CHAPTER 20
ALL-WAY STOP-CONTROLLED INTERSECTIONS

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1. INTRODUCTION

Chapter 20, All-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing these types of intersections (*1*). A glossary and list of symbols, including those used for all-way STOP-controlled (AWSC) intersections, is provided in Chapter 9.

AWSC intersections require every vehicle to stop at the intersection before proceeding. Because each driver must stop, the decision to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after stopping. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that no vehicles are currently in the intersection and that it is the driver's turn to proceed.

Field observations indicate that standard four-leg AWSC intersections operate in either a two-phase or a four-phase pattern, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north–south and east–west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach intersection).

If traffic is present on the subject approach only, vehicles depart as rapidly as individual drivers can safely accelerate into and clear the intersection. This case is illustrated as Case 1 in Exhibit 20-1.

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16. Urban Street Facilities
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18. Signalized Intersections
19. TWSC Intersections
20. AWSC Intersections
21. Roundabouts
22. Interchange Ramp Terminals
23. Off-Street Pedestrian and Bicycle Facilities

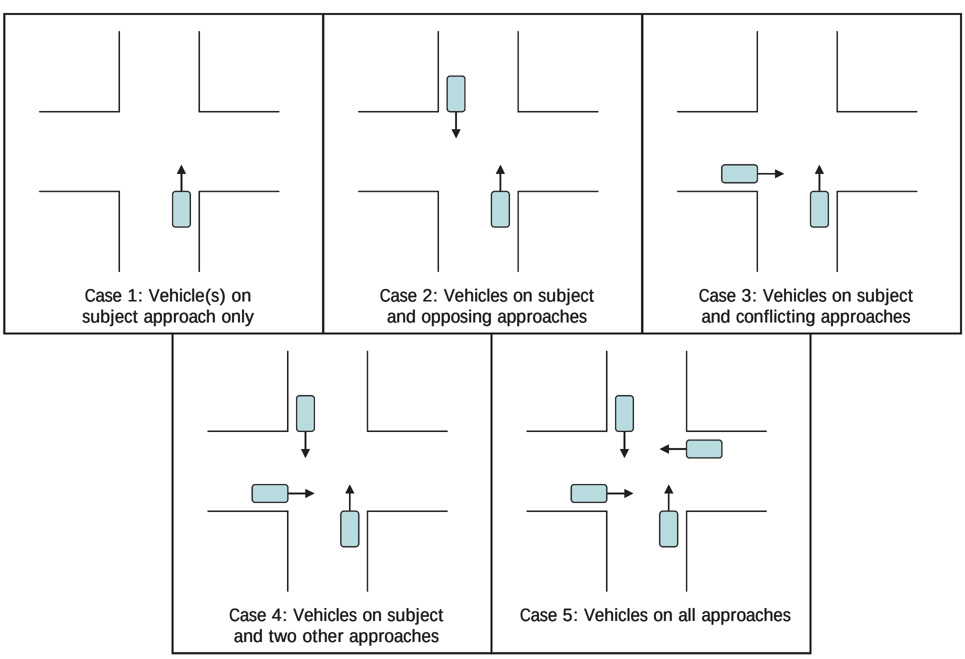


Exhibit 20-1
Analysis Cases for AWSC Intersections

If traffic is present on the other approaches, as well as on the subject approach, the saturation headway (the time between subsequent vehicle departures) on the subject approach will increase somewhat, depending on the degree of conflict that results between the subject approach vehicles and the vehicles on the other approaches. In Case 2, some uncertainty is introduced with a vehicle on the opposing approach, and thus the saturation headway will be greater than for Case 1. In Case 3, vehicles on one of the conflicting approaches further restrict the departure rate of vehicles on the subject approach, and the saturation headway will be longer than for Case 1 or Case 2. In Case 4, two vehicles are waiting on opposing or conflicting approaches, and saturation headways are even longer. When vehicles are present on all approaches, as in Case 5, saturation headways are the longest of any of the cases because the potential for conflict between vehicles is greatest. The increasing degree of potential conflict translates directly into longer driver decision times and longer saturation headways. Because no traffic signal controls the stream movement or allocates the right-of-way to each conflicting traffic stream, the rate of departure is controlled by the interactions between the traffic streams.

Capacity of an AWSC can be described by saturation headway, departure headway, and service time.

Therefore, the operation at an AWSC intersection can be described numerically by a few key time-based terms:

- The saturation headway, h_{si}, is the time between departures of successive vehicles on a given approach for a particular case (case i), as described above, assuming a continuous queue.
- The departure headway, h_d, is the average time between departures of successive vehicles on a given approach accounting for the probability of each possible case.
- The service time, t_s, is the average time spent by a vehicle in first position waiting to depart. It is equal to the departure headway minus the time it takes a vehicle to move from second position into first position (the move-up time, m).

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The intersection analysis boundaries for an AWSC analysis are assumed to be those of an isolated intersection; that is, no upstream or downstream effects are accounted for in the analysis. The present methodology is limited to motor vehicles.

LEVEL-OF-SERVICE CRITERIA

The level-of-service (LOS) criteria for AWSC intersections are given in Exhibit 20-2. As the exhibit notes, LOS F is assigned if the volume-to-capacity (v/c) ratio of a lane exceeds 1.0, regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

Exhibit 20-2
LOS Criteria: Automobile Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio*	
	$v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: * For approaches and intersectionwide assessment, LOS is defined solely by control delay.

REQUIRED INPUT DATA

Analysis of an AWSC intersection requires the following data:

1. Number and configuration of lanes on each approach;
2. Percentage of heavy vehicles;
3. Turning movement demand flow rate for each entering lane or, alternatively, hourly demand volume and peak hour factor; and
4. Length of analysis period—generally a peak 15-min period within the peak hour, although any 15-min period can be analyzed.

SCOPE OF THE METHODOLOGY

This chapter focuses on the operation of AWSC intersections. This version of the AWSC intersection analysis procedures is primarily a result of studies conducted by National Cooperative Highway Research Program Project 3-46 (*1*).

LIMITATIONS OF THE METHODOLOGY

Automobile Mode

The methodologies in this chapter apply to isolated AWSC intersections with up to three lanes on each approach. They do not account for interaction effects with other intersections. The methodologies do not apply to AWSC intersections with more than four approaches. In addition, the effect of conflicting pedestrians on automobiles is not considered in this procedure. Conflicting pedestrian movements are likely to increase the saturation headway of affected vehicular movements, but the magnitude of this effect is unknown as of the publication of this edition of the HCM.

