TOPIC 5

Intersection Design and Control

General Introduction

- An intersection is an area shared by two or more roads
- Main function is to allow the change of route directions
- It is an area of decision for all drivers and thus requires additional effort
- Intersections normally perform at levels below those of the rest of the street or highway and thus control the quality of traffic flow

Types of Intersections

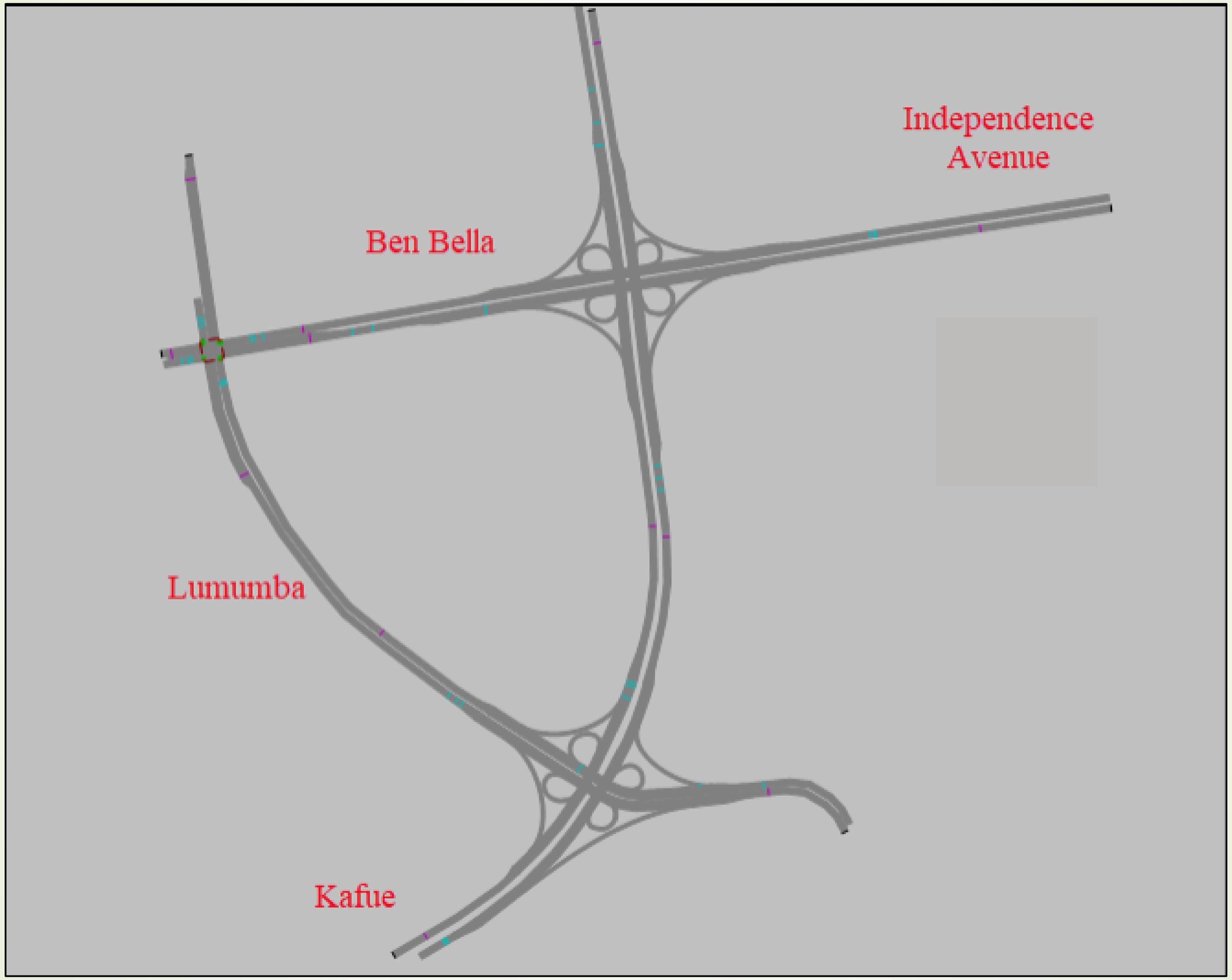
❖ Intersections can be classified as:

1. **Grade-separated without ramps:**
   - Structures allow traffic to cross at different levels
   - Uninterrupted traffic flow
   - Examples include the fly over bridges being constructed under the Lusaka City Decongestion Project

Types of Intersections

2. **Grade-separated with ramps:**
    – Interchanges

Types of Intersections



Types of Intersections

Types of Intersections

3. At-grade:

– All roads intersect at the same level

– Conflicts exist between the intersecting streams of traffic

– May be plain or channelized

– May be three-leg (T or Y), four-leg, or multi-leg

– Multi-leg intersections are not recommended

– Includes traffic circles and roundabouts

Types of Intersections

Types of At-Grade Intersections

(TAC 2017)

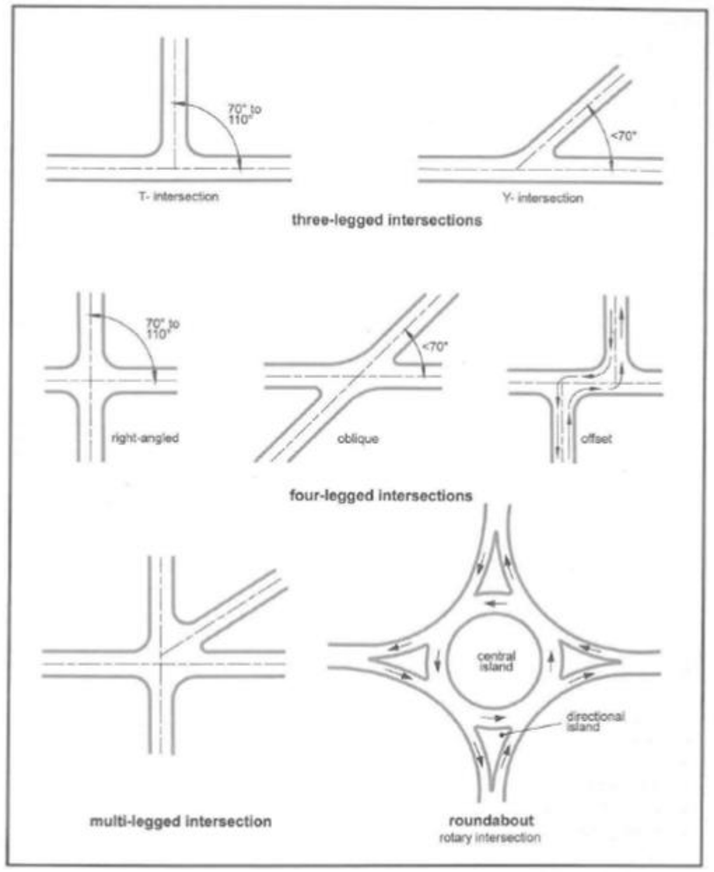
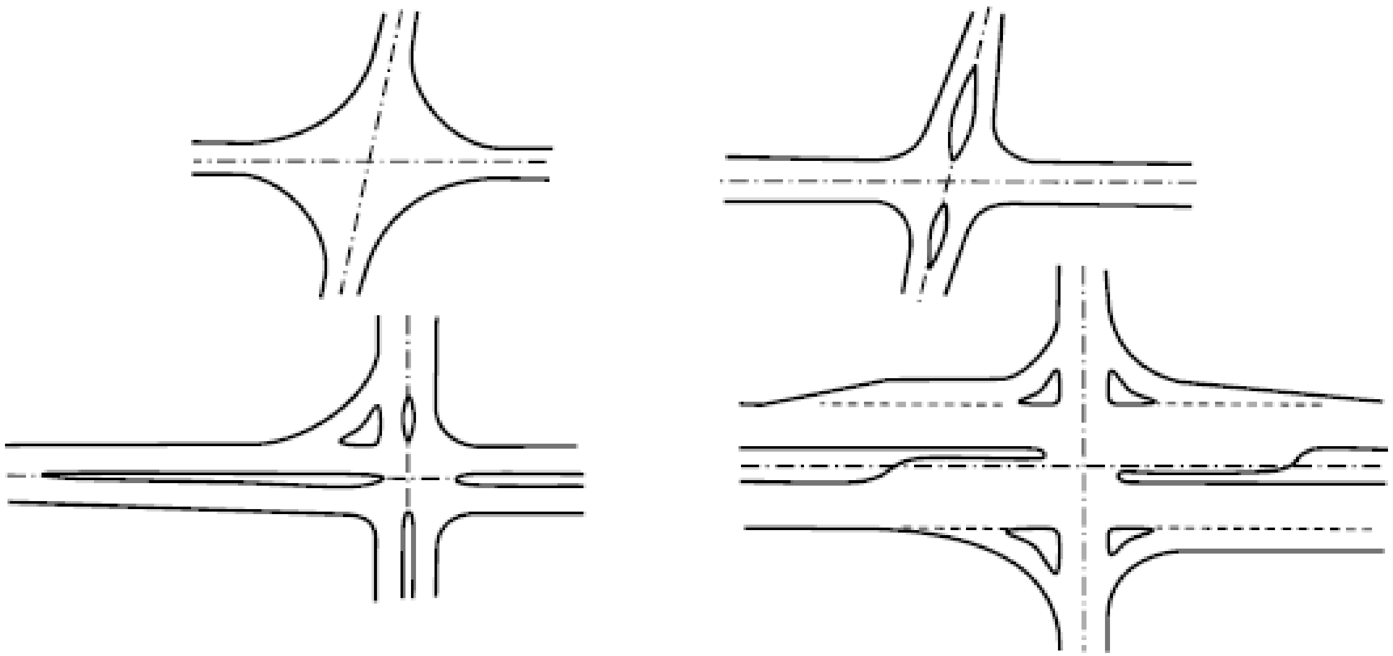


Figure 9.2.1: Intersection Configurations

Types of Intersections

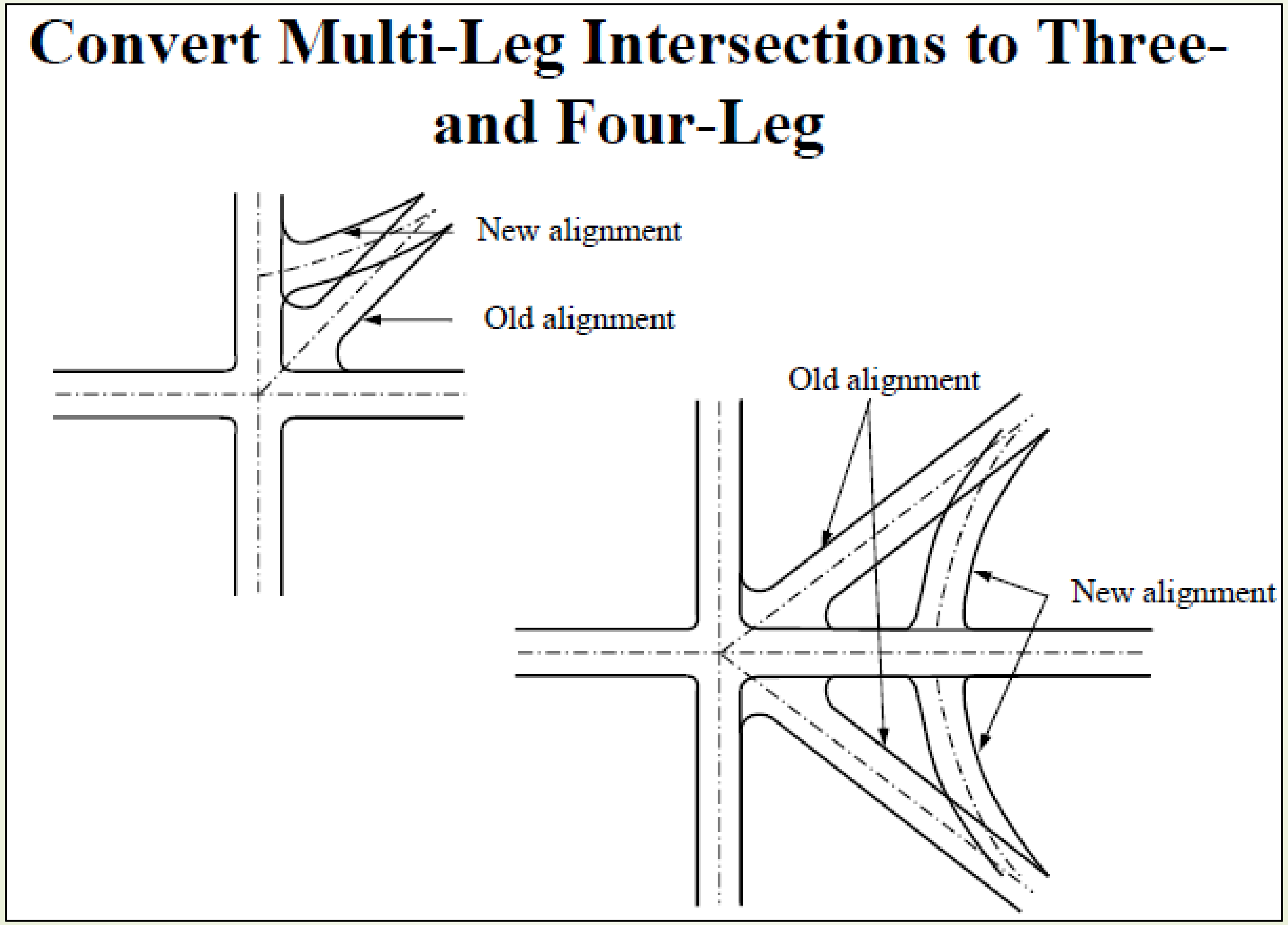
Types of At-Grade Intersections

Plain vs Channelized At-Grade Intersections



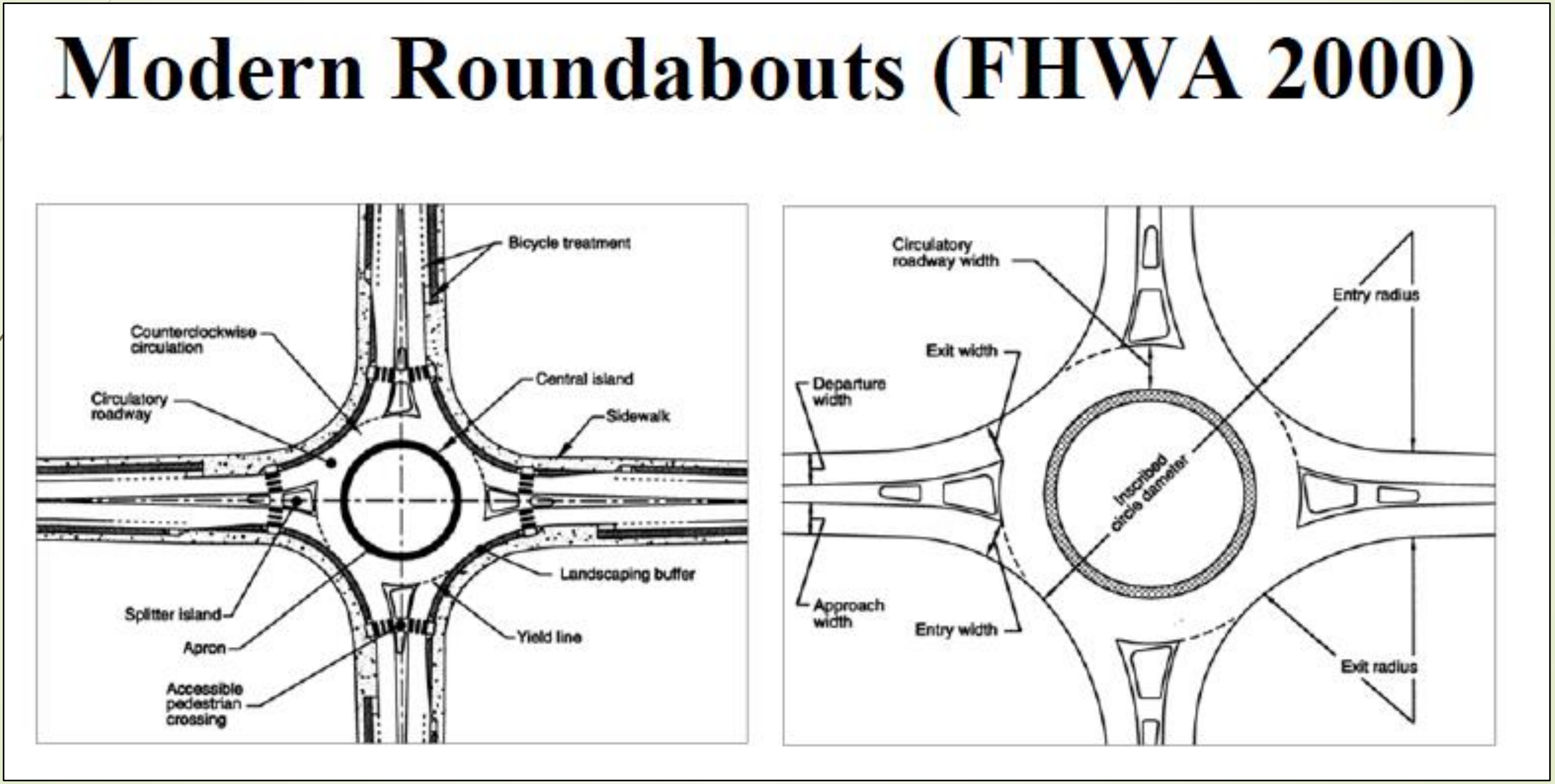
Types of Intersections

Types of At-Grade Intersections



Types of Intersections

Types of At-Grade Intersections



General Concepts of Traffic Control

- The purpose of traffic control is to assign the right of way to drivers, and thus to facilitate highway safety by ensuring the orderly and predictable movement of all traffic on highways

- Control can be achieved by using traffic signals, signs, or markings that regulate, guides, warning signs, and/or channel traffic

- A traffic control device must:
  - – Fulfill a need
  - – Command attention
  - – Convey a clear simple meaning
  - – Command the respect of road users
  - – Give adequate time for proper response

General Concepts of Traffic Control

❖ For the traffic control device to have these five properties, five factors should be considered:

- Design:
- Size, color, shape, …
- Placement:
- Within the cone of vision with adequate response time
- Operation:
  - Used in a manner that ensures the fulfillment of traffic requirements
- Maintenance:
  - Regularly maintained to sustain legibility
- Uniformity: To ensure recognition and understanding of these devices

❖ Guidelines for the different types of traffic control devices are provided in the Manual on Uniform Traffic Control Devices (MUTCD)

Conflict Points at Intersections

- Conflicts occur when traffic streams moving in different directions interfere with each other
- Three types of conflicts:
  - – Merging
  - – Diverging
  - – Crossing
- The number of possible conflict points at any intersection depends on:
  - – Number of approaches
  - – Turning movements
  - – Type of traffic control

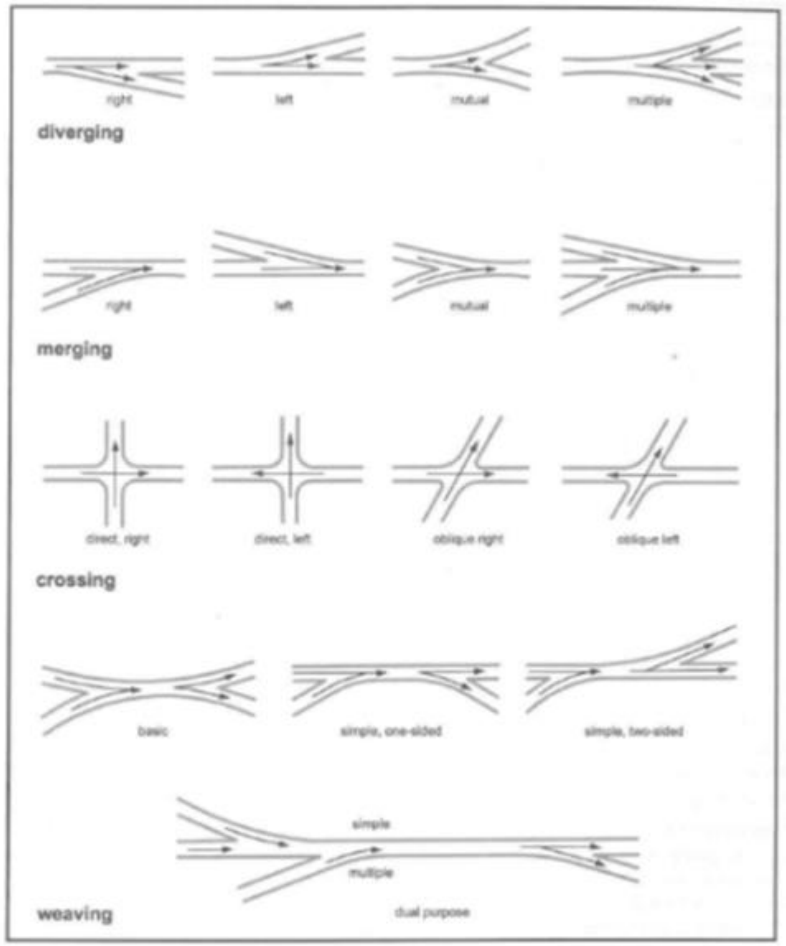
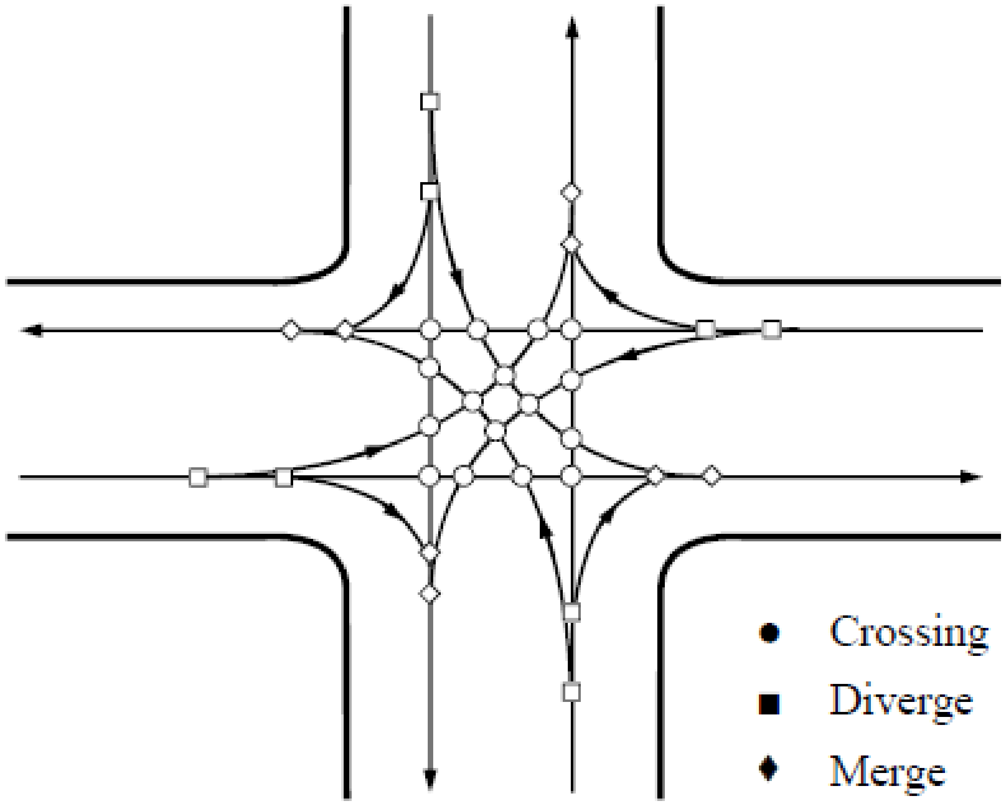


Figure 9.3.1: Typical Traffic Movements within an Intersection and its Approach

Conflict Points at Intersections

Example: conflict points at a four-leg intersection



●	Crossing	= 16
■	Diverge	= 8
♦	Merge	= 8
	Total	= 32

Types of Intersection Control

- The primary objective of a traffic control system at an intersection is to reduce the number of conflict points

- The choice of one method for traffic control at the intersection depends on many factors:
  - – Vehicle volume
  - – Turning movements
  - – Pedestrian volume
  - – Accident experience
  - – Delay
  - – Other considerations

- Warrants for the different types of traffic control devices are given in the MUTCD

Types of Intersection Control

❖ **The types of intersection control are:**

– Yield Signs:

– Drivers on approaches with yield signs are required to slow down and yield the right of way to all conflicting vehicles at the intersection

– Stopping is not mandatory unless it interferes with a traffic stream that has the right of way

– Stop Signs:

– Approaching vehicles are required to stop before entering the intersection

– Use of stop signs results in considerable inconvenience to motorists and thus must be used only when warranted

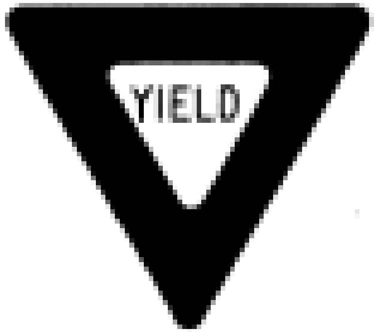
– Stop signs may be warranted at intersection with restricted view

– Multi-way Stop Signs:

– All vehicles approaching the intersection stop before entering it

– Used as a safety measure at some intersections with traffic volumes on all approaches are approximately equal

Types of Intersection Control



Yield Sign Sizes

Conventional Roads	36" × 36" × 36"
Expressways	48" × 48" × 48"
Freeways	48" × 48" × 48"
Minimum	30" × 30" × 30"

Stop Sign Sizes

Conventional Roads	30" × 30"
Expressways	36" × 36"
Minimum	24" × 24"

Types of Intersection Control

– **Intersection Channelization:**

– Used to separate turn lanes from through lanes

– Solid lines or raised barriers guide traffic within a lane so that vehicles can safely negotiate a complex intersection

– Raised islands can also provide a **refuge** for pedestrians

– **Traffic Signals:**

– Traffic signals are used to assign the use of the intersection to different traffic streams at different times, and thus eliminate many conflicts

– Efficient operation of a traffic signal requires proper timing of the different color indications

Geometric Design of Intersections

Design Objectives of Intersections:

– Minimize the severity of potential conflicts among different streams of traffic and between pedestrians and turning vehicles

– Incorporate the operating characteristics of both the vehicles and pedestrians using the intersection

– Ensure adequate pavement widths of turning roadways

– Ensure adequate approach sight distances

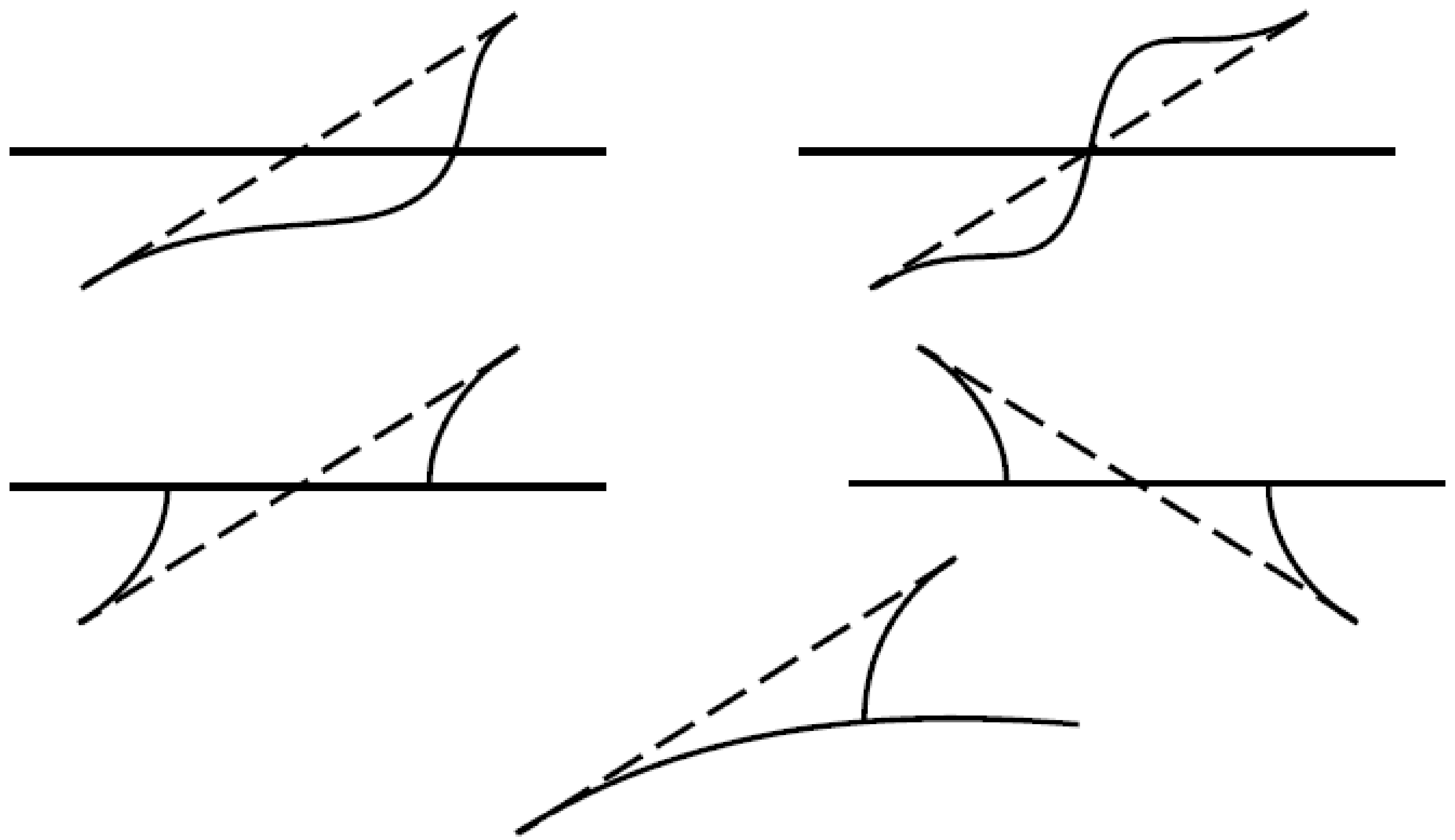
Geometric Design of Intersections

Alignment:

- ❖ Roads best intersect at right or nearly right angles
  - – Less road area required for turning
  - – Lower exposure time for vehicles crossing
  - – Visibility limitations not as serious as acute-angled
  - – More favorable condition for drivers to judge the relative position and speed of approaching vehicles
- ❖ It is recommended to realign the minor crossing road to avoid acute-angle intersections

Geometric Design of Intersections

Realignment of Acute-Angle Intersections



Geometric Design of Intersections

Figure 9.7.2 (TAC 2017)

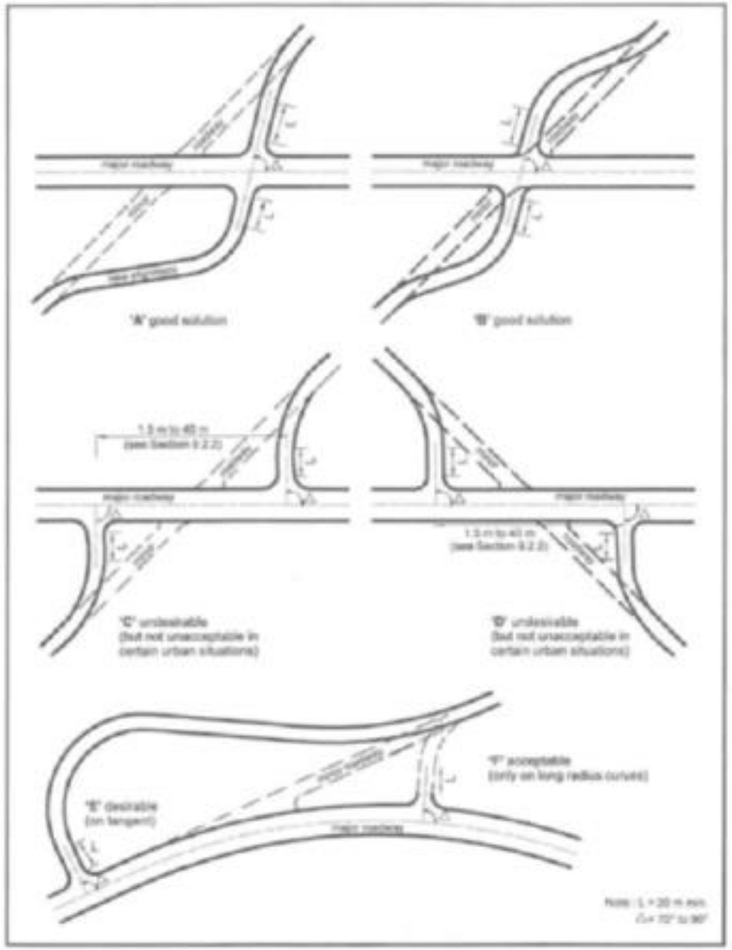


Figure 9.7.2: Examples of Realignment of Intersections

Geometric Design of Intersections

Vertical Profile:

- Large changes in grade should be avoided
  - – Grades should not be greater than 3%
  - – Stopping and acceleration distances are not very different from flat grades
  - – In any case, grades should not be higher than 6%

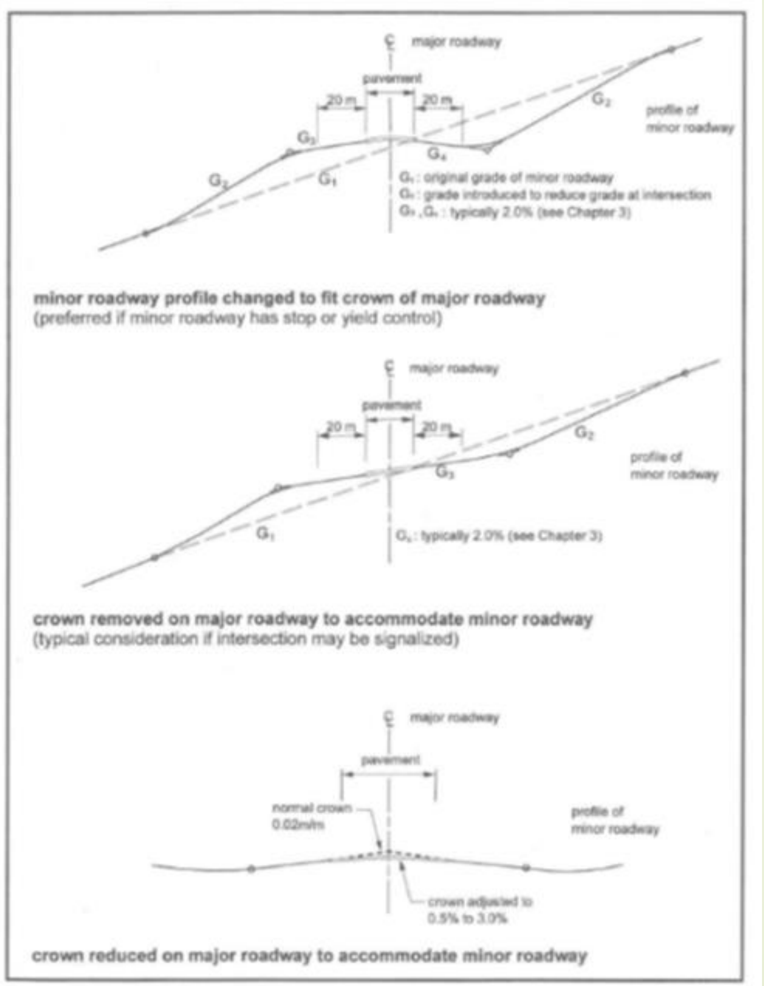


Figure 9.7.3: Fitting Minor Roadway Profiles to the Major Roadway Cross Section

Geometric Design of Intersections

Curves:

❖ Main factors governing the design of curves at at-grade intersections:
- Angle of turn
- Turning speed
- Design vehicle
- Traffic volume

❖ In an urban environment:
- Turning speed is assumed to be 20 km/h (15 mi/h) or less
- Radii need to conform to the minimum turning radius

❖ In a rural environment:
- Turning speed is assumed to be 30-40 km/h
- Curve should be designed for this speed

Geometric Design of Intersections

Table 7.2 (Garber & Hoel 2013)

Table 7.2 Edge-of-Traveled-Way design for Turns at Intersections - Simple Curve Radius with Taper

Angle of Turn (°)	*Design Vehicle*	*Simple Curve Radius (ft)*	*Simple Curve Radius with Taper*		
			Radius (ft)	*Offset (ft)*	*Taper L:T*
30	P	60	—	—	—
	SU-30	100	—	—	—
	SU-40	140	—	—	—
	WB-40	150	—	—	—
	WB-62	360	220	3.0	15:1
	WB-67	380	220	3.0	15:1
	WB-92D	365	190	3.0	15:1
	WB-100T	260	125	3.0	15:1
	WB-109D	475	260	3.5	20:1
45	P	50	—	—	—
	SU-30	75	—	—	—
	SU-40	115	—	—	—
	WB-40	120	—	—	—
	WB-62	230	145	4.0	15:1
	WB-67	250	145	4.5	15:1
	WB-92D	270	145	4.0	15:1
	WB-100T	200	115	2.5	15:1
	WB-109D	—	300	4.5	20:1
60	P	40	—	—	—
	SU-30	60	—	—	—
	SU-40	100	—	—	—
	WB-40	90	—	—	—
	WB-62	170	140	4.0	15:1
	WB-67	200	140	4.5	15:1
	WB-92D	230	120	5.0	15:1
	WB-100T	150	95	2.5	15:1
	WB-109D	—	180	4.5	20:1
75	P	35	25	2.0	10:1
	SU-30	55	45	2.0	10:1
	SU-40	90	60	2.0	10:1
	WB-40	—	60	2.0	15:1
	WB-62	—	145	4.0	20:1
	WB-67	—	145	4.5	20:1
	WB-92D	—	110	5.0	15:1
	WB-100T	—	85	3.0	15:1
	WB-109D	—	140	5.5	20:1
90	P	30	20	2.5	10:1
	SU-30	50	40	2.0	10:1
	SU-40	80	45	4.0	10:1
	WB-40	—	45	4.0	10:1
	WB-62	—	120	4.5	30:1
	WB-67	—	125	4.5	30:1
	WB-92D	—	95	6.0	10:1
	WB-100T	—	85	2.5	15:1
	WB-109D	—	115	2.9	15:1
105	P	—	20	2.5	8:1
	SU-30	—	35	3.0	10:1
	SU-40	—	45	4.0	10:1
	WB-40	—	40	4.0	10:1
	WB-62	—	115	3.0	15:1
	WB-67	—	115	3.0	15:1
	WB-92D	—	80	8.0	10:1
	WB-100T	—	75	3.0	15:1
	WB-109D	—	90	9.2	20:1
120	P	—	20	2.0	10:1
	SU-30	—	30	3.0	10:1
	SU-40	—	35	6.0	8:1
	WB-40	—	35	5.0	8:1
	WB-62	—	100	5.0	15:1
	WB-67	—	105	5.2	15:1
	WB-92D	—	80	7.0	10:1
	WB-100T	—	65	3.5	15:1
	WB-109D	—	85	9.2	20:1
135	P	—	20	1.5	10:1
	SU-30	—	30	4.0	10:1
	SU-40	—	40	4.0	8:1
	WB-40	—	30	8.0	15:1
	WB-62	—	80	5.0	20:1
	WB-67	—	85	5.2	20:1
	WB-92D	—	75	7.3	10:1
	WB-100T	—	65	5.5	15:1
	WB-109D	—	85	8.5	20:1
150	P	—	18	2.0	10:1
	SU-30	—	30	4.0	8:1
	SU-40	—	35	7.0	8:1
	WB-40	—	30	6.0	8:1
	WB-62	—	60	10.0	10:1
	WB-67	—	65	10.2	10:1
	WB-92D	—	65	11.0	10:1
	WB-100T	—	65	7.3	10:1
	WB-109D	—	65	15.1	10:1
180	P	—	15	0.5	20:1
	SU-30	—	30	1.5	10:1
	SU-40	—	35	6.4	10:1
	WB-40	—	20	9.5	5:1
	WB-62	—	55	10.0	15:1
	WB-67	—	55	13.8	10:1
	WB-92D	—	55	16.8	10:1
	WB-100T	—	55	10.2	10:1
	WB-109D	—	55	20.0	10:1

Table 7.2 Edge-of-Traveled-Way design for Turns at Intersections - Simple Curve Radius with Taper (continued)

SOURCE: From *A Policy on Geometric Design of Highways and Streets*, 2011, AASHTO, Washington, D.C. Used by permission.

Geometric Design of Intersections

Table 7.3 (Garber & Hoel 2013)

Table 7.3 Edge-of-Traveled-Way Design for Turns at Intersections – Three-Centered Curves

Angle of Turn (°)	Design Vehicle	*Three-Centered Compound*		*Three-Centered Compound*		Angle of Turn (°)	Design Vehicle	*Three-Centered Compound*		*Three-Centered Compound*	
		Curve Radii (ft)	*Symmetric Offset (ft)*	*Curve Radii (ft)*	*Asymmetric Offset (ft)*			*Curve Radii (ft)*	*Symmetric Offset (ft)*	*Curve Radii (ft)*	*Asymmetric Offset (ft)*
30	P	—	—	—	—	75	P	100-25-100	2.0	—	—
	SU-30	—	—	—	—		SU-30	120-45-120	2.0	—	—
	SU-40	—	—	—	—		SU-40	200-35-200	5.0	60-45-200	1.0–4.5
	WB-40	—	—	—	—		WB-40	120-45-120	5.0	120-45-195	2.0–6.5
	WB-62	—	—	—	—		WB-62	440-75-440	15.0	140-100-540	5.0–12.0
	WB-67	460-175-460	4.0	300-175-550	2.0–4.5		WB-67	420-75-420	10.0	200-80-600	1.0–10.0
	WB-92D	550-155-550	4.0	200-150-500	2.0–6.0		WB-92D	500-95-500	7.0	150-100-500	1.0–8.0
	WB-100T	220-80-220	4.5	200-80-300	2.5–5.0		WB-100T	250-80-250	4.5	100-80-300	1.5–5.0
	WB-109D	550-250-550	5.0	250-200-650	1.5–7.0		WB-109D	700-125-700	6.5	150-110-550	1.5–11.5
45	P	—	—	—	—	90	P	100-20-100	2.5	—	—
	SU-30	—	—	—	—		SU-30	120-40-120	2.0	—	—
	SU-40	—	—	—	—		SU-40	200-30-200	7.0	60-45-200	1.0–4.5
	WB-40	—	—	—	—		WB-40	120-40-120	5.0	120-40-200	2.0–6.5
	WB-62	460-240-460	2.0	120-140-500	3.0–8.5		WB-62	400-70-400	10.0	160-70-360	6.0–10.0
	WB-67	460-175-460	4.0	250-125-600	1.0–6.0		WB-67	440-65-440	10.0	200-70-600	1.0–11.0
	WB-92D	525-155-525	5.0	200-140-500	1.5–6.0		WB-92D	470-75-470	10.0	150-90-500	1.5–8.5
	WB-100T	250-80-250	4.5	200-80-300	2.5–5.5		WB-100T	250-70-250	4.5	200-70-300	1.0–5.0
	WB-109D	550-200-550	5.0	200-170-650	1.5–7.0		WB-109D	700-110-700	6.5	100-95-550	2.0–11.5
60	P	—	—	—	—	105	P	100-20-100	2.5	—	—
	SU-30	—	—	—	—		SU-30	100-35-100	3.0	—	—
	SU-40	—	—	—	—		SU-40	200-35-200	6.0	60-40-190	1.5–6.0
	WB-40	—	—	—	—		WB-40	100-35-100	5.0	100-55-200	2.0–8.0
	WB-62	400-100-400	15.0	110-100-220	10.0–12.5		WB-62	520-50-520	15.0	360-75-600	4.0–10.5
	WB-67	400-100-400	8.0	250-125-600	1.0–6.0		WB-67	500-50-500	13.0	200-65-600	1.0–11.0
	WB-92D	480-110-480	6.0	150-110-500	3.0–5.0		WB-92D	500-80-500	8.0	150-80-500	2.0–10.0
	WB-100T	250-80-250	4.5	200-80-300	2.0–5.5		WB-100T	250-60-250	5.0	100-60-300	1.5–6.0
	WB-109D	650-150-650	5.5	200-140-600	1.5–8.0		WB-109D	700-95-700	8.0	150-80-500	3.0–15.0

Geometric Design of Intersections

Channelization:

❖ It is the separation of conflicting traffic movements into definite paths of travel by traffic islands or pavement markings

❖ Proper channelization will result in:

– Increased capacity

– Enhanced safety for vehicles and pedestrians

– Increased driver confidence

Geometric Design of Intersections

❖ Factors influencing channelization design:

– Availability of right of way

– Design vehicle

– Expected vehicular and pedestrian volumes

– Cross sections of crossing roads

– Approach speeds

– Location and type of traffic control devices

❖ Minimum area for islands is preferably 10 m^2 but should not be less than 5 m^2 in urban areas or 7 m^2 in rural areas

– Minimum dimensions are also suggested based on the island's shape and function

Geometric Design of Intersections

Pavement width of turning roadways:

- ❖ For low turning speeds, use templates for minimum turning radius of the design vehicle

- ❖ For higher turning speeds, increase pavement width such that:

  – Case I: one-lane, one-way operation with no provision for passing a stalled vehicle

  – Case II: one-lane, one-way operation with provision for passing a stalled vehicle

  – Case III: two-lane operation, one-way or two-way

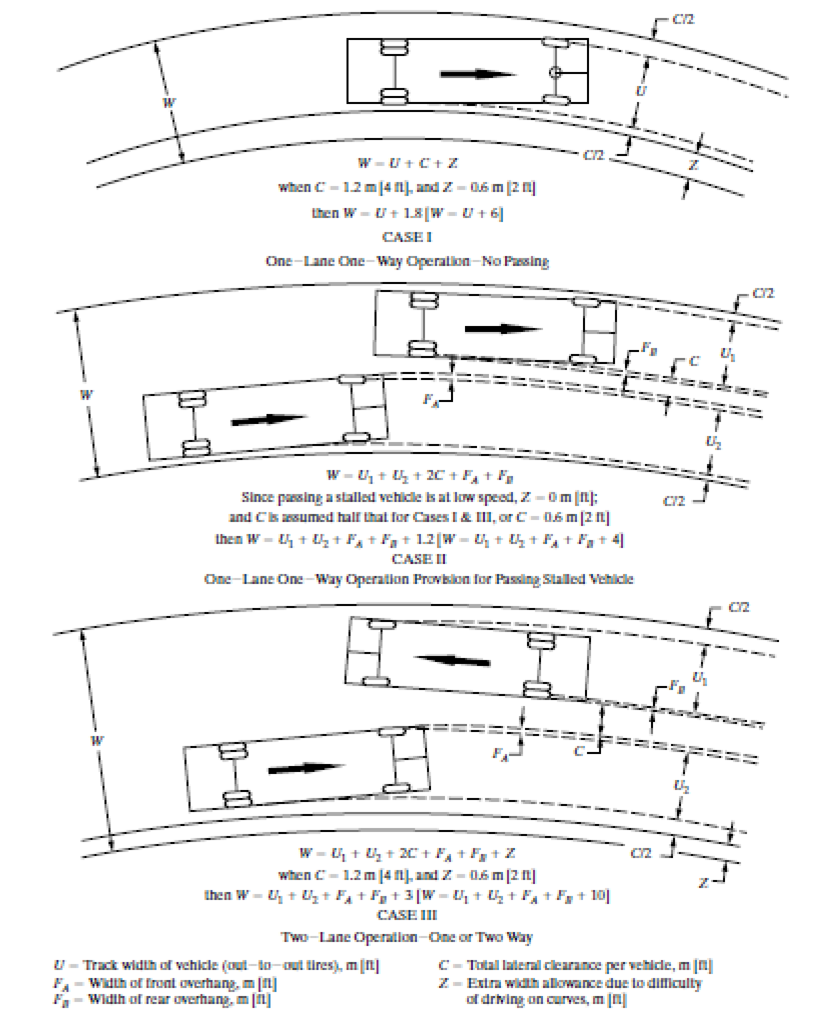
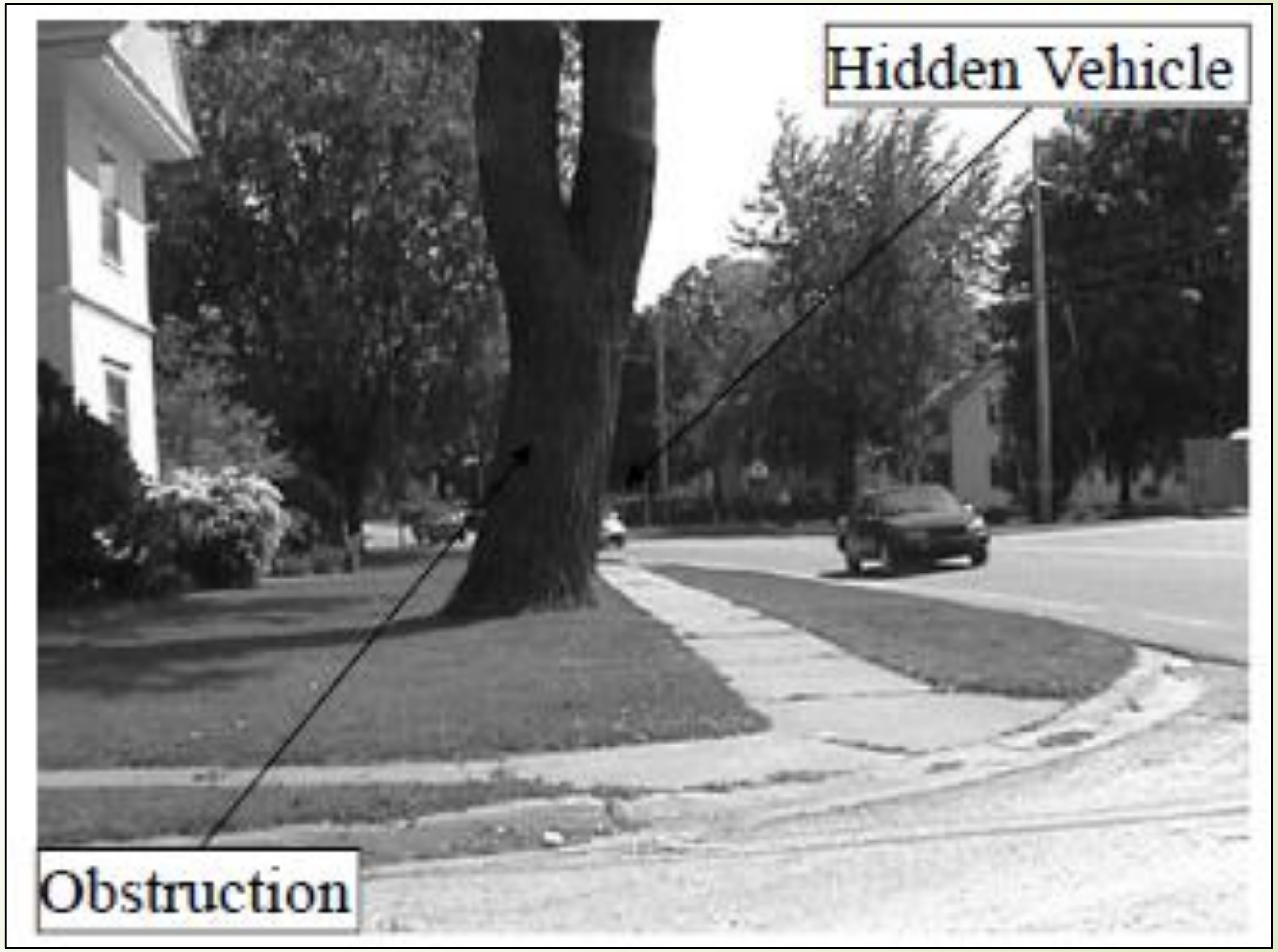


Figure 7.17 Basis for Deriving the Appropriate Pavement Width at Turning Roadways for Any Design Vehicle

Sight Distance at Intersections

- ❖ The high accident potential at an intersection can be reduced by providing sight distance that allows drivers to have an unobstructed view of the entire intersection at a distance great enough to permit control of the vehicle
- ❖ The required sight distance depends on the type of control at the intersection

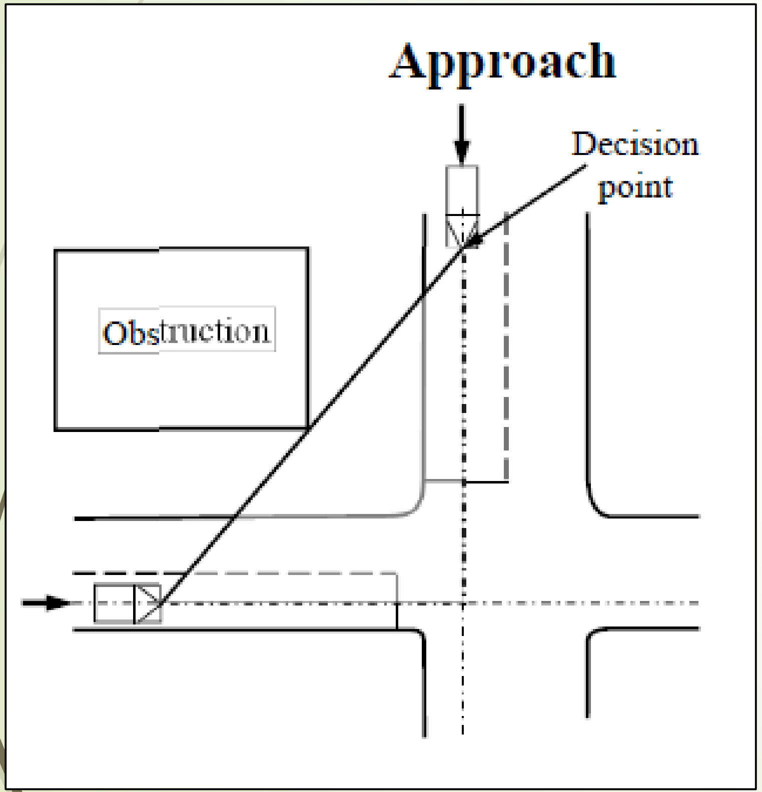


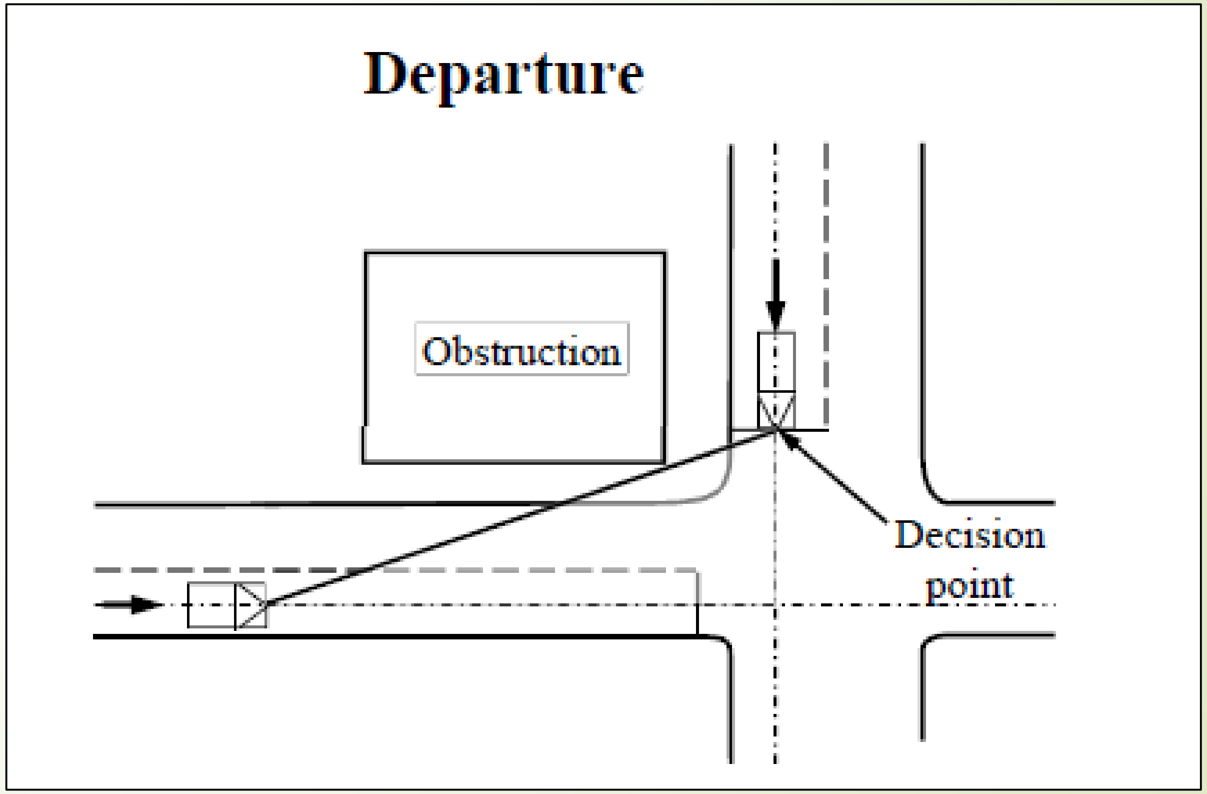
Sight Distance at Intersections

- Analysis of intersection sight distance is carried out using sight distance triangles

– Approach sight distance triangle

– Departure sight distance triangle





Sight Distance at Intersections

❖ According to AASHTO, there are six procedures to determine intersection sight distance based on the type of control:

– Case A: No control

– Case B: Stop control on minor road

– Case C: Yield control on minor road

– Case D: Signal control

– Case E: All-way stop control

– Case F: Left turn from major road

Sight Distance at Intersections

Case A: No Control Intersection

❖ The intersection is not controlled by a yield sign, stop sign, or traffic signal

❖ Historically, SD in this case needed to be enough for the driver approaching the intersection to:

– See a crossing vehicle

– If necessary, adjust vehicle speed

❖ This distance includes:

– The distance travelled by the vehicle during the perception-reaction time (usually 2 s)

– The distance travelled during brake actuation (usually 1 s)

Sight Distance at Intersections

- Therefore, the distance travelled during a period of 3 s was required on each approach

- This approach has updated based on the following:

  – Vehicles on the minor road are found to decelerate to around 50% of the midblock speed

  – Deceleration rate is estimated as 1.5 m/s^2(5 ft/s^2)

  – Then, time for detection and recognition of a conflicting vehicle is 2.5 s

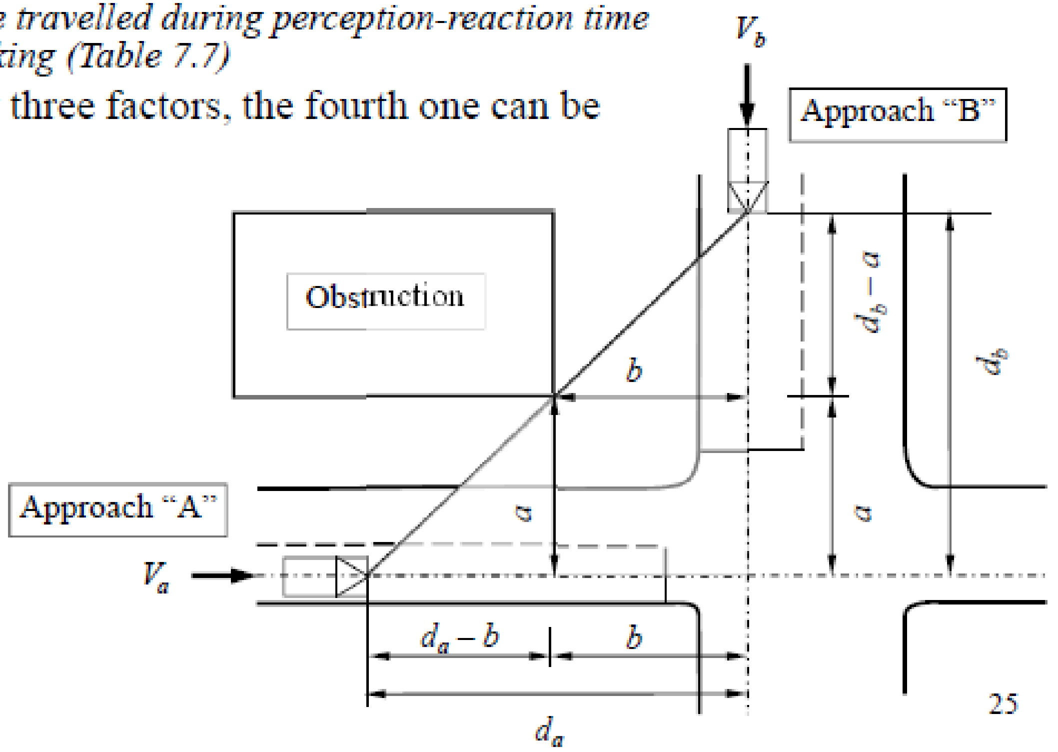
Sight Distance at Intersections

Sight distance triangle at intersection with no control:

- From the similarity of triangles:

  $$\frac{d_b}{d_a} = \frac{a}{d_a - b}$$

  - a, b = distances from the obstruction to the centreline of approaches A and B
  - V_a, V_b = speeds of vehicles on approaches A and B
  - d_a, d_b = distances travelled by vehicles on approaches A and B
    - *Distance travelled during perception-reaction time and braking (Table 7.7)*
- By knowing any three factors, the fourth one can be estimated



Sight Distance at Intersections

Table 7.7 (Garber & Hoel 2013)

Table 7.7 Suggested Lengths and Adjustments of Sight-Triangle Leg Case A—No Traffic Control

Design Speed (mi/h)	*Length of Leg (ft)*
15	70
20	90
25	115
30	140
35	165
40	195
45	220
50	245
55	285
60	325
65	365
70	405
75	445
80	485

(a)

Approach Grade (%)	*Design Speed (mi/h)*													
	15	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*	*65*	*70*	*75*	*80*
−6	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2
−5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2
−4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
−3 to 3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

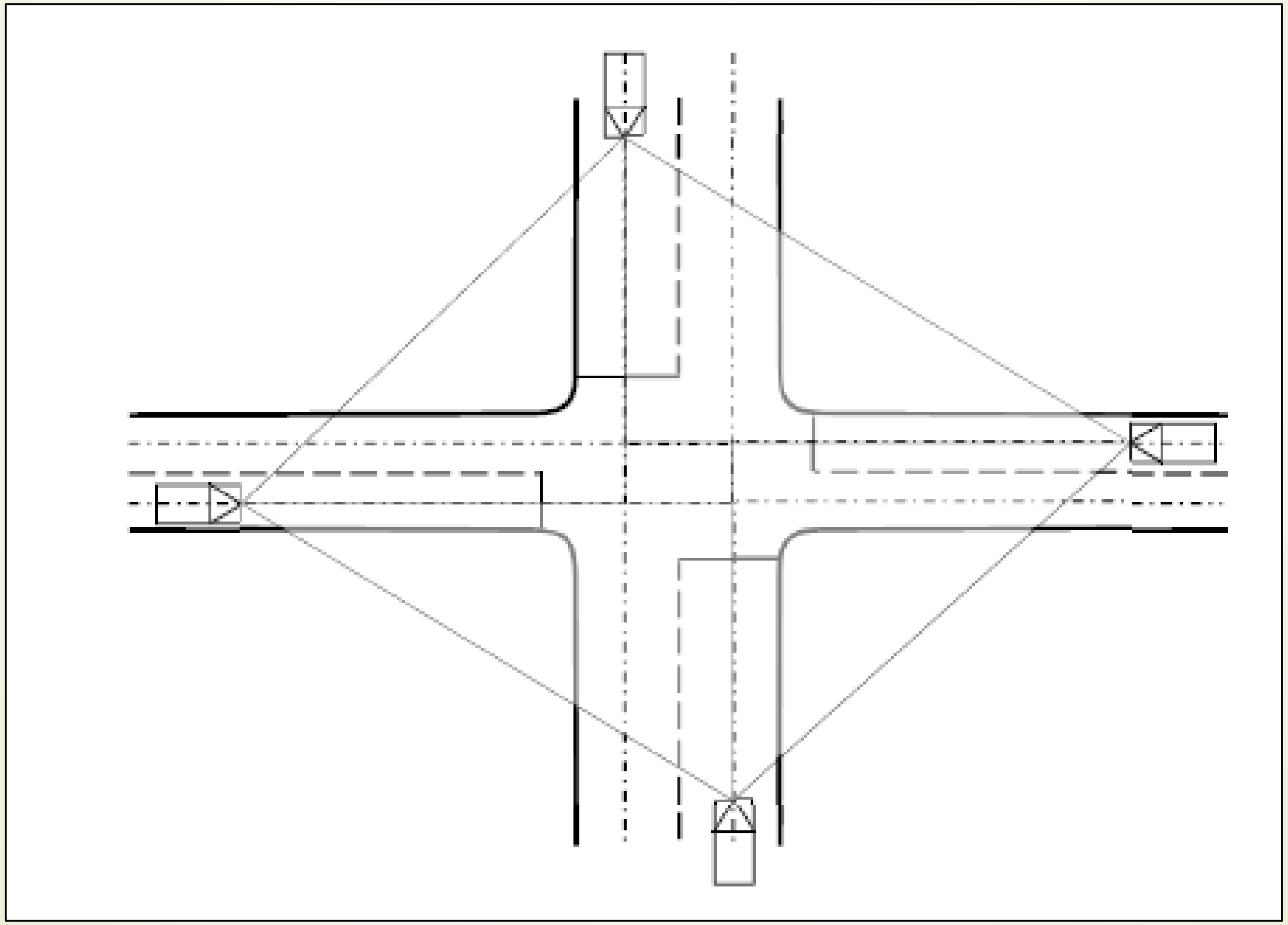
(b)

Note: For approach grades greater than 3 percent, sight distances given in Table 7.7a should be multiplied by the appropriate values given in Table 7.7b.

SOURCE: From *A Policy on Geometric Design of Highways and Streets*, 2011, AASHTO, Washington, D.C. Used by permission.

Sight Distance at Intersections

❖ A four-leg intersection would have four sight distance triangles to be checked



Sight Distance at Intersections

Case B: Stop-Control on Minor Road

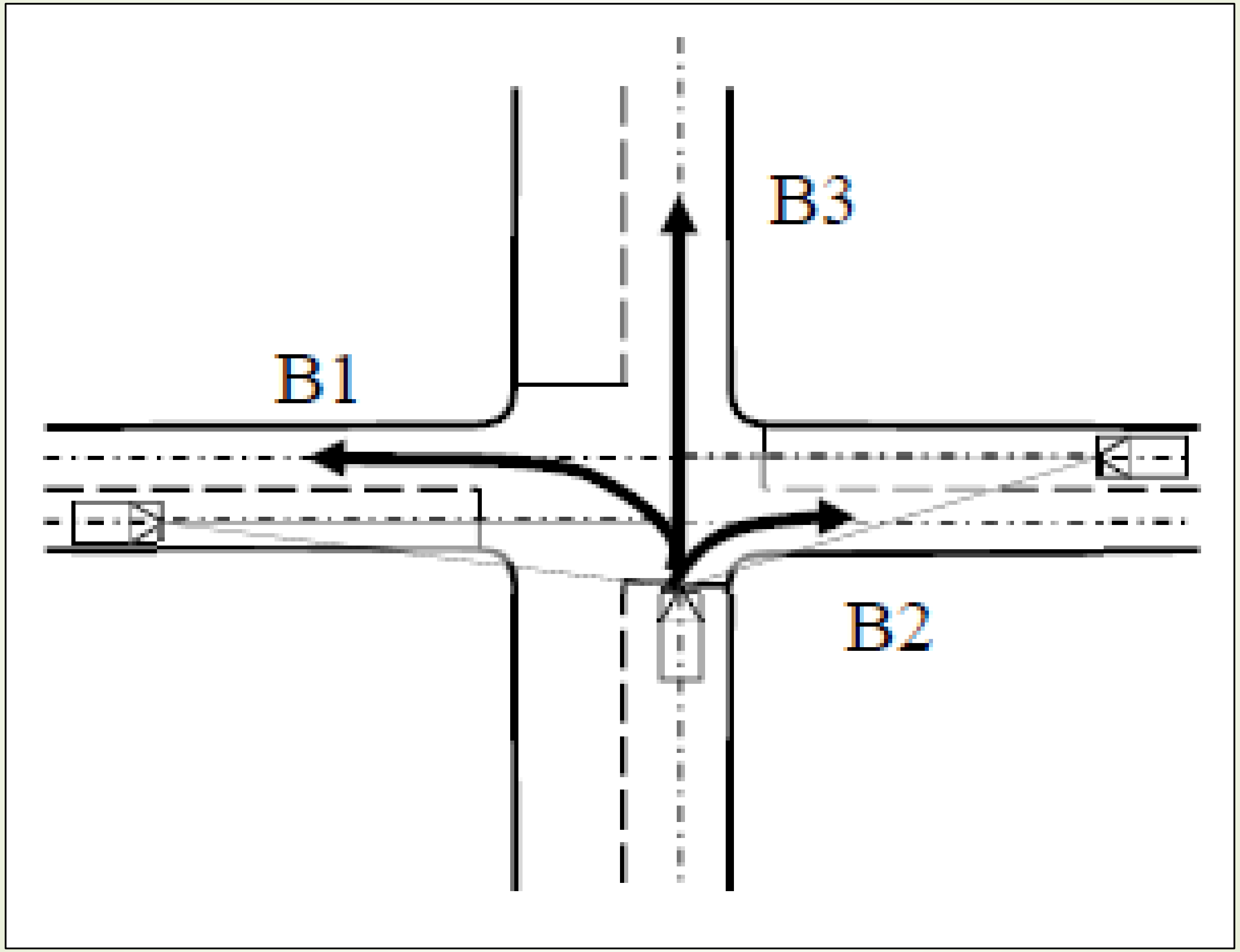
- ❖ SD in this case has to be enough for a safe departure from the stopped position
- ❖ There are three possible manoeuvres:

  – Case B1: Turning left onto the crossroad

  – Case B2: Turning right onto the crossroad

  – Case B3: Crossing the intersection

Sight Distance at Intersections



Sight Distance at Intersections

For the three cases:

$$d_{ISD} = 0.278\, V_{major}\, t_g \text{ (metric)}$$

$$d_{ISD} = 1.47\, V_{major}\, t_g \text{ (US customary)}$$

- d_{ISD} = sight distance along the major road
- V_{major} = design speed of the major road
- t_g = time gap for the minor road vehicle
  - Case B1: get t_g from Table 7.8
  - Case B2: reduce t_g by 1.0 s as drivers accept shorter gaps
  - Case B3: no need to check except in special cases

Sight Distance at Intersections

Table 7.8 (Garber & Hoel 2013)

Table 7.8 Time Gap for Case B1—Left Turn from Stop

Design Vehicle	*Time Gap (t_g) (sec) at Design Speed of Major Road*
Passenger Car	7.5
Single-Unit Truck	9.5
Combination Truck	11.5

Note: Time gaps are for a stopped vehicle to turn left onto a two-lane highway with no median and grade 3 percent or less. The table values require adjustment as follows:

For multilane highways:

For left turns onto two-way highways with more than two lanes, add 0.5 sec for passenger cars or 0.7 sec for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.

For minor road approach grades:

If the approach grade is an upgrade that exceeds 3 percent, add 0.2 sec for each percent grade for left turns.

SOURCE: *A Policy on Geometric Design of Highways and Streets*, American Association of State Highway and Transportation Officials, Washington, D.C., 2011, p. 9-37. Used with permission.

Sight Distance at Intersections

Case C: Yield-Control on Minor Road

❖ According to AASHTO, there are two possible cases:

– Case C1: Crossing a yield-controlled intersection from a minor road

– Case C2: Turning right or left from a minor road at a yield controlled intersection

Case 1:

- Similar to Case A but vehicles on the minor road are assumed to decelerate to 60% of the design speed
- Distance on major road is calculated as follows:

$$t_g = t_a + \frac{w + L_a}{0.167V_{minor}} \text{ (metric)}$$

$$d_{ISD} = 0.278V_{major}t_g \text{ (metric)}$$

  - » t_a = *travel time to reach the major road from the decision point (Table 7.9)*
  - » w = *width of intersection*
  - » L_a = *length of design vehicle*
  - » t_g = *travel time to reach to reach and clear the major road*
  - » V_{minor} & V_{major} = *design speed of the minor and major roads*
  - » d_{ISD} = *sight distance along the major road (also given in Table 7.10)*
- In US customary units, the equations become:

$$t_g = t_a + \frac{w + L_a}{0.88V_{minor}} \text{ (US customary)}$$

$$d_{ISD} = 1.47V_{major}t_g \text{(US customary)}$$

Sight Distance at Intersections

Table 7.9 (Garber & Hoel 2013)

Table 7.9 Case C1—Crossing Maneuvers from Yield-Controlled Approaches—Length of Minor Road Leg and Travel Times

	Minor-Road Approach		*Travel Time t_g (seconds)*	
Design Speed (mi/h)	*Length of Leg*[1] *(ft)*	*Travel Time $t_a^{1,2}$ (seconds)*	*Calculated Value*	*Design Value*[3,4]
15	75	3.4	6.7	6.7
20	100	3.7	6.1	6.5
25	130	4.0	6.0	6.5
30	160	4.3	5.9	6.5
35	195	4.6	6.0	6.5
40	235	4.9	6.1	6.5
45	275	5.2	6.3	6.5
50	320	5.5	6.5	6.5
55	370	5.8	6.7	6.7
60	420	6.1	6.9	6.9
65	470	6.4	7.2	7.2
70	530	6.7	7.4	7.4
75	590	7.0	7.7	7.7
80	660	7.3	7.9	7.9

[1]For minor-road approach grades that exceed 3 percent, multiply the distance or the time in this table by the appropriate adjustment factor from Table 7.7.
[2]Travel time applies to a vehicle that slows before crossing the intersection but does not stop.
[3]The value of t_g should equal or exceed the appropriate time gap for crossing the major road from a stop-controlled approach.
[4]Values shown are for a passenger car crossing a two-lane highway with no median and grades 3 percent or less.
SOURCE: From *A Policy on Geometric Design of Highways and Streets*, 2011, AASHTO, Washington, D.C. Used by permission.

Sight Distance at Intersections

Table 7.10 (Garber & Hoel 2013)

Table 7.10 Length of Sight Triangle Leg along Major Road—Case C1—Crossing Maneuver at Yield-Controlled Intersection

Major Road Design Speed (mi/h)	*Stopping Sight Distance (ft)*	*Minor Road Design Speed (mi/h)*							
		15	*20–50*	*55*	*60*	*65*	*70*	*75*	*80*
15	80	150	145	150	155	160	165	170	175
20	115	200	195	200	205	215	220	230	235
25	155	250	240	250	255	265	275	285	295
30	200	300	290	300	305	320	330	340	350
35	250	345	335	345	360	375	385	400	410
40	305	395	385	395	410	425	440	455	465
45	360	445	430	445	460	480	490	510	525
50	425	495	480	495	510	530	545	570	585
55	495	545	530	545	560	585	600	625	640
60	570	595	575	595	610	640	655	680	700
65	645	645	625	645	660	690	710	740	755
70	730	690	670	690	715	745	765	795	815
75	820	740	720	740	765	795	795	850	875
80	910	790	765	790	815	850	850	910	930

Note: Values in the table are for passenger cars and are based on the unadjusted distance and times in Table 7.9. The distance and times in Table 7.9 need to be adjusted using the factors in Table 7.7b.
SOURCE: From *A Policy on Geometric Design of Highways and Streets*, 2011, AASHTO, Washington, D.C. Used by permission.

Sight Distance at Intersections

Case C: Yield-Control on Minor Road

Case C2:

- Drivers are assumed to decelerate to a turning speed of 16 km/h (10 mi/h)
- Leg of approach sight distance triangle on the minor road is 25 m (82 ft)
- Leg of approach sight distance triangle on the major road is similar to the departure triangle in Cases B1 and B2 but t_g should be increased by 0.5 s

Signal Control

Types of traffic signal controllers:

– Fixed time (or pre-timed) : Use cycles and phases of pre-determined length; these cannot respond to short-term demand fluctuations. Different plans for different times of day (TOD plan)

– Traffic actuated : These can respond to short-term demand fluctuations; can change cycle and phase lengths in response to changes in traffic demand as detected by various types of actuation devices, such as tuned-circuit loops

– Adaptive : Most advanced system for traffic control; senses traffic changes on all intersections, processes the information and adjusts signal cycle, phases, offsets, etc. for all intersections linked in the network

Signal Control

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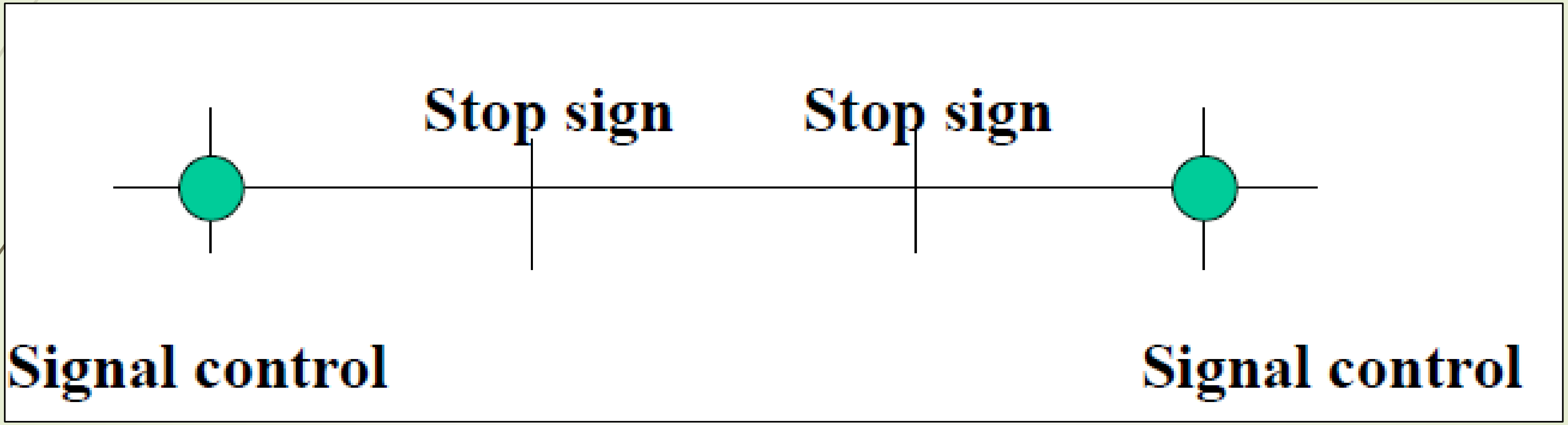
– **Adaptive :** Most advanced system for traffic control; senses traffic changes on all intersections, processes the information and adjusts signal cycle, phases, offsets, etc. for all intersections linked in the network

Signal Control

Types of traffic signals:

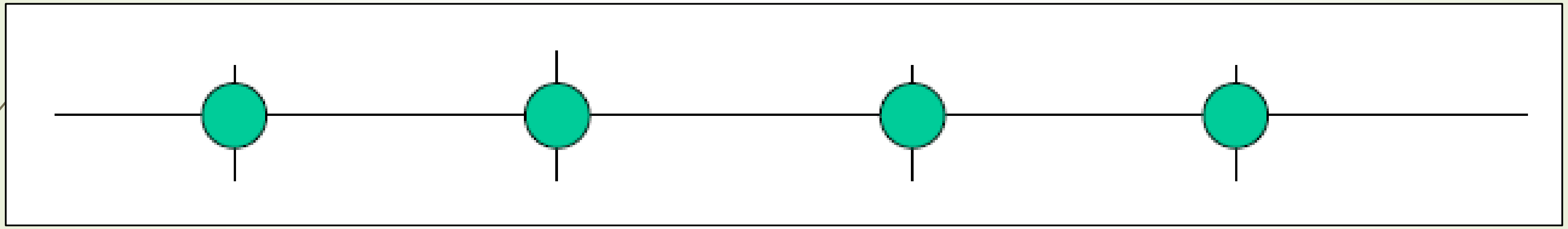
– Isolated intersections - If an intersection is far from other signalized intersections, its control is independent of other far away intersections



Signal Control

Types of traffic signals:

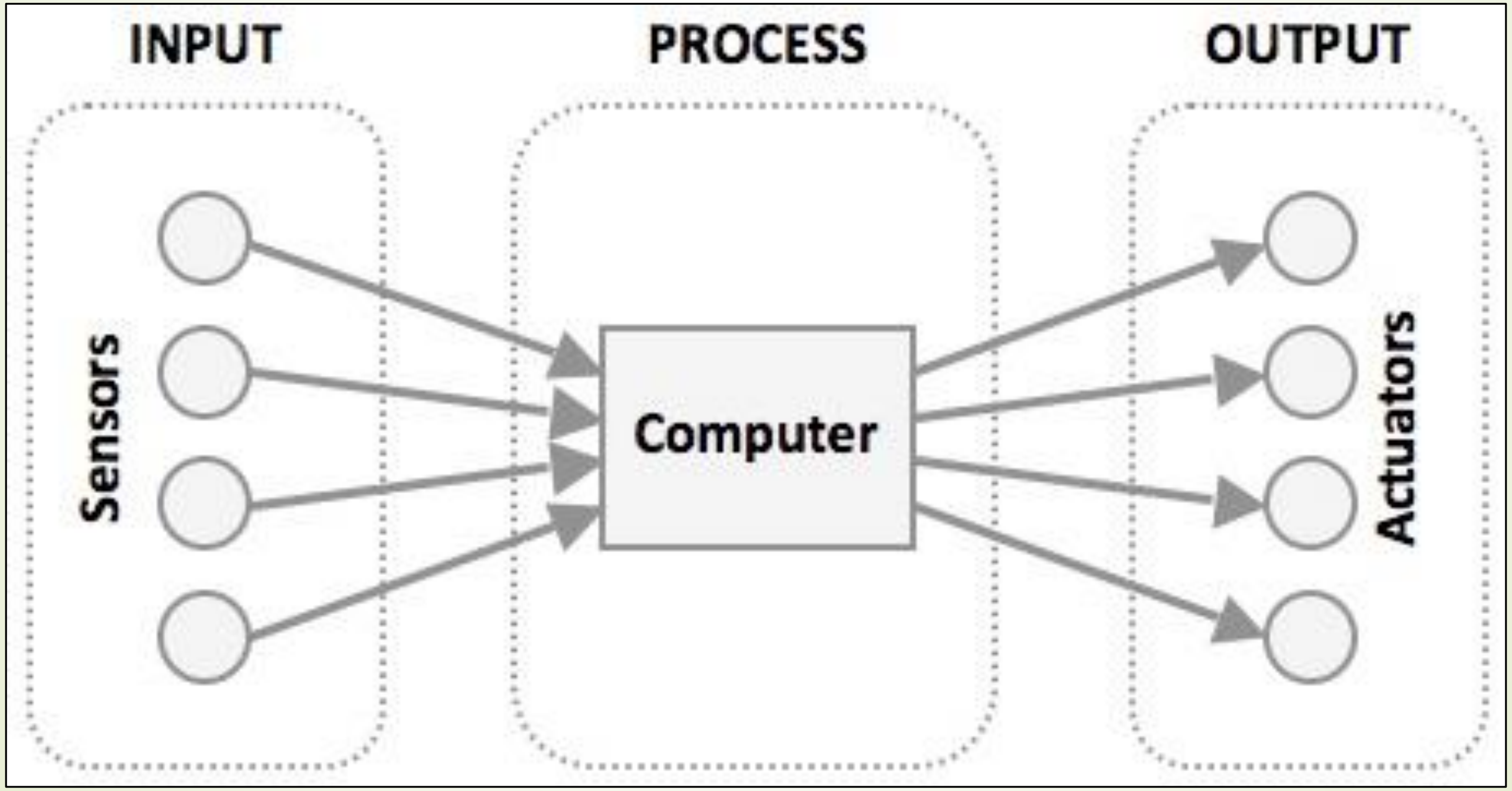
– Coordinated signals along an arterial – Closely spaced signals enable vehicles in one direction to get continuous green



Signal Control

Types of traffic signals:

– Central computer control – Closely spaced signals enable vehicles in one direction to get continuous green



Signal Timing at Isolated Intersections

- An isolated intersection is one in which the signal time is not coordinated with that of any other intersection and therefore operates independently
- Signal indication: Green, Yellow (amber), Red

<table>
<tr><th colspan="7">Traffic Signals</th></tr>
<tr><th>Intersection</th><th colspan="6">Color of Signal</th></tr>
<tr><td>Gladstone</td><td>30 (Green)</td><td>3 (Yellow)</td><td rowspan="2">3 (All Red)</td><td colspan="2">33 (Red)</td><td rowspan="2">3 (All Red)</td></tr>
<tr><td>Preston</td><td colspan="2">36 (Red)</td><td>30 (Green)</td><td>3 (Yellow)</td></tr>
</table>

<table>
<tr><th colspan="7">Pedestrian Signals</th></tr>
<tr><th>Intersection</th><th colspan="6">Color of Signal</th></tr>
<tr><td>Preston</td><td>23 (White Walking Person- Walk)</td><td>7 (Flashing Red Hand - Do Not Enter)</td><td colspan="4">42 (Red , Upper Raised Hand - Do Not (Walk)</td></tr>
<tr><td>Gladstone</td><td colspan="3">36 (Red , Upper Raised Hand - Do Not Walk)</td><td>23 (White Walking Person- Walk)</td><td>7 (Flashing Red Hand - Do Not Enter)</td><td>6 (Red)</td></tr>
</table>

Signal Timing at Isolated Intersections

Definitions:

– **Cycle (cycle length):** the time in seconds required for one complete colour sequence of signal indication

– **Phase (signal phase):** that part of a cycle allocated to a stream of traffic, or a combination of two or more streams of traffic, having the right of way simultaneously during one or more intervals

– **Interval:** any part of the cycle length during which signal indications do not change

– **Change and clearance interval:** total length of time in seconds of the yellow and all-red signal indications (allows vehicles to clear the intersection before conflicting movements are released)

Signal Timing at Isolated Intersections

Definitions:

– **All-red interval:** the display time of a red indication for all approaches

– **Peak-hour factor (PHF):** a measure of variability of demand during the peak hour, and is equal to the ratio of the volume during the peak hour to the maximum rate of flow during a given period within the peak hour

Peak Hour Factor (PHF) for 15 Minute Interval

$$\text{PHF} = \frac{\text{Hourly Volume}}{4^*\text{Maximum Rate of Flow for 15 Minute Interval}}$$

Design Hourly Volume (DHV) can be calculated from PHF using the following expression

$$\text{DHV} = \frac{\text{Peak Hour Volume}}{\text{PHF}}$$

Signal Timing at Isolated Intersections

Example: PHF

Given the following PM peak hour data from 4:15 to 5:15 p.m for a particular intersection, determine the Peak Hour Factor (PHF) for 15 Minute Interval and DHV.

Time	Traffic Volume
4:15 – 4:30 p.m.	481
4:30 – 4:45 p.m.	470
4:45 – 5:00 p.m.	500
5:00 – 5:15 p.m.	492

Peak Hour Factor (PHF) for 15 Minute Interval

$$= \frac{\text{Hourly Volume}}{4 * \text{Maximum Rate of Flow for 15 Minute Interval}}$$

$$= \frac{481 + 470 + 500 + 472}{4 * 500} = 0.972$$

$$\text{DHV} = \frac{\text{Peak Hour Volume}}{\text{PHF}} = \frac{1943}{0.972} = 1999$$

Signal Timing at Isolated Intersections

Definitions (Cont'd):

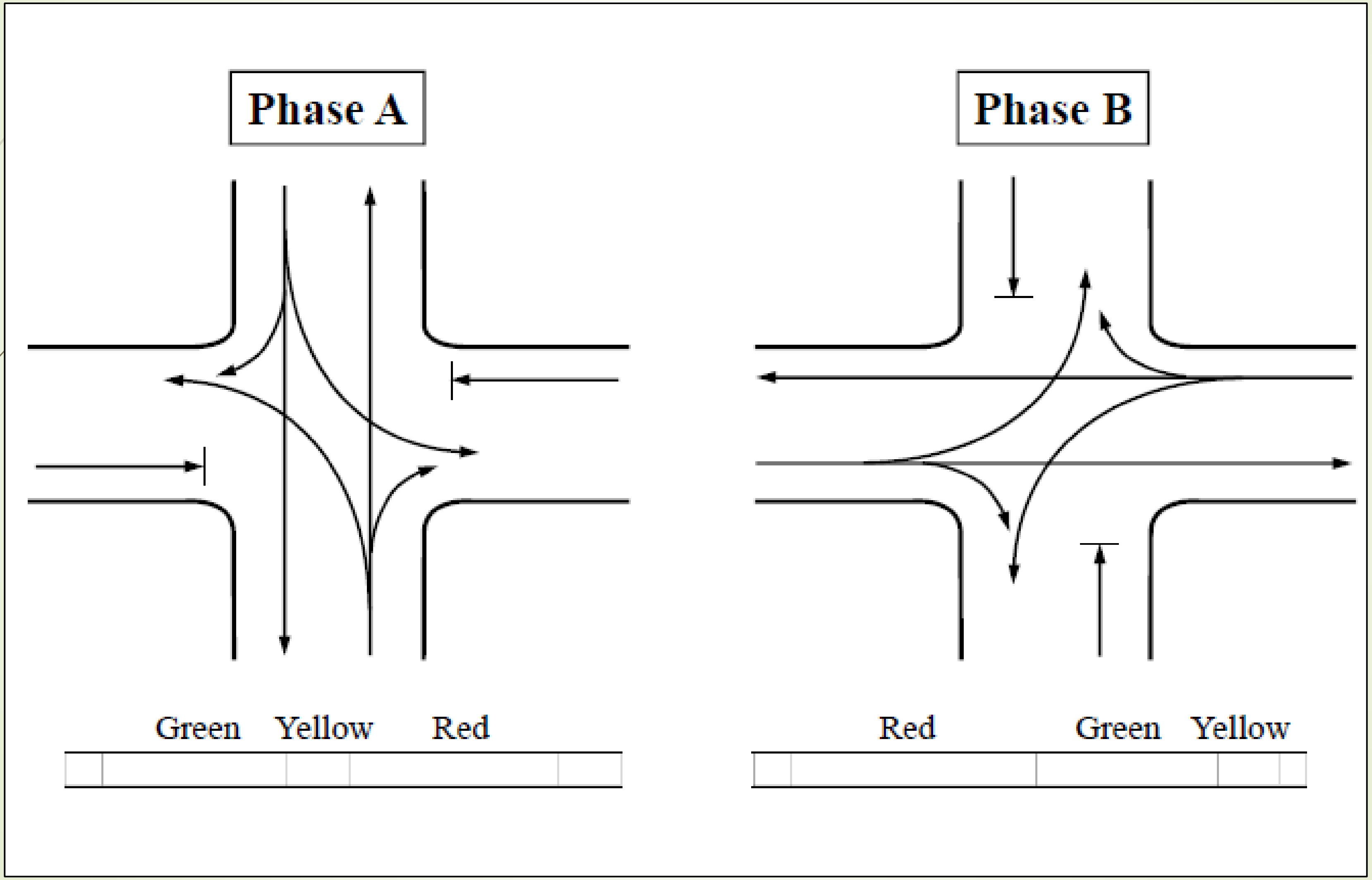
– **Passenger car equivalent (PCE):** a factor to convert straight-through volumes of buses and trucks to straight-through volumes of passenger cars (1.6–2.5 for intersections)

– **Turning movement factors:** factors to convert turning vehicles to equivalent straight through vehicles (1.4–1.6 for left-turning vehicles and 1.0–1.4 for right-turning)

– **Critical lane volume:** maximum lane volume in a phase (veh/h)

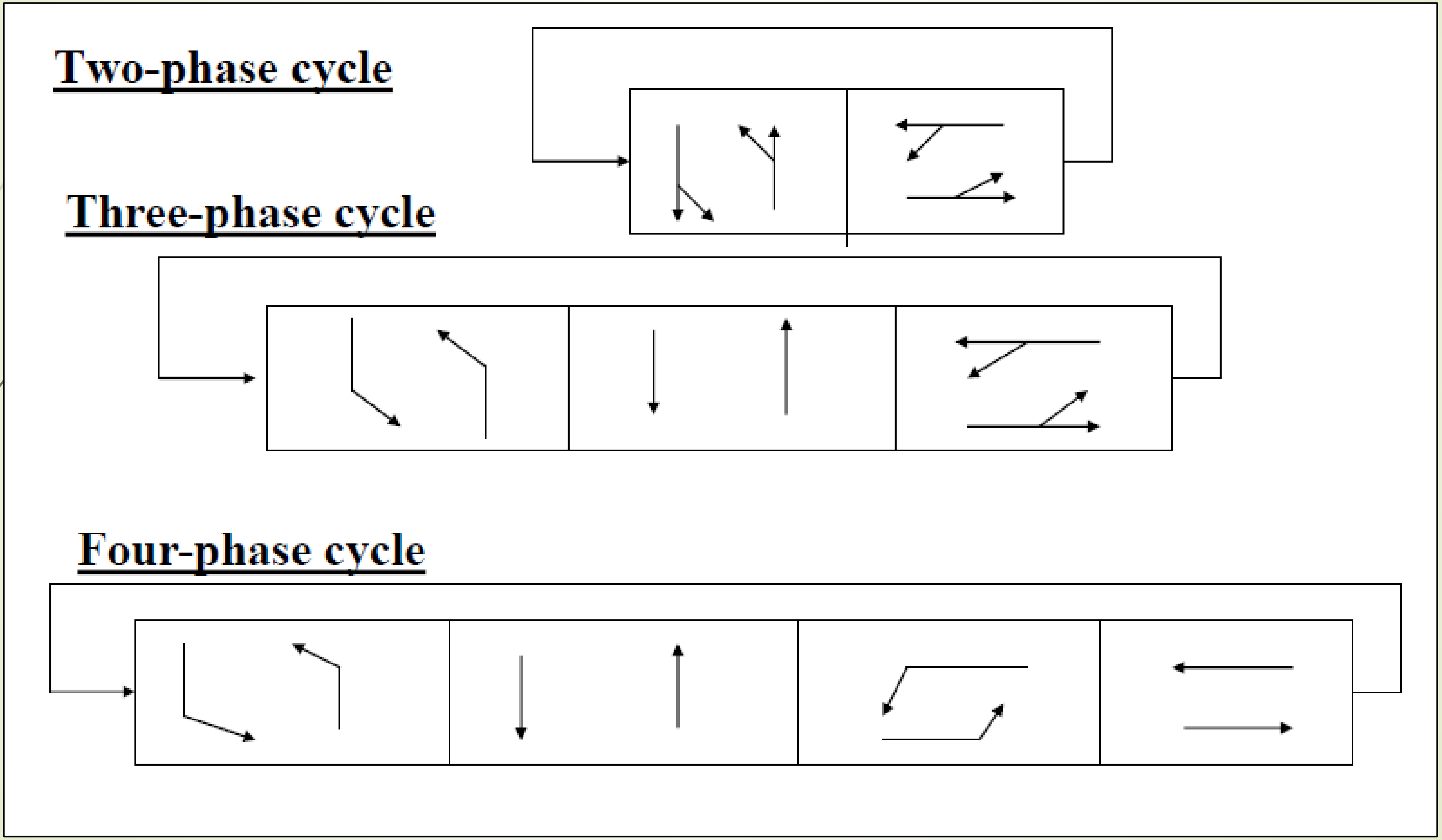
Signal Timing at Isolated Intersections

Phase Diagrams for traffic signals



Signal Timing at Isolated Intersections

Phase Diagrams for traffic signals



Signal Timing at Isolated Intersections

- The main objectives of signal timing are:
  - To reduce the average delay of all vehicles
  - To reduce the probability of accidents
- The two objectives may conflict with each other:
  - Increasing the number of phases will reduce the probability of accidents (by reducing conflict points of traffic) and increase average delay

Signal Timing at Isolated Intersections

Yellow Interval:

❖ The objectives of the yellow indication after the green are:

– To alert motorists to the fact that the green time is about to change to red
– To allow vehicles already in the intersection to cross it

❖ A bad choice of yellow interval may lead to the creation of a dilemma zone:

– An area in which vehicles can neither stop safely before the intersection nor clear it without speeding before the red signal comes on

❖ Therefore, the yellow interval must guarantee that an approaching vehicle can either:

– Stop safely, or
– Proceed through the intersection without speeding

Signal Timing at Isolated Intersections

At the minimum yellow interval required to eliminate the dilemma zone (τ_{min}): $X_0 = X_c$

- For vehicles to just clear the intersection:

$$X_c = u_0 \tau_{min} - (W + L)$$

- u_0 = speed limit on the approach (m/s)
- W = width of intersection (m)
- L = length of vehicle (m)

- For vehicles to stop before the intersection:

$$X_0 = u_0 \delta + \frac{u_0^2}{2a}$$

- δ = perception-reaction time (s)
- a = rate of braking deceleration (m/s^2)

- Therefore,

$$u_0 \tau_{min} - (W + L) = u_0 \delta + \frac{u_0^2}{2a}$$

- and

$$\tau_{min} = \delta + \frac{(W + L)}{u_0} + \frac{u_0}{2a}$$

- If the effect of grade is added:

$$\tau_{min} = \delta + \frac{(W + L)}{u_0} + \frac{u_0}{2(a + Gg)}$$

- G = grade of the approach
- g = acceleration due to gravity (m/s^2)

Note:

– For safety considerations, the yellow interval should not be less than 3 s

– To encourage motorists' respect for the yellow interval, it should not be greater than 5 s

– If a longer yellow interval is required, use the maximum yellow interval and add an all-red interval

Signal Timing at Isolated Intersections

Example: Yellow Interval

Determine the minimum yellow interval at a flat intersection whose width is 12 m if the maximum allowable speed on the approach roads is 50 km/h. Assume average length of vehicle is 6.0 m, comfortable deceleration rate is 0.27g, and perception-reaction time is 1.0 sec.

Solution:

$$\tau_{min} = \delta + \frac{(W + L)}{u_0} + \frac{u_0}{2a} = 1.0 + \frac{(12 + 6)}{50 \times 0.278} + \frac{50 \times 0.278}{2 \times 0.27 \times 9.81}$$

$$= 4.92 \text{ s}$$

Therefore, use a 5-sec yellow interval

Signal Timing at Isolated Intersections

Cycle Length:

- This section discusses pre-timed (fixed) signals only

  – Each signal has a preset cycle length that remains fixed for a specific period of the day or for the whole day

- Methods that exist for determining the cycle length include:

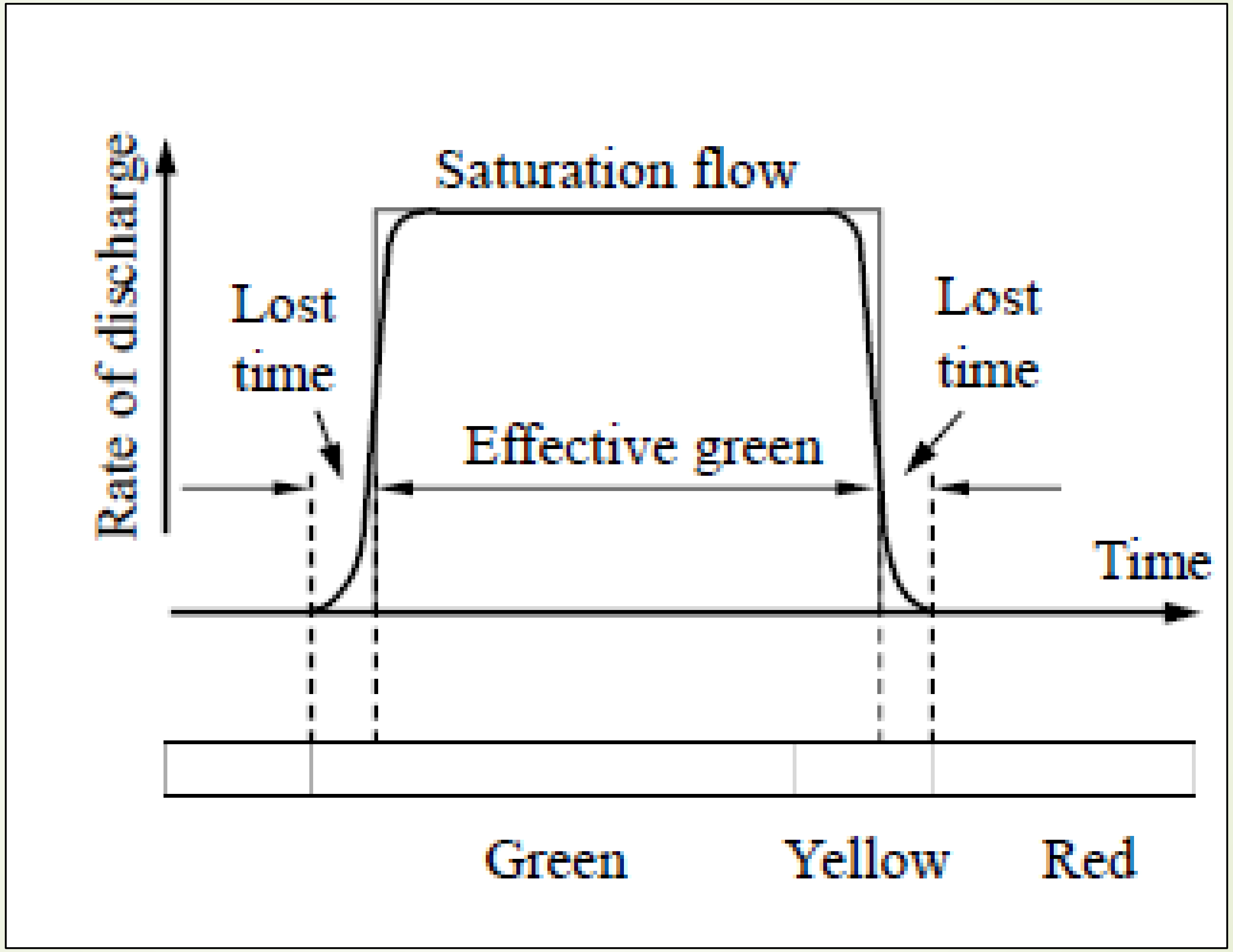
  –Webster method and

  – TRB's Highway Capacity Manual Method

Signal Timing at Isolated Intersections

- Rate of discharge of vehicles at an intersection:
  - At the beginning of the green interval, some time is lost before the vehicles start moving
  - The rate of discharge then increases to a maximum (saturation flow, S)
  - If there are sufficient vehicles in the queue to use the available green time, the saturation flow will be sustained until the yellow interval occurs
  - The rate of discharge then falls to zero when the yellow signal changes to red
  - The number of vehicles discharged through the intersection is represented by the area under the curve
  - Dividing the number of vehicles by the saturation flow will give the effective green time
  - The effective green is less than the sum of the green and yellow; the difference is considered lost time

Signal Timing at Isolated Intersections



Signal Timing at Isolated Intersections

Webster Method:

- For a wide range of practical conditions, minimum intersection delay is obtained when the cycle length is obtained by:

$$C_0 = \frac{1.5L + 5}{1 - \sum_{i=1}^{\phi} Y_i}$$

  - C_0 = optimum cycle length (s)
  - L = total lost time per cycle (s)
  - $Y_i = q_{ij}/S_j$ = maximum value of the ratios of approach flows to saturation flows for all traffic streams using phase i
  - ϕ = number of phases
  - q_{ij} = flow on lane j having the right of way during phase i
  - S_j = saturation flow on lane j
- Lost time for each phase can be estimated as:

$$\ell_i = G_{ai} + \tau_i - G_{ei}$$

  - ℓ_i = lost time for phase i
  - G_{ai} = actual green time for phase i
  - τ_i = yellow time for phase i
  - G_{ei} = effective green time for phase i

Signal Timing at Isolated Intersections

Webster Method:

- Total lost time is given as:

$$L = \sum_{i=1}^{\emptyset} \ell_i + R$$

  - R = total all-red time during the cycle

- Total effective green time per cycle is:

$$G_{te} = C - L = C - \left(\sum_{i=1}^{\emptyset} \ell_i + R \right)$$

  - C = actual cycle length (the value of C_0 rounded to the nearest 5 s)

- The total effective green time is distributed among the different phases in proportion to their Y values:

$$G_{ei} = \frac{Y_i}{\sum_{i=1}^{\emptyset} Y_i} G_{te}$$

- The actual green time is obtained as:

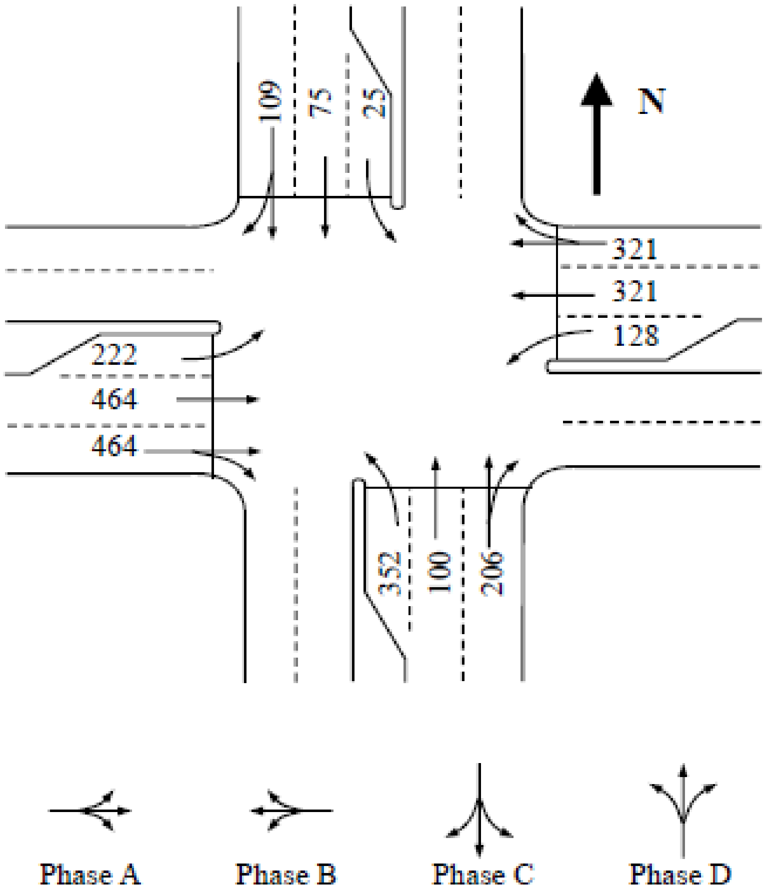
$$G_{ai} = G_{ei} + \ell_i - \tau_i$$

Signal Timing at Isolated Intersections

Example - Signal Timings

The following figure shows peak-hour volumes for a major intersection on an expressway. Using the Webster method, determine suitable signal timing for the intersection using a four-phase system and the additional data given in the figure. Use a yellow interval of 3 s and assume the total lost time is 3.5 s per phase. Additional information:

- PHF = 0.95
- Left-turn factor = 1.4
- PCE for buses and trucks = 1.6
- Truck percentages:
  - **4% for the west approach**
  - **0% for the other approaches**
- Saturation flow rate = 2000 pc/h for all lanes
- Assume the given phasing system

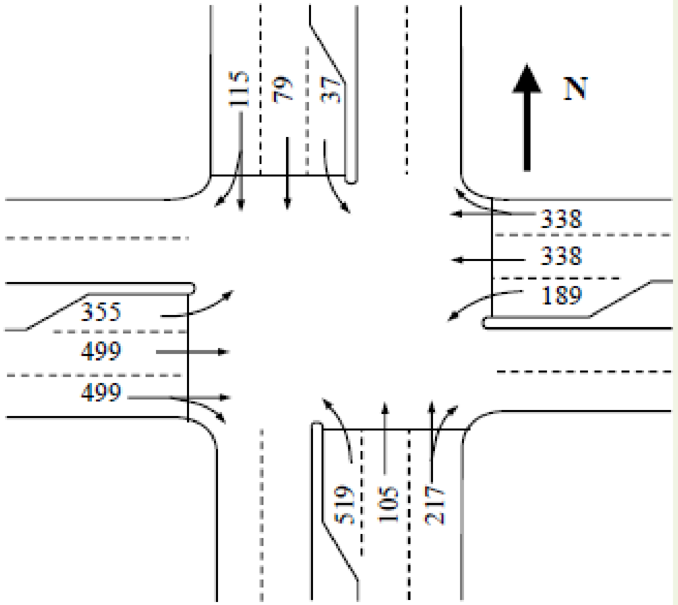


Signal Timing at Isolated Intersections

Solution:

Step 1: Adjust Volumes in each direction to PCs

- First, convert mixed volumes to equivalent straight-through passenger cars
- Example: EB (West Approach) through traffic
  - DHV = 464 veh/h
  - q = 464/0.95 = 488 veh/h
  - Among them:
    - **0.04 x 488 = 19.5 trucks & buses**
    - **488 – 19.5 = 468.5 PC**
  - PCE = 468.5 + 19.5 × 1.6 ≈ 499 PC/h
- Calculations are based on an assumption of RT equivalency factor = 1.0
- You may then calculate the critical lane volume for each phase



Phase, ϕ	Critical Lane Volume
A	500
B	338
C	115
D	519
Total	1472

Signal Timing at Isolated Intersections

Step 1: Adjust Volumes in each direction to PCs

	North Bound Traffic From South			South Bound Traffic From North			East Bound Traffic From West			West Bound Traffic From East		
	NBT	NBLT	NBRT	SBT	SBLT	SBRT	EBT	EBLT	EBRT	WBT	WBLT	WBRT
Traffic Volume	100	352	206	75	25	109	464	222	464	321	128	321
Equivalent Hourly Flow (=PHV/PHF)	105	371	217	79	26	115	488	234	488	338	135	338
% Trucks	0	0	0	0	0	0	4	4	4	0	0	0
Total Trucks and Buses	0	0	0	0	0	0	20	9	20	0	0	0
Convert Buses and Trucks to PCE =1.6*(Trucks + Buses)	0	0	0	0	0	0	31	15	31	0	0	0
Total PC in each direction before Adj of LT traffic	105	371	217	79	26	115	500	239	500	338	135	338
Total PC in each direction after Adj of LT traffic	105	519	217	79	37	115	500	335	500	338	189	338
Critical Lane in each phase		519			115			500			338	
Total Critical Volume							**1472**					

Signal Timing at Isolated Intersections

Step 2: Calculate Yi (v/c) for each Direction and Obtain the maximum value (Yi = qij/Sj)

Step 2: Calculate Yi for each Direction and Obtain the maximum value (Yi = qij/Sj)												
	NB			SB			EB			WB		
	NBT	NBLT	NBRT	SBT	SBLT	SBRT	EBT	EBLT	EBRT	WBT	WBLT	WBRT
qij = flow on lane j having the right of way during phase I (veh/ln)	105	519	217	79	37	115	500	335	500	338	189	338
Sj = saturation flow on lane j (pc/h/ln)	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000
Volume to Capacity ratios (Yi = qij/Sj)	0.05	0.26	0.11	0.04	0.02	0.06	0.25	0.17	0.25	0.17	0.09	0.17
Maximum Yi (v/c) ratio (=qij/Sj) in each phase	0.26			0.06			0.25			0.17		
ΣYi	0.74											

Signal Timing at Isolated Intersections

Step 3: Obtain Optimum Cycle Length from Webster Equation

Optimum cycle length:

$$C_0 = \frac{1.5L + 5}{1 - \sum_{i=1}^{\emptyset} Y_i}$$

1. ***L*** **= Total Lost Time per cycle= Total lost time per phase*Number of phases**
   = 3.5*4 = 14 Seconds

2. $$\sum Y_i = 0.74$$

3. Optimum cycle length:

$$C_0 = \frac{1.5L + 5}{1 - \sum_{i=1}^{\emptyset} Y_i} = \frac{1.5 \times 14 + 5}{1 - 0.74} =$$ 100 s (to the nearest 5 s)

Signal Timing at Isolated Intersections

Step 4 : Calculate the Total Effective Green (Gte= C-L)

Gte = C-L
= 100-14
= 86 s

Step 5 : Calculate the Effective Green for each phase (Gei = [(Yi/ΣYi)*Gte]

Phase	Yi	ΣYi	Gei
Phase A	0.25	0.74	29
Phase B	0.17	0.74	20
Phase C	0.06	0.74	7
Phase D	0.26	0.74	30

For Phase A:

Gei = (Yi/ΣYi)*Gte

$$\text{Gei} = \frac{0.25}{0.74} * 86 = 29 \text{ s}$$

Signal Timing at Isolated Intersections

Step 6 : Calculate the Actual Green for each phase (Gai = Gei + li - τi)

Phase	Gei	li	τi	Gai
Phase A	29	3.5	3	30
Phase B	20	3.5	3	20
Phase C	7	3.5	3	6
Phase D	30	3.5	3	30

For Phase A:

Gai = Gei + li – τi

Gai = 29 + 3.5 – 3 = 30 s

- Check:
  - Sum of all actual green, yellow, and all-red is equal to cycle length

$\sum_{i=1}^{\emptyset}$ Gai = Cycle Length = 30+20+6+30 = 86 s

Thank You!!!