted.

2. On the multilane roads having no median, the width of lateral clearance on the side of median is zero.

Figure 1.4.1 Method of Measuring the Width of Lateral Clearance

3) Adjustment for roadside condition, f_R

1. Values of MSF must be adjusted to reflect the prevailing conditions of parking and stopping. In the urbanized area, the conditions of parking and stopping have a great effect on the MSF. This adjustment is made using the factor f_R.

2. The factor f_R is given in Table 1.4.5 to take the effect of parking and stopping, and the urbanization of roadside areas into account.

Table 1.4.5 Adjustment Factor for Roadside Condition, f_R

a) Don't consider the effect of parking and stopping

Urbanization	f_R
not urbanized area	1.0
a little urbanized area	0.95
urbanized area	0.90

b) Consider the effect of parking and stopping

Urbanization	f_R
not urbanized area	0.95
a little urbanized area	0.85
urbanized area	0.75

4) Adjustments for others

The impact of heavy vehicles on traffic flow greatly relate to the grade conditions as well as the traffic composition. However, in the Bangkok Metropolitan Area, there are few grade sections, so the factor for heavy vehicles is not taken into account. If necessary, this adjustment for heavy vehicles can be made by referring to "Highway Capacity Manual, 1985, U.S.A." and/or "Highway Capacity Manual, Japan".

(5) Application of traffic capacity for planning and design of traffic capacity

1) Design hourly volume

1. Design hourly volume is the basic volume for planning and design of road.

2. Design hourly volume is obtained from design volume to consider characteristics of variation of traffic volume on the route.

3. It is a standard that design hourly volume is defined as the hourly volume of the first 30 hours at the design target year.

4. Design hourly volumes are calculated by the following equations.

   DHV = AADT x K (two-lane road) .....(5)

   DDHV = AADT x K x D (multilane road).....(6)

where:

DHV = the design hourly volume, in vph;
DDHV = the directional design hour volume, in vph;
AADT = the average annual daily traffic, in vph;
K = proportion of AADT occurring in the peak direction, and
D = proportion of peak-hour traffic in the peak direction

(6) Urban and Suburban Arterials

Urban and suburban arterials are defined as surface highways having signal intervals of less than 3 Km. Such facilities are characterized by platoon flow. Operational quality is controlled, and is affected by how individual signalized intersections operate along the arterial.

Arterials do not have a capacity per se. Mid-block sections and characteristics do not have a measurable effect on capacity, which is controlled by the capacity of signalized intersections along the arterial. Thus, this methodology depends on individual signalized intersection analyses, and is linked to the procedures of the Section, 1.4.3, (1) "Signalized intersection". For details refer to "Highway Capacity Manual, 1985, U.S.A.".

1.4.3 At-grade intersection

This section contains procedures for the analysis of signalized intersection capacity. The section does not contain the capacity of unsignalized intersections. The capacity analysis of unsignalized intersections is referred to "Highway Capacity Manual, 1985, U.S.A.".

(1) Signalized intersections

1) Basic approach

The methodology adopted for this guideline is based upon critical movement analysis. Critical movement analysis focuses on the identification and analysis of those movements in particular lanes which control the requirements for green time allocation. Once critical movements are identified and provided for, all other movements will operate as well or better.

It is important to note that capacity analysis computations for signalized intersections are reciprocal to signal timing computations.

2) Operational analysis

The procedure evaluates a broad range of factors affecting operations, falling generally into three categories; geometrics, signal control, and traffic.

Geometric factors include the number and use of lanes, lane widths, grades, location of bus stops, and similar factors. Control variables involve the type, phasing, and timing of traffic signal. Traffic factors include the usual composition variables, parking activity, pedestrian crossing flows, and vehicular flows by movement.

Because of the complexity of the analysis procedures, computations are done in a modular fashion, as depicted in Figure 1.4.2. The detailed flow chart of calculating capacity is shown in Figure 1.4.3.

1. INPUT MODULE
   . Geometric conditions
   . Traffic conditions
   . Signalization conditions

↓

2. SATURATION FLOW RATE MODULE
   . Ideal saturation flow rate
   . Adjustment

↓

3. CAPACITY ANALYSIS MODULE
   . Compute lane capacities
   . Aggregate results

Figure 1.4.2 Operational Analysis Procedure

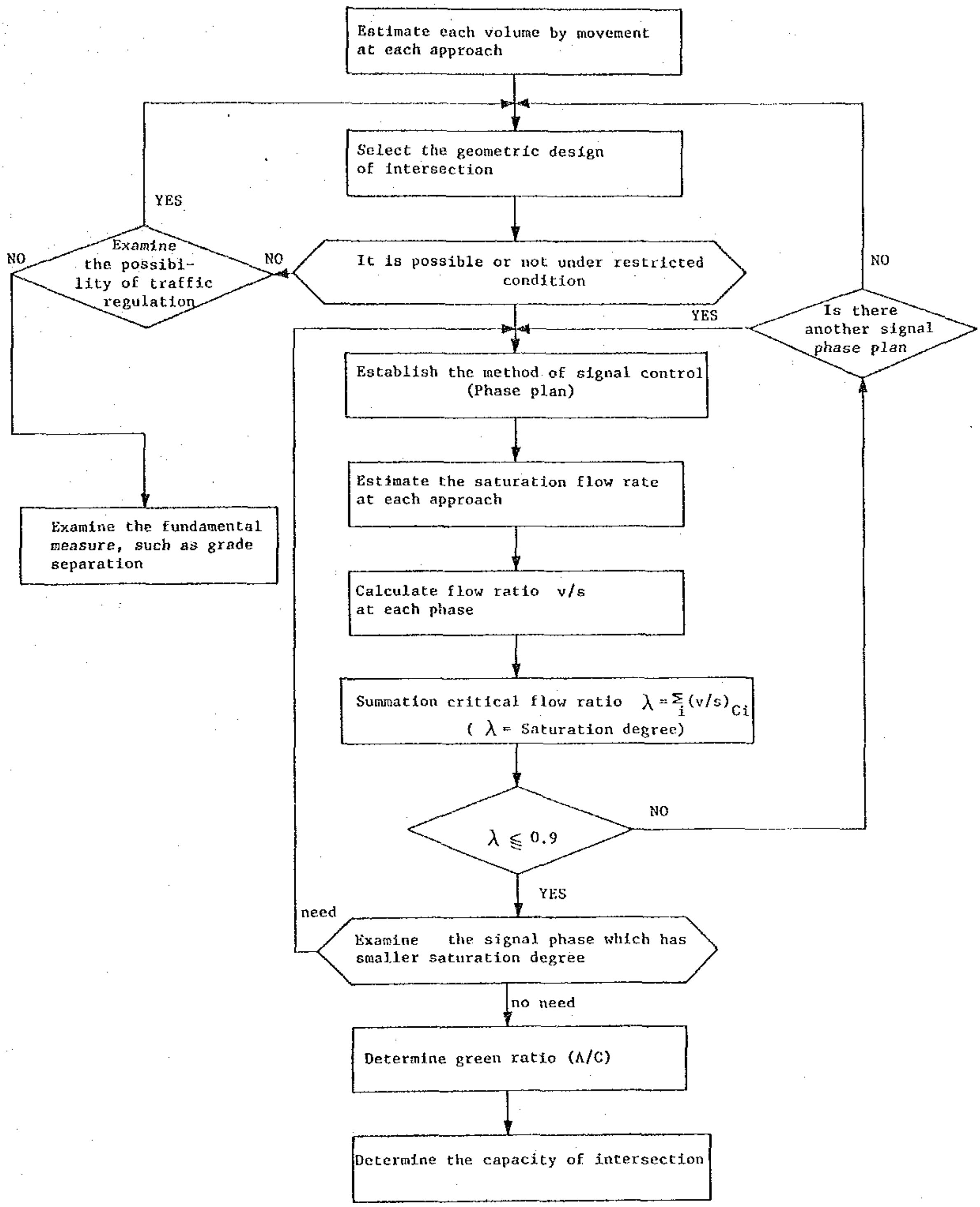


Figure 1.4.3 Detailed Flow Chart of Calculating Capacity

3) Saturation flow rate

1. The saturation flow rate can be obtained by multiplying the ideal saturation flow rate and the adjustment factors for a variety of prevailing conditions that are not ideal, as follows;

$$S = S_o \times N \times f_w \times f_g \times f_{HV} \times f_{RT} \times f_{LT} \quad .....(7)$$

where:
S = saturation flow rate under prevailing conditions, in vphg;
S_o = ideal saturation flow rate per lane, in pcphgpl
N = number of lanes,
f_w = adjustment factor for lane width,
f_g = adjustment factor for approach grade,
f_{HV} = adjustment factor for heavy vehicles in the traffic stream,
f_{RT} = adjustment factor for right turns in the shared lane,
f_{LT} = adjustment factor for left turns in the shared lane.

4) Ideal saturation flow rate

1. The values of "ideal" saturation flow rate are shown in Table 1.4.6.

Table 1.4.6 Ideal Saturation Flow Rate, So

Type of lane	Ideal saturation flow rate (pcphgpl)
Through	2,200
Left-turn	2,000
Right-turn	2,000

pcphgpl : passenger cars per hour of green time per lane

5) Adjustment factors

a) Adjustment for lane width, f_W

1. The lane width factor, f_W, accounts for the deleterious impact of narrow lanes on saturation flow rate, given in Table 1.4.7. Three-meters lane is the standard. As for right turn lane, when the width is 2.75 m or more, f_w is taken as 1.0.

Table 1.4.7 Adjustment Factor for Lane Width, f_W

Lane width	Lane Width Factor, f_W
$2.50 \leq W < 3.00$ m.	0.95
$3.00 \leq W < 3.50$ m.	1.00

b) Adjustment for approach grade, f_g

1. The grade factor, fg, accounts for the effect of grades on the operation of all vehicles, given in Table 1.4.8. The effects of heavy vehicles and grades are treated by separate factor, f_{HV} and f_g respectively. Their separate treatment recognizes that passenger cars are affected by approach grades, as are heavy vehicles.

Table 1.4.8 Adjustment Factor for Grade, f_g

Verticle grade (present)	Grade Factor, f_g
-6	0.95
-5	0.96
-4	0.97
-3	0.98
-2	0.99
-1	1.00
0	1.00
1	1.00
2	0.95
3	0.90
4	0.85
5	0.80
6	0.75

c) Adjustment for heavy vehicles, f_{HV}

1. The heavy vehicles factor, f_{HV}, accounts for the additional space occupied by these vehicles and for the differential in the operating

capabilities of heavy vehicles with respect to passenger cars, given in Table 1.4.9.

Table 1.4.9 Adjustment Factor for Heavy Vehicles, f_{HV}

Precent Heavy vehicles (% HV)	Heavy Vehicle Factor f_{HV}
5	0.95
10	0.91
15	0.87
20	0.83
25	0.80
30	0.77
35	0.74
40	0.71
45	0.69
50	0.67

or $f_{HV} = 1/(1+P_T(E_T-1))$

where

f_{HV} = adjustment factor for heavy vehicle

E_T = passenger-car equivalent for heavy vehicle (=2.0)

P_T = proportion of heavy vehicles in the traffic stream

d) Adjustment for right turns

The right-turn factor depends on a number of variables, including;
- Whether right turns are made from exclusive or shared lanes.
- Type of signal phasing (protected, permitted, or protected plus permitted).
- Proportion of right turning vehicles using a shared lane.
- Opposing flow rate when permitted right turns are made.

i) Exclusive right-turn lane for rotected phasing

1. The saturation flow rate of right-turn lane, S_{RO} is obtained using the ideal saturation flow rate, 2000 pcphgpl, and the adjustment factors.

$$S_{RO} = 2000 \times f_W \times f_g \times f_{HV} \quad .....(8)$$

where:
S_{RO} = saturation flow rate of right-turn lane, in pcphgpl

2. The right turn flow rate is given in the following equation.

$$V_{RT} = S_{RO} \times \frac{t_{RT}}{C} + K \times \frac{3,600}{C} \quad ....(9)$$

where:
V_{RT} = right-turn flow rate, in vph;
C = cycle length, in sec;
t_{RT} = green time for right turn, in sec;
K = vehicles turning during phase change, in veh (all-red period) (small-sized intersection...2 vehicles, large-sized intersection...3 vehicles)

ii) Exclusive right-turn lane for permitted phasing

1. Right-turn vehicles can make turning using the gap of opposing through traffic

$$V_{RT} = S_{RO} \times f \times \frac{S_{op} \times g - V_o \cdot C}{S_{op} - V_o} \times \frac{1}{C} + K \times \frac{3,600}{C} \quad .....(10)$$

where:
V_{RT} = right-turn flow rate, in vph;
S_{op} = saturationflow ratefor opposing through approach, in vphg;
V_o = opposing through flow rate, in vph;
g = effective green time, in sec;
f = probability of making right turning using the gap (See Table 1.4.10).

Table 1.4.10 Probability of Marking Right Turning Using the Gap

V_o	0	200	400	600	800	1000	> 1000
f	1.00	0.81	0.65	0.54	0.45	0.37	0

iii) Right-turning vehicles using a shared lane for permitted phasing

1. Equivalent of right turn vehicle to through vehicle is calculated by the following equation, when S_{RO} = 2000 in the equation (10).

$$E_{RT} = \frac{2,200\ g/c}{2,000\ f\ (S_{op}.g - V_o C)/[C(S_{op} - V_o)] + 3,600\ K/C}$$

$$= \frac{1.1}{f \times (S_{op}.g - V_o C)/(g(S_{op} - V_o)) + 1.8\ K/G} \quad .....(11)$$

where:

E_{RT} = equivalent ofrightturnvehicleto through vehicle

2. The adjustment factor for right turns, f_{RT}, is given in the following equation.

$$f_{RT} = 1\ /\ (1 + P_R\ (E_{RT} - 1)) \quad .....(12)$$

where:

f_{RT} = adjustment factor for right turn
E_{RT} = equivalent ofrightturnvehicleto through vehicle
P_R = proportion of right turns in shared lane

In the case of no opposing through flow rate, put v_o=0 and f=1 in eq(11), and E_{RT} becomes;

$$E_{RT} = \frac{2,200\ g/c}{2,000\ g/c + 3,600\ K/C} \quad .....(13)$$

e) Adjustment for left turns

The left-turn factor, depends on a number of variables, including;

- Whether left turns are made from a exclusive or shared lane.
- Type of signal phasing (protected, permitted, or protected plus permitted); a protected left-turn phase has no conflicting pedestrian movements.
- Volume of pedestrians using the conflicting crosswalk.
- Proportion of left turns using a shared lane.
- Proportion of left turns using the protected portion of a protected plus permitted phase.

Where left-turn-on-red (LTOR) is permitted, the left-turn volume may be reduced by the volume of left-turning vehicles moving on the red phase. This is generally done on the basis of hourly volumes, before converting to flow rates.

i) Exclusive left-turn lane for protected phasing

1. The saturation flow rate of left-turn lane, S_{LO} is obtain by the same way as in the case of right-turn lane. However, the adjustment factors should be applied when the turning angle is sharp or width of lane is not sufficient for especially large-sized vehicles.

$$S_{LO} = 2{,}000 \times f_W \times f_g \times f_{HV} \qquad .....(14)$$

where:

S_{LO} = saturation flow rate of left-turn lane, in pcphgpl

2. The left-turn flow rate is given in the following equation.

$$V_{LT} = S_{LO} \times \frac{t_{LT}}{C} \qquad .....(15)$$

where:

V_{LT} = left-turn flow rate, in vph;
C = cycle length, in sec.
t_{LT} = green time for left-turn, in sec.

ii) Exclusive left-turn lane for permitted phasing.

1. Generally, left-turn vehicle flow is affected by pedestrian crossing which is given green phase at the same time. The left-turn flow rate is given in the following equation.

$$V_{LT} = \frac{S_{LO} \times \left[(1 - f_{pe}) \times g_{pe} + (g - g_{pe})\right]}{C} \qquad .....(16)$$

wnere:

V_{LT} = left turn flow rate, in vph;
S_{LO} = saturation flow rate of left turn lane, in pcphgpl;
g = effective green time, in sec;
g_{pe} = green time given to pedestrian crossing, in sec; ($g_{pe} \leqq g$)
f_{pe} = reduction rate of left turning due to pedestrian crossing.

2. The adjustment factor for pedestrian crossing, f_{Lpe}, is obtained from the following equation, when S_{LO} = 2,000 (pcphgpl).

$$f_{Lpe} = \frac{V_{LT}}{2{,}000\ g/c} = \frac{(1 - f_{pe})\ g_{pe} + (g - g_{pe})}{g} \qquad .....(17)$$

when the following assumptions, $g-g_{pe}$ = 5 sec, f_{pe} = 0.15, f_{pe} = 0.50, are put in the abovementioned equation, the pedestrian crossing factor, f_{Lpe}, is given in Table 1.4.11.

Table 1.4.11 Adjustment Factor for Pedestrian Crossing, f_{Lpe}

Effective green time g (seconds)	f_{Lpe}	
	Light pedestrian volume $f_{pe} = 0.15$	Heavy pedestrian volume $f_{pe} = 0.50$
20	0.89	0.63
30	0.88	0.58
40	0.87	0.56
50	0.87	0.55
60	0.86	0.54

iii) Left-turn vehicles using a shared lane

1. Equivalent of left turn vehicle to through vehicle is calculated by the following equation, when $S_{LO} = 2000$ is put into the equation (16).

$$E_{LT} = \frac{2{,}200\ g/c}{V_{LT}} = \frac{2{,}200\ g/c}{2{,}000\ \left[(1 - f_{pe})\ g_{pe} + (g - g_{pe})\right] / c}$$

$$= \frac{1.1\ g}{(1 - f_{pe})\ g_{pe} + (g - g_{pe})} \quad .....(18)$$

2. The adjustment factor for left turns, f_{LT}, is given in the following equation.

$$f_{LT} = 1/\left[1 + P_L\ (E_{LT} - 1)\right] \quad .....(19)$$

where:

f_{LT} = adjustment factor for left turn

E_{LT} = equivalentof left turn vehicleto through vehicle.

P_L = proportion of left turns in shared lane.

When pedestrian volume is small and negligible, in Table 1.4.12.

Table 1.4.12 Adjustment Factor for Left Turn, f_{LT}
(In case of no effect of pedestrian)

Left turn vehicle rate (%)	f_{LT}
5	0.99
10	0.97
15	0.96
20	0.94
25	0.93
30	0.91
35	0.90
40	0.88
45	0.87
50	0.85

Table 1.4.12 also gives representative f_{LT} for rather heavy pedestrian volume conflicting left turning, which are obtained from eq.(19) by giving 0.50 and 5 seconds for f_{pe} and $(g-g_{pe})$ respectively in eq.(18).

Table 1.4.13 Adjustment Factor for Left Turn, f_{LT}
(Heavy Pedestrian Volume)

Left turn vehicle rate (%)	Green time (second)				
	20	30	40	50	60
5	0.96	0.96	0.95	0.95	0.95
10	0.93	0.92	0.91	0.91	0.91
15	0.90	0.88	0.87	0.87	0.87
20	0.87	0.85	0.84	0.83	0.83
25	0.84	0.82	0.81	0.80	0.80
30	0.81	0.79	0.78	0.77	0.76
35	0.79	0.76	0.75	0.74	0.73
40	0.77	0.74	0.72	0.71	0.71
45	0.75	0.72	0.70	0.69	0.68
50	0.72	0.69	0.68	0.67	0.66

TECHNICAL GUIDELINE
FOR
TRAFFIC SAFETY DEVICES

2. TECHNICAL GUIDELINE FOR TRAFFIC SAFETY DEVICES

2.1 Introduction

In general, the traffic safety can be promoted from the highway/traffic engineering point of view, through the improvements of traffic environment based on the following principles:

- To separate the conflicting traffics by time and/or by space.
- To simplify the traffic flows.
- To create the proper driving circumstances.

Since traffic accidents, in nature, cannot be avoided completely, it is also an important principle in the traffic safety improvement to mitigate the accident severity.

To accomplish traffic safety improvement based on these principles, two engineering approaches should be harmonized appropriately; one is the installation of road appurtenances for traffic control, and the other is the improvement of road itself. Table 2.1.1 is the list of typical measures for traffic safety improvement classified in compliance with the above mentioned two approaches. These measures should be selected and applied with a due consideration.

These measures prove to be very effective in safety improvement particularly when they are appropriately planned and accompanied by adequate educational programs and the strict enforcement of laws and regulations to the road users. On the other hand, improper installation of them are not only ineffective but sometimes even causes danger because they may induce uneasiness to road users. Therefore, it is very important to prepare the guidelines on the traffic safety improvement (they are referred to as traffic safety devices for practical purpose hereinafter). Traffic safety devices in this study include not only the road appurtenances but the portion of road structure such as sidewalk.

Moreover, some of the traffic safety devices such as traffic sign, pavement marking, traffic signal are intended to convey particular "message" to the drivers and pedestrians. This also inevitably leads to the necessity of establishing technical guidelines which unify the application and result in the maximum effectiveness of these devices.

Table 2.1.1 Traffic Safety Improvement Measures

Principle of Safety Improvement	Traffic Safety Improvement Measures: Installation of Road Appurtenances	Traffic Safety Improvement Measures: Improvement of Road
To separate the conflicting traffics by time and/or by space	Traffic Signal Stop Control (Sign, Marking) Guardfence Longitudinal Pavement Marking Raised Pavement Markers Crosswalk	Construction of By-pass and Expressway Sidewalk Bicycle Path Bicycle-Pedestrian Path Pedestrian Overpass Median Frontage Road Refuge Island
To simplify the traffic stream	Channelization of Intersection Pavement Markings Guide Signs Access Control One-Way System Parking Restriction	Bus Bay Grade Separation Traffic Island
To create proper driving circumstances	Highway Lighting Post Delineator Curve Mirror Warning Signs Guide Signs Glare Screen Traffic Information system	Elongation of Sight Distance Improvement of Shoulders Road Geometry Improvement (Alignment, Cross-Section) Anti-Skid Treatment Pavement Leveling
To mitigate the accident severity	Guardfence Speed Control Overtaking Control Breakaway Treatment of Roadside Appurtenance	Pavement Grooving Side Slope Flattening

Note; Measures listed in the table are not all-inclusive.

Classification was made according to the principal facet of each measure.

2.2 Preparation of Technical Guidelines

The technical guidelines for the uniform and effective application of traffic safety devices should cover warranting condition of installation, installation planning, standard shape and dimension of safety devices, and operation and maintenance method. BMA and other relevant agencies have already developed the guidelines for several essential safety devices. In addition, the guidelines for the following devices were proposed to DOH through the "Traffic Safety Plan for Roads in the Kingdom of Thailand", a project conducted by JICA.

- Traffic signal.
- Guardfence.
- Lighting.
- Delineator.
- Sidewalk and bicycle path.
- Crossing facility for pedestrians.

Since it is desirable to utilize the uniform guideline on the safety devices for the whole country, the technical guidelines on the following safety devices are prepared in this report, mainly based on the proposed guidelines for DOH by the previous JICA Study.

- Guardfence.
- Delineator.
- Sidewalk.
- Crossing facility for pedestrians.

In addition, the existing technical guidelines on the following safety devices are reviewed after the consultation with BMA as well as the relevant agencies.

- Traffic signal.
- Traffic signs.
- Pavement markings.
- Street lighting