Pedestrian and Bicycle Modes

The current methodologies for analyzing LOS and delay at AWSC intersections do not extend to pedestrians and apply to bicycles only in limited situations that are not supported by research at the time of publication of this edition. As such, there are no set LOS standards that apply to pedestrians or bicycles at AWSC intersections, nor can pedestrian or bicycle delay, capacity, or quality of service be quantitatively assessed by using the procedures described in this chapter. Additional research on pedestrian and bicyclist behavior and operations at AWSC intersections needs to be done before procedures can be developed that adequately address these issues. A discussion of qualitative effects is included in the methodology section of this chapter.

2. METHODOLOGY

OVERVIEW

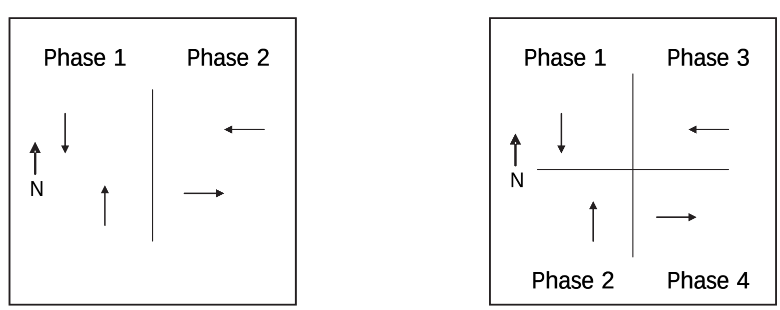
AWSC intersections require drivers on all approaches to stop before proceeding into the intersection. While giving priority to the driver on the right is a recognized rule in some areas, it is not a good descriptor of actual intersection operations. What happens is the development of a consensus of right-of-way that alternates between the drivers on the intersection approaches, a consensus that depends primarily on the intersection geometry and the arrival patterns at the stop line.

The methodology analyzes each intersection approach independently. The approach under study is called the subject approach. The opposing approach and the conflicting approaches create conflicts with vehicles on the subject approach.

Phase Patterns

A two-phase pattern, as shown in Exhibit 20-3(a), is observed at a standard four-leg AWSC intersection (one approach lane on each leg), where drivers from opposing approaches enter the intersection at roughly the same time. Some interruption of this pattern occurs when there are conflicts between certain turning maneuvers (such as a northbound left-turning vehicle and a southbound through vehicle), but generally the north–south streams alternate right-of-way with the east–west streams. A four-phase pattern, as shown in Exhibit 20-3(b), emerges at multilane four-leg intersections, where development of the right-of-way consensus is more difficult. Here drivers from each approach enter the intersection together as right-of-way passes from one approach to the next and each is served in turn. A similar three-phase pattern emerges at multilane three-leg intersections.

Exhibit 20-3
Operation Patterns at AWSC Intersections



(a) Two-phase (single-lane approaches) (b) Four-phase (multilane approaches)

Two cases for departure headways.

The headways of vehicles departing from the subject approach fall into one of two cases. If there are no vehicles on any of the other approaches, subject approach vehicles can enter the intersection immediately after stopping. However, if vehicles are waiting on a conflicting approach, a vehicle from the subject approach must wait for consensus with the next conflicting vehicle. The headways between consecutively departing subject approach vehicles will be shorter in the first case than in the second case. Thus, the headway for a departing subject approach vehicle depends on the degree of conflict experienced with vehicles on the other intersection approaches. The degree of conflict increases with two factors: the number of vehicles on the other approaches and the complexity of the intersection geometry.

Two other factors affect the departure headway of a subject approach vehicle: vehicle type and turning movement. The headway for a heavy vehicle will be longer than that for a passenger car. Furthermore, the headway for a left-turning vehicle will be longer than that for a through vehicle, which in turn will be longer than that for a right-turning vehicle.

Vehicle type and turning movement affect departure headway. These effects are captured empirically in the method.

In summary:

1. Standard four-leg AWSC intersections operate in either two-phase or four-phase patterns, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north–south and east–west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach).
2. The headways between consecutively departing subject approach vehicles depend on the degree of conflict between these vehicles and the vehicles on the other intersection approaches. The degree of conflict is a function of the number of vehicles faced by the subject approach vehicle and of the number of lanes on the intersection approaches.
3. The headway of a subject approach vehicle also depends on its vehicle type and its turning maneuver (if any).

Capacity Concepts

Capacity defined.

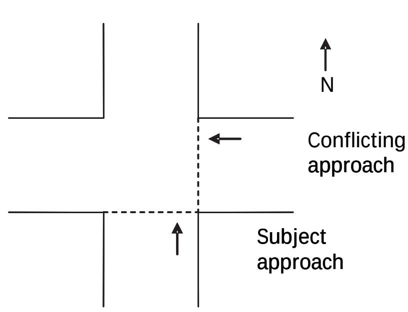
Capacity is defined as the maximum throughput on an approach given the flow rates on the other intersection approaches. The capacity model described here is an expansion of earlier work (2). The model is described for four increasingly complex cases: the intersection of two one-way streets with no turning movements, the intersection of two two-way streets with no turning movements, a generalized model for single-lane sites, and a generalized model for multilane sites. The methodology described later in this chapter is an implementation of the latter and most general case.

Intersection of Two One-Way Streets

The impact of turning movements is considered later, as part of the generalized models.

The first formulation of the model is based on the intersection of two one-way streets, each STOP-controlled. In this basic model, vehicles on either approach travel only straight through the intersection, as shown in Exhibit 20-4.

Exhibit 20-4
AWSC Configuration: Formulation 1



The saturation headway for a vehicle assumes one of two values: h_{s1} is the saturation headway if no vehicle is waiting on the conflicting approach, and h_{s2} is the saturation headway if the conflicting approach is occupied. The departure headway for vehicles on an approach is the expected value of this bivalued distribution. For the northbound approach, the mean service time is computed by Equation 20-1:

Equation 20-1

$$h_{d,N} = h_{s1}(1 - x_W) + h_{s2}x_W$$

where x_W is the degree of utilization of the westbound approach and is equal to the probability of finding at least one vehicle on that approach. Thus $1 - x_W$ is the probability of finding no vehicle on the westbound approach.

By symmetry, the mean service time for the westbound approach is given by Equation 20-2.

Equation 20-2

$$h_{d,W} = h_{s1}(1 - x_N) + h_{s2}x_N$$

Since the degree of utilization x is the product of the arrival rate λ and the mean departure headway h_d, the departure headways for each approach can be expressed in terms of the bivalued saturation headways and the arrival rates on each approach, as in Equation 20-3 and Equation 20-4.

Equation 20-3

$$h_{d,N} = \frac{h_{s1}\left[1 + \lambda_W\left(h_{s2} - h_{s1}\right)\right]}{1 - \lambda_N\lambda_W\left(h_{s2} - h_{s1}\right)^2}$$

Equation 20-4

$$h_{d,W} = \frac{h_{s1}\left[1 + \lambda_N\left(h_{s2} - h_{s1}\right)\right]}{1 - \lambda_N\lambda_W\left(h_{s2} - h_{s1}\right)^2}$$

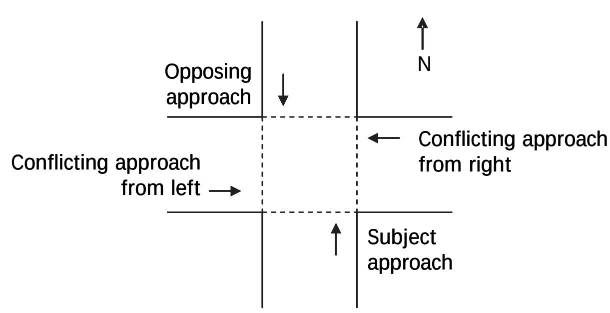
Intersection of Two Two-Way Streets

In this simplified model, the saturation headway for a vehicle assumes one of two values, h_{s1} or h_{s2}, because vehicles are again assumed to pass straight through the intersection. The departure headway for vehicles on an approach is the expected value of this bivalued distribution. A northbound vehicle will have a saturation headway of h_{s1} if the eastbound and westbound approaches are empty simultaneously. The probability of this event is the product of the probability of an empty westbound approach and the probability of an empty eastbound approach. The departure headway for the northbound vehicle is computed with Equation 20-5. See Exhibit 20-5.

Equation 20-5

$$h_{d,N} = h_{s1}\left(1 - x_E\right)\left(1 - x_W\right) + h_{s2}\left[1 - \left(1 - x_E\right)\left(1 - x_W\right)\right]$$

Exhibit 20-5
AWSC Configuration: Formulation 2



Unlike Formulation 1, it is not possible to solve directly for the departure headway in terms of a combination of arrival rates and the bivalued saturation headways. The departure headway on any approach depends on, or is directly coupled with, the traffic intensity on the two conflicting approaches. This coupling prevents a direct solution. However, it is possible to solve for the departure headway on each approach in an iterative manner, by using a system of equations similar in form to Equation 20-5.

Generalized Model for Single-Lane Sites

Capacity is determined by an iterative procedure.

The generalized model is based on five saturation headway values, each reflecting a different level or degree of conflict faced by the subject approach driver. Exhibit 20-6 specifies the conditions for each case and the probability of occurrence of each. The probability of occurrence is based on the degree of utilization on the opposing and conflicting approaches. The essence of the model, and its complexity, is evident when one realizes that the traffic intensity on one approach is computed from its capacity, which in turn depends on the traffic intensity on the other approaches. The interdependence of the traffic flow on all intersection approaches creates the need for iterative calculations to obtain stable estimates of departure headway and service time—and thus capacity.

Exhibit 20-6
Probability of Degree-of-Conflict Case

Degree-of-Conflict Case	Approach Opp	Approach Con-L	Approach Con-R	Probability of Occurrence
1	N	N	N	$(1 - x_O)(1 - x_{CL})(1 - x_{CR})$
2	Y	N	N	$(x_O)(1 - x_{CL})(1 - x_{CR})$
3	N	Y	N	$(1 - x_O)(x_{CL})(1 - x_{CR})$
3	N	N	Y	$(1 - x_O)(1 - x_{CL})(x_{CR})$
4	Y	N	Y	$(x_O)(1 - x_{CL})(x_{CR})$
4	Y	Y	N	$(x_O)(x_{CL})(1 - x_{CR})$
4	N	Y	Y	$(1 - x_O)(x_{CL})(x_{CR})$
5	Y	Y	Y	$(x_O)(x_{CL})(x_{CR})$

Note: Opp = opposing approach, Con-L = conflicting approach from the left, Con-R = conflicting approach from the right, N = no, Y = yes.

The probability, $P(C_i)$, for each degree-of-conflict case given in Exhibit 20-6 can be computed with Equation 20-6 through Equation 20-10. The degrees of utilization on the opposing approach, the conflicting approach from the left, and the conflicting approach from the right are given by x_O, x_{CL}, and x_{CR}, respectively.

$$P(C_1) = (1 - x_O)(1 - x_{CL})(1 - x_{CR})$$ **Equation 20-6**

$$P(C_2) = (x_O)(1 - x_{CL})(1 - x_{CR})$$ **Equation 20-7**

$$P(C_3) = (1 - x_O)(x_{CL})(1 - x_{CR}) + (1 - x_O)(1 - x_{CL})(x_{CR})$$ **Equation 20-8**

$$P(C_4) = (x_O)(1 - x_{CL})(x_{CR}) + (x_O)(x_{CL})(1 - x_{CR}) + (1 - x_O)(x_{CL})(x_{CR})$$ **Equation 20-9**

$$P(C_5) = (x_O)(x_{CL})(x_{CR})$$ **Equation 20-10**

The departure headway for an approach is the expected value of the saturation headway distribution, computed by Equation 20-11:

Equation 20-11

$$h_d = \sum_{i=1}^{5} P(C_i) h_{si}$$

where $P(C_i)$ is the probability of the degree-of-conflict case C_i and h_{si} is the saturation headway for that case, given the traffic stream and geometric conditions of the intersection approach.

Capacity is determined by increasing volume on the subject approach until x > 1.0.

The capacity is computed by incrementally increasing the volume on the subject approach until the degree of utilization exceeds 1.0. This flow rate is the maximum possible flow or throughput on the subject approach under the conditions used as input to the analysis.

Generalized Model for Multilane Sites

Saturation headways at multilane sites are typically longer than at single-lane sites, all other conditions being equal. This situation is the result of two factors:

- A larger intersection (i.e., greater number of lanes) requires more travel time through the intersection, thus increasing the saturation headway; and
- Additional lanes also result in an increasing degree of conflict with opposing and conflicting vehicles, again increasing driver decision time and the saturation headway.

By contrast, some movements may not conflict with each other as readily at multilane sites as at single-lane sites. For example, a northbound vehicle turning right may be able to depart simultaneously with an eastbound through movement if the two vehicles are able to occupy separate receiving lanes when departing to the east. Consequently, in some cases, the saturation headway may be lower at multilane sites.

The theory described earlier proposed that the saturation headway is a function of the directional movement of the vehicle, the vehicle type, and the degree of conflict faced by the subject vehicle. This theory is extended here for multilane sites with respect to the concept of degree of conflict: saturation headway is affected to a large extent by the number of opposing and conflicting vehicles faced by the subject driver. For example, in degree-of-conflict Case 2, a subject vehicle is faced only by a vehicle on the opposing approach. At a two-lane approach intersection, there can be either one or two vehicles on the opposing approach. Each degree-of-conflict case is expanded to consider the number of vehicles present on each of the opposing and conflicting approaches. The cases are defined in Exhibit 20-7 and Exhibit 20-8 for two-lane and three-lane approaches, respectively.

For multilane sites, separate saturation headway values are computed for the number of vehicles faced by the subject vehicle for each degree-of-conflict case. This calculation requires a further extension of the service time model to account for the increased number of subcases. These combinations can be further subdivided if a vehicle can be present on any lane on a given approach.

Exhibit 20-7
Degree-of-Conflict Cases for Two-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2
3		x	x	1, 2
4	x x 	x x	 x x	2, 3, 4
5	x	x	x	3, 4, 5, 6

Exhibit 20-8
Degree-of-Conflict Cases for Three-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2, 3
3		x	x	1, 2, 3
4	x x 	x x	 x x	2, 3, 4, 5, 6
5	x	x	x	3, 4, 5, 6, 7, 8, 9

The probability of a vehicle being at the stop line in a given lane is x, the degree of utilization. The product of the six degrees of saturation, encompassing each of the six lanes on the opposing or conflicting approaches (two lanes on the opposing approach and two lanes on each of the conflicting approaches), gives the probability of any particular combination occurring.

The iterative procedure to compute the departure headways and capacities for each approach as a function of the departure headways on the other approaches is the same as described earlier. However, the additional subcases clearly increase the complexity of this computation.

AUTOMOBILE MODE

The AWSC intersection methodology for the automobile mode is applied through a series of steps that relate to input data, saturation headways, departure headways, service time, capacity, and LOS. They are illustrated in Exhibit 20-9.

Step 1: Convert Movement Demand Volumes to Flow Rates

Flow rates for each turning movement at the intersection must be converted from hourly volumes in vehicles per hour (veh/h) to peak 15-min flow rates in vehicles per hour as given in Equation 20-12:

Equation 20-12

$$v_i = \frac{V_i}{PHF}$$

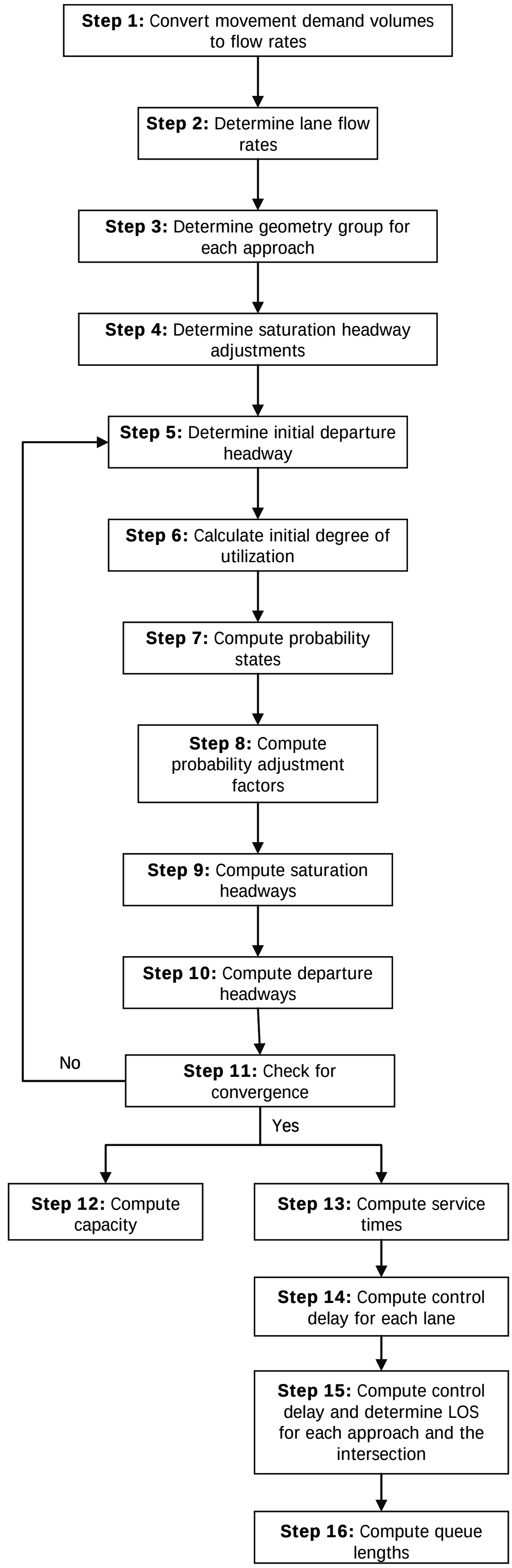
where

v_i = demand flow rate for movement i (veh/h),

V_i = demand volume for movement i (veh/h), and

PHF = peak hour factor.

Exhibit 20-9
AWSC Intersection Methodology



Step 2: Determine Lane Flow Rates

For multilane approaches, the flow rate for each lane by movement is determined. If a certain movement can use more than one lane and its traffic volume distribution per lane is unknown, an equal distribution of volume among the lanes can be assumed.

Step 3: Determine Geometry Group for Each Approach

Exhibit 20-10 is consulted to determine the geometry group for each approach. The geometry group is needed to look up base saturation headways and headway adjustment factors.

Exhibit 20-10 Geometry Groups

Intersection Configuration	Number of Lanes: Subject Approach	Number of Lanes: Opposing Approach	Number of Lanes: Conflicting Approaches[a]	Geometry Group
Four leg or T	1	0 or 1	1	1
Four leg or T	1	0 or 1	2	2
Four leg or T	1	2	1	3a/4a
T	1	2	2	3b
Four leg	1	2	2	4b
Four leg or T	1	0 or 1	3	5
	1	3	1	
	2	0, 1, or 2	1 or 2	
	3	0 or 1	1	
	3	0 or 1	2 or 3	
	3	2 or 3	1	
Four leg or T	1	3	2	6
	1	2	3	
	1	3	3	
	2	3	1, 2, or 3	
	2	0, 1, 2 or 3	3	
	3	2 or 3	2 or 3	

Note: [a] If the number of lanes on conflicting approaches is different, the higher of the two should be used.

Step 4: Determine Saturation Headway Adjustments

The headway adjustment for each lane is computed by Equation 20-13. Saturation headway adjustments for left turns, right turns, and heavy vehicles are given in Exhibit 20-11.

Equation 20-13

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

where

h_{adj} = headway adjustment (s),

$h_{LT,adj}$ = headway adjustment for left turns (see Exhibit 20-11) (s),

$h_{RT,adj}$ = headway adjustment for right turns (see Exhibit 20-11) (s),

$h_{HV,adj}$ = headway adjustment for heavy vehicles (see Exhibit 20-11) (s),

P_{LT} = proportion of left-turning vehicles in the lane,

P_{RT} = proportion of right-turning vehicles in the lane, and

P_{HV} = proportion of heavy vehicles in the lane.

Exhibit 20-11
Saturation Headway Adjustments by Geometry Group

Factor	Saturation Headway Adjustment (s)							
	Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
LT	0.2	0.2	0.2	0.2	0.2	0.2	0.5	0.5
RT	–0.6	–0.6	–0.6	–0.6	–0.6	–0.6	–0.7	–0.7
HV	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

Note: LT = left turns, RT = right turns, HV = heavy vehicles.

Step 5: Determine Initial Departure Headway

The process of determining departure headways (and thus service times) for each of the lanes on each of the approaches is iterative. For the first iteration, an initial departure headway of 3.2 s should be assumed. For subsequent iterations, the calculated values of departure headway from the previous iteration should be used if the calculation has not converged (see Step 11).

Step 6: Calculate Initial Degree of Utilization

By using the lane flow rates from Step 2 and the assumed initial departure headway from Step 5, the initial degree of utilization, x, is computed with Equation 20-14. If it is not the final iteration, and the degree of utilization exceeds 1, then the degree of utilization should be reset to 1.

Equation 20-14

$$x = \frac{vh_d}{3{,}600}$$

Step 7: Compute Probability States

The probability state of each combination i is found with Equation 20-15.

Equation 20-15

$$P(i) = \prod_j P(a_j)$$

where

j = O1 (opposing approach, Lane 1), O2 (opposing approach, Lane 2), CL1 (conflicting left approach, Lane 1), CL2 (conflicting left approach, Lane 2), CR1 (conflicting right approach, Lane 1), and CR2 (conflicting right approach, Lane 2) for a two-lane, two-way AWSC intersection;

$P(a_j)$ = probability of a_j, computed on the basis of Exhibit 20-12, where V_j is the lane flow rate; and

a_j = 1 (indicating a vehicle present) or 0 (indicating no vehicle present in the lane) (values of a_j for each lane in each combination i are listed in Exhibit 20-13).

Exhibit 20-12
Probability of a_j

a_j	V_j	$P(a_j)$
1	0	0
0	0	1
1	> 0	x_j
0	> 0	$1 - x_j$

Note: x is the degree of utilization defined in Equation 20-14.

Tables for three-lane approaches are given in Chapter 32, STOP-Controlled Intersections: Supplemental

Exhibit 20-13 provides the 64 possible combinations when alternative lane occupancies are considered for two-lane approaches. A 1 indicates that a vehicle is in the lane, and a 0 indicates that a vehicle is not in the lane. A similar table for three lanes on each approach is provided in Chapter 32 in Volume 4.

Exhibit 20-13
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Two-Lane Approaches, by Lane)

i	DOC Case	Number of Vehicles	Opposing Approach		Conflicting Left Approach		Conflicting Right Approach	
			L1	L2	L1	L2	L1	L2
1	1	0	0	0	0	0	0	0
2	2	1	1	0	0	0	0	0
3			0	1	0	0	0	0
4		2	1	1	0	0	0	0
5	3	1	0	0	1	0	0	0
6			0	0	0	1	0	0
7			0	0	0	0	1	0
8			0	0	0	0	0	1
9		2	0	0	1	1	0	0
10			0	0	0	0	1	1
11	4	2	0	0	0	1	0	1
12			0	0	1	0	0	1
13			0	0	1	0	1	0
14			0	0	0	1	1	0
15			0	1	0	1	0	0
16			1	0	1	0	0	0
17			0	1	0	0	1	0
18			1	0	0	1	0	0
19			0	1	1	0	0	0
20			0	1	0	0	0	1
21			1	0	0	0	1	0
22			1	0	0	0	0	1
23		3	0	0	0	1	1	1
24			0	0	1	1	0	1
25			0	0	1	1	1	0
26			1	0	1	1	0	0
27			1	1	1	0	0	0
28			1	1	0	0	1	0
29			1	1	0	0	0	1
30			0	1	1	1	0	0
31			1	0	0	0	1	1
32			0	0	1	0	1	1
33			1	1	0	1	0	0
34			0	1	0	0	1	1
35		4	1	1	0	0	1	1
36			0	0	1	1	1	1
37			1	1	1	1	0	0
38	5	3	0	1	0	1	0	1
39			1	0	0	1	1	0
40			0	1	1	0	1	0
41			0	1	0	1	1	0
42			0	1	1	0	0	1
43			1	0	1	0	0	1
44			1	0	0	1	0	1
45			1	0	1	0	1	0
46		4	1	0	0	1	1	1
47			0	1	1	1	1	0
48			0	1	1	1	0	1
49			1	0	1	0	1	1
50			1	0	1	1	1	0
51			0	1	0	1	1	1
52			1	1	1	0	0	1
53			1	0	1	1	0	1
54			0	1	1	0	1	1
55			1	1	0	1	1	0
56			1	1	0	1	0	1
57			1	1	1	0	1	0
58		5	1	0	1	1	1	1
59			1	1	0	1	1	1
60			1	1	1	0	1	1
61			0	1	1	1	1	1
62			1	1	1	1	1	0
63			1	1	1	1	0	1
64		6	1	1	1	1	1	1

Note: DOC case is the degree-of-conflict case, number of vehicles is the total number of vehicles on the opposing and conflicting approaches, L1 is Lane 1, and L2 is Lane 2.

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 20-16 through Equation 20-20 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined (assuming the 64 cases presented in Exhibit 20-13).

Equation 20-16
$$P(C_1) = P(1)$$

Equation 20-17
$$P(C_2) = \sum_{i=2}^{4} P(i)$$

Equation 20-18
$$P(C_3) = \sum_{i=5}^{10} P(i)$$

Equation 20-19
$$P(C_4) = \sum_{i=11}^{37} P(i)$$

Equation 20-20
$$P(C_5) = \sum_{i=38}^{64} P(i)$$

The probability adjustment factors are then computed with Equation 20-21 through Equation 20-25.

Equation 20-21
$$AdjP(1) = \alpha\left[P(C_2) + 2P(C_3) + 3P(C_4) + 4P(C_5)\right]/1$$

Equation 20-22
$$AdjP(2) \text{ through } AdjP(4) = \alpha\left[P(C_3) + 2P(C_4) + 3P(C_5) - P(C_2)\right]/3$$

Equation 20-23
$$AdjP(5) \text{ through } AdjP(10) = \alpha\left[P(C_4) + 2P(C_5) - 3P(C_3)\right]/6$$

Equation 20-24
$$AdjP(11) \text{ through } AdjP(37) = \alpha\left[P(C_5) - 6P(C_4)\right]/27$$

Equation 20-25
$$AdjP(38) \text{ through } AdjP(64) = -\alpha\left[10P(C_5)\right]/27$$

where α equals 0.01 (or 0.00 if correlation among saturation headways is not taken into account).

The adjusted probability $P'(i)$ for each combination is simply the sum of $P(i)$ and $AdjP(i)$, as given by Equation 20-26.

Equation 20-26
$$P'(i) = P(i) + AdjP(i)$$

Step 9: Compute Saturation Headways

The saturation headway h_{si} is the sum of the base saturation headway as presented in Exhibit 20-14 and the saturation headway adjustment factor from Step 4. It is shown in Equation 20-27.

Equation 20-27
$$h_{si} = h_{base} + h_{adj}$$

Case	No. of Veh.	Base Saturation Headway (s) Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
1	0	3.9	3.9	4.0	4.3	4.0	4.5	4.5	4.5
2	1	4.7	4.7	4.8	5.1	4.8	5.3	5.0	6.0
	2							6.2	6.8
	≥3								7.4
3	1	5.8	5.8	5.9	6.2	5.9	6.4	6.4	6.6
	2							7.2	7.3
	≥3								7.8
4	2	7.0	7.0	7.1	7.4	7.1	7.6	7.6	8.1
	3							7.8	8.7
	4							9.0	9.6
	≥5								12.3
5	3	9.6	9.6	9.7	10.0	9.7	10.2	9.7	10.0
	4							9.7	11.1
	5							10.0	11.4
	≥6							11.5	13.3

Step 10: Compute Departure Headways

The departure headway of the lane is the expected value of the saturation headway distribution, given by Equation 20-28.

Equation 20-28

$$h_d = \sum_{i=1}^{64} P'(i) h_{si}$$

where i represents each combination of the five degree-of-conflict cases and h_{si} is the saturation headway for that combination.

Step 11: Check for Convergence

The calculated values of h_d are checked against the initial values assumed for h_d. If the values change by more than 0.1 s (or a more precise measure of convergence), Steps 5 through 10 are repeated until the values of departure headway for each lane do not change significantly.

Step 12: Compute Capacity

Capacity is estimated for a stated set of opposing and conflicting volumes.

The capacity of each lane in a subject approach is computed under the assumption that the flows on the opposing and conflicting approaches are constant. The given flow rate on the subject lane is increased and the departure headways are computed for each lane on each approach until the degree of utilization for the subject lane reaches 1. When this occurs, the final value of the subject lane flow rate is the maximum possible throughput or capacity of this lane.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time with Equation 20-29.

Equation 20-29

$$t_s = h_d - m$$

where t_s is the service time, h_d is the departure headway, and m is the move-up time (2.0 s for Geometry Groups 1 through 4; 2.3 s for Geometry Groups 5 and 6).

Step 14: Compute Control Delay for Each Lane

The delay experienced by a motorist is made up of a number of factors that relate to control, geometrics, traffic, and incidents. Control delay is the difference between the travel time that is actually experienced and the reference travel time that would result during conditions in the absence of traffic control or conflicting traffic.

Equation 20-30 can be used to compute control delay for each lane.

Equation 20-30

$$d = t_s + 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{h_d x}{450T}}\right] + 5$$

where

d = average control delay (s/veh),

x = $vh_d/3{,}600$ = degree of utilization,

t_s = service time (s),

h_d = departure headway (s), and

T = length of analysis period (h).

Step 15: Compute Control Delay and Determine LOS for Each Approach and the Intersection

The control delay for an approach is calculated by computing a weighted average of the delay for each lane on the approach, weighted by the volume in each lane. The calculation is shown in Equation 20-31.

Equation 20-31

$$d_{\text{approach}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

d_{approach} = control delay for the approach (s/veh),

d_i = control delay for lane i (s/veh), and

v_i = flow rate for lane i (veh/h).

The control delay for the intersection as a whole is similarly calculated by computing a weighted average of the delay for each approach, weighted by the volume on each approach. It is shown in Equation 20-32.

Equation 20-32

$$d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

$d_{\text{intersection}}$ = control delay for the entire intersection (s/veh),

d_i = control delay for approach i (s/veh), and

v_i = flow rate for approach i (veh/h).

The LOS for each approach and for the intersection are determined with Exhibit 20-2 and the computed values of control delay.

Step 16: Compute Queue Lengths

Research (3) has determined that the methodology for predicting queues at TWSC intersections can be applied to AWSC intersections. As such, the mean queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest.

Equation 20-33 can be used to calculate the 95th percentile queue for each approach lane.

$$Q_{95} \approx \frac{900T}{h_d}\left[(x-1)+\sqrt{(x-1)^2+\frac{h_d x}{150T}}\right]$$

Equation 20-33

where

Q_{95} = 95th percentile queue (veh),

x = $vh_d/3{,}600$ = degree of utilization,

h_d = departure headway (s), and

T = length of analysis period (h).

PEDESTRIAN MODE

Data collection and research are needed to determine an appropriate LOS methodology for pedestrians at AWSC intersections.

Applying the LOS procedures used to determine pedestrian delay at TWSC intersections to AWSC intersections does not produce intuitive or usable results. The TWSC delay calculations apply only for crossings where conflicting traffic is not STOP-controlled (i.e., pedestrians crossing the major street at a TWSC intersection). Approaches where conflicting traffic is STOP-controlled (i.e., pedestrians crossing the minor street at a TWSC intersection) are assumed to result in negligible delay for pedestrians, as vehicles are required to stop and wait for conflicting vehicle and pedestrian traffic before proceeding.

As such, applying the TWSC methodology to pedestrians at AWSC intersections results in negligible delay for all pedestrians at all approaches. The reality of AWSC intersection operations for pedestrians is much different, however, and generally results in at least some delay for pedestrians. The amount of delay incurred will depend on a number of operating and geometric characteristics of the intersection in question. While no quantitative methodology accounting for these factors is available, several of the most important factors are discussed qualitatively below.

The operational characteristics of AWSC intersections for pedestrians largely depend on driver behavior. In most cases, drivers are legally required to yield to pedestrians crossing or preparing to cross AWSC intersections. However, it should be expected that operations differ significantly depending on enforcement levels, region of the country, and location (e.g., urban, suburban, or rural).

Traffic Volumes

At intersections with relatively low traffic volumes, there are generally no standing queues of vehicles at AWSC approaches. In these cases, pedestrians attempting to cross an approach of the intersection will typically experience little or no delay, as they will be able to proceed almost immediately after reaching the intersection.

At AWSC intersections with higher volumes, there are typically standing queues of motor vehicles on each approach. These intersections operate in a two-phase or four-phase sequence, as described earlier and depicted in Exhibit 20-3. In these situations, the arrival of a pedestrian does not typically disrupt the normal phase operations of the intersection. Rather, the pedestrian is often forced to wait until the phase arrives for vehicles in the approach moving adjacent to the pedestrian.

Under a scenario in which the intersection functions under the operations described above for pedestrians, average pedestrian delay might be expected to be half of the time needed to cycle through all phases for the particular intersection, assuming random arrival of pedestrians. However, several other factors may also affect pedestrian delay and operations at AWSC intersections, as described below.

Number of Approach Lanes

As the number of approach lanes at AWSC intersections increases, pedestrian crossing distance increases proportionally, resulting in significantly longer pedestrian crossing times compared with single-lane intersections. In addition, vehicles already in the intersection or about to enter the intersection take longer to complete their movement. As a result, pedestrians at multilane AWSC intersections may wait longer before taking their turn to cross.

Proportion of Turning Traffic

The ability of a pedestrian to cross at an AWSC intersection may also depend on the proportion of through motor vehicle traffic to turning motor vehicle traffic. As described above, pedestrians may often cross during the phase in which adjacent motor vehicle traffic traverses the intersection. However, when an adjacent motor vehicle is turning, that vehicle will conflict with pedestrians attempting to cross. Because of the additional conflicts with pedestrians created by turning vehicles at AWSC intersections, pedestrian delay may be expected to rise as the proportion of turning vehicles increases, similar to the effect that turning proportion has on vehicular delay.

Pedestrian Volumes

Under most circumstances, there is adequate capacity for all pedestrians queued for a given movement at an AWSC intersection to cross simultaneously with adjacent motor vehicle traffic. However, in locations with very high pedestrian volumes, this may not be the case. The total pedestrian capacity of a particular AWSC intersection phase is limited by both the width of the crosswalk (how many pedestrians can cross simultaneously) and driver behavior.

In situations in which not all queued pedestrians may cross during a particular phase, pedestrian delay will increase, as some pedestrians will be forced to wait through an additional cycle of intersection phases before crossing. However, pedestrian volumes in this range are unlikely to occur often; rather, intersections with pedestrian volumes high enough to cause significant delay are typically signalized.

BICYCLE MODE

Where bicycles queue with motor vehicles on AWSC approaches, the procedures described to estimate motor vehicle delay can be applied to bicycles. However, bicycles differ from motor vehicles in that they do not queue linearly at STOP signs. Instead, multiple bicycles often cross at the same time as the adjacent vehicular traffic stream. This phenomenon has not been researched as of the time of publication of this edition of the HCM and is not explicitly included in the methodology.

Where an AWSC approach provides a bicycle lane, bicycle delay will be significantly different and, in general, lower than motor vehicle delay. The exception is bicycles intending to turn left; those cyclists will typically queue with motor vehicles. Where bicycle lanes are available, bicycles are able to move unimpeded until reaching the stop line, as the bike lane allows the cyclist to pass any queued motor vehicles on the right. In this situation, bicycles will still incur delay upon reaching the intersection.

In most cases, bicycles will be able to travel through the intersection concurrently with adjacent motor vehicle traffic. This, in effect, results in multilane operations, with the bike lane serving as the curb lane, meaning that bicycles will be delayed from the time of arrival at the intersection until the adjacent motor vehicle phase occurs. As noted above, multiple bicycles will likely be able to cross simultaneously through the intersection. Finally, even where bicycle lanes are not available, many cyclists still pass queued motor vehicles on the right, resulting in lower effective bicycle delay compared with motor vehicle delay.

3. APPLICATIONS

DEFAULT VALUES

A comprehensive presentation of potential default values for interrupted-flow facilities is available elsewhere (*4*). These defaults cover the key characteristics of peak hour factor (*PHF*) and percent heavy vehicles (%HV). Recommendations are based on geographic region, population, and time of day. All general default values for interrupted flow facilities may be applied to analysis of AWSC intersections in the absence of field data or projections of conditions.

Both demand volumes and the number and configuration of lanes at the intersection are site specific and do not lend themselves to default values. The following default values may be applied to an AWSC intersection analysis:

- Peak hour factor (*PHF*) = 0.92
- Percent heavy vehicles (%HV) = 3%

As the number of default values used in any analysis increases, the accuracy of the result becomes more approximate and may differ significantly from the actual outcome, depending on local conditions.

ESTABLISH INTERSECTION ANALYSIS BOUNDARIES

This methodology assumes that the AWSC intersection under investigation is isolated. When interaction effects are likely between the subject AWSC intersection and other intersections (e.g., queue spillback, demand starvation), the use of alternative tools may result in a more accurate analysis. Analysis boundaries may also include different demand scenarios related to time of day or to different development scenarios that produce different demand flow rates.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design analysis.

Operational Analysis

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement (veh/h), heavy vehicle percentages for each approach, peak hour factor for all demand volumes, and lane configuration. The outputs of an operational analysis are estimates of capacity and control delay. The steps of the methodology, described in the Methodology section, are followed directly without modification.

Design Analysis

The operational analysis methodology for AWSC intersections can also be used for design analysis and planning and preliminary engineering analysis.

The operational analysis described earlier in this chapter can be used for design purposes by using a given set of traffic flow data and iteratively determining the number and configuration of lanes that would be required to produce a given LOS.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of an AWSC intersection. To estimate LOS for a future time horizon, a planning analysis based on the operational method is used. The planning method uses all the geometric and traffic flow data required for an operational analysis, and the computations are identical. However, input variables for heavy-vehicle percentage and peak hour factor are typically estimated (or defaults used) when planning applications are performed.

USE OF ALTERNATIVE TOOLS

Except for the effects of interaction with other intersections, the limitations of the methodology that were stated earlier in this chapter have minimal potential to be addressed by alternative tools. Therefore, insufficient experience with alternative tools is available as of the time of publication of this edition of the HCM to support the development of useful guidance for their application to AWSC intersections.

4. EXAMPLE PROBLEM

EXAMPLE PROBLEM 1: SINGLE-LANE, T-INTERSECTION

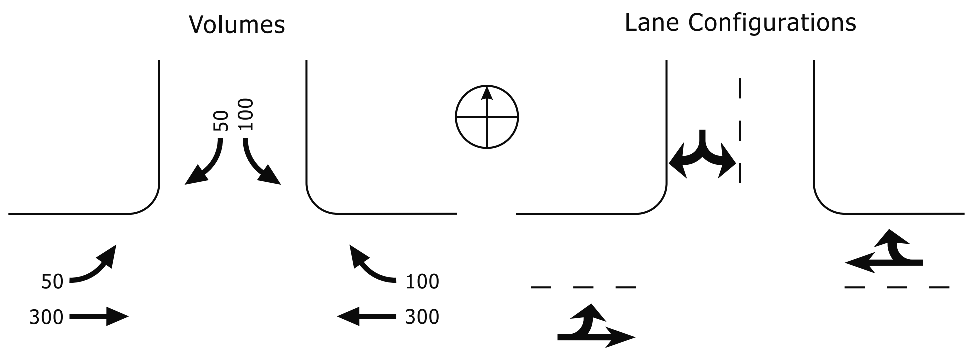
An additional AWSC example problem is provided in Chapter 32, STOP-Controlled Intersections: Supplemental.

The Facts

The following describes this location's traffic and geometric characteristics:

- Three legs (T-intersection),
- One-lane entries on each leg,
- Percent heavy vehicles on all approaches = 2%,
- Peak hour factor = 0.95, and
- Volumes and lane configurations as shown below (Exhibit 20-15).

Exhibit 20-15 Volumes and Lane Configurations for Example Problem 1



Comments

The use of a spreadsheet or software for AWSC intersection analysis is recommended because of the repetitive and iterative computations required.

All input parameters are known, so no default values are needed or used. The use of a spreadsheet or software is recommended because of the repetitive computations required. Slight differences in reported values may result from rounding differences between manual and software computations.

Step 1: Convert Movement Demand Volumes to Flow Rates

Peak 15-min flow rates for each turning movement at the intersection are equal to the hourly volumes divided by *PHF*. For example, the peak 15-min flow rate for the eastbound through movement is as follows:

$$v_{EBTH} = \frac{V_{EBTH}}{PHF} = \frac{300}{0.95} = 316 \text{ veh/h}$$

Step 2: Determine Lane Flow Rates

This step does not apply because the intersection has one-lane approaches on all legs.

Step 3: Determine Geometry Group for Each Approach

Exhibit 20-10 shows that each approach should be assigned to Geometry Group 1.

Step 4: Determine Saturation Headway Adjustments

Exhibit 20-11 shows that the headway adjustments for left turns, right turns, and heavy vehicles are 0.2, –0.6, and 1.7, respectively. These values apply to all approaches because all are assigned Geometry Group 1. Therefore, the saturation headway adjustment for the eastbound approach is as follows:

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

$$h_{adj} = 0.2\frac{53}{53+316} - 0.6(0) + 1.7(0.02) = 0.063$$

Similarly, the saturation headway adjustment for the westbound approach is as follows:

$$h_{adj} = 0.2(0) - 0.6\frac{105}{105+316} + 1.7(0.02) = -0.116$$

Finally, the saturation headway adjustment for the southbound approach is as follows:

$$h_{adj} = 0.2\frac{105}{105+53} - 0.6\frac{53}{105+53} + 1.7(0.02) = -0.034$$

Steps 5 Through 11: Determine Departure Headway

These steps are iterative. The following narrative highlights some of the key calculations using the eastbound approach for Iteration 1 but does not attempt to reproduce all calculations for all iterations. Full documentation of the example problem is included in Chapter 32, STOP-Controlled Intersections: Supplemental.

Step 6: Calculate Initial Degree of Utilization

By using the lane flow rates from Step 2 and the assumed initial departure headway from Step 5, the initial degree of utilization, x, for the eastbound approach is computed as follows:

$$x, EB = \frac{vh_d}{3{,}600} = \frac{(368)(3.2)}{3{,}600} = 0.327$$

$$x, WB = \frac{(421)(3.2)}{3{,}600} = 0.374$$

$$x, NB = \frac{vh_d}{3{,}600} = \frac{(158)(3.2)}{3{,}600} = 0.140$$

Step 7: Compute Probability States

The probability state of each combination i is determined with Equation 20-15.

$$P(i) = \prod_j P(a_j) = P(a_O)P(a_{CL})P(a_{CR})$$

For an intersection with single-lane approaches, only these eight cases from Exhibit 20-13 apply:

i	DOC Case	Number of Vehicles	Opposing Approach	Conflicting Left Approach	Conflicting Right Approach
1	1	0	0	0	0
2	2	1	1	0	0
5	3	1	0	1	0
7	3	1	0	0	1
13	4	2	0	1	1
16	4	2	1	1	0
21	4	2	1	0	1
45	5	3	1	1	1

For example, the probability state for the eastbound leg under the condition of no opposing vehicles on the other approaches (degree of conflict Case 1, $i = 1$) is as follows (using Exhibit 20-6):

$P(a_O) = 1 - x_O = 1 - 0.374 = 0.626$ (no opposing present)

$P(a_{CL}) = 1 - x_{CL} = 1 - 0.140 = 0.860$ (no conflicting from left present)

$P(a_{CR}) = 1$ (no approach conflicting from right)

Therefore,

$$P(1) = P(a_O)P(a_{CL})P(a_{CR}) = (0.626)(0.860)(1) = 0.538$$

Similarly,

$$P(2) = (0.374)(0.860)(1) = 0.322$$

$$P(5) = (0.626)(0.140)(1) = 0.088$$

$$P(7) = (0.626)(0.860)(0) = 0$$

$$P(13) = (0.626)(0.140)(0) = 0$$

$$P(16) = (0.374)(0.140)(1) = 0.052$$

$$P(21) = (0.374)(0.860)(0) = 0$$

$$P(45) = (0.374)(0.140)(0) = 0$$

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed as follows:

$$P(C_1) = P(1) = 0.538$$

$$P(C_2) = P(2) = 0.322$$

$$P(C_3) = P(5) + P(7) = 0.088 + 0 = 0.088$$

$$P(C_4) = P(13) + P(16) + P(21) = 0 + 0.052 + 0 = 0.052$$

$$P(C_5) = P(45) = 0$$

The probability adjustment factors for the nonzero cases are as follows:

$$AdjP(1) = 0.01[0.322 + 2(0.088) + 3(0.052) + 0]/1 = 0.0065$$

$$AdjP(2) = 0.01[(0.088) + 2(0.052) + 0 - 0.322]/3 = -0.0004$$

$$AdjP(5) = 0.01[(0.052) + 2(0) - 3(0.088)]/6 = -0.0004$$

$$AdjP(16) = 0.01[0 - 6(0.052)]/27 = -0.0001$$

Therefore, the adjusted probability for Combination 1, for example, is as follows:

$$P'(1) = 0.538 + 0.0065 = 0.5445$$

Step 9: Compute Saturation Headways

The base saturation headways for each combination can be determined with Exhibit 20-14. They are adjusted by using the adjustment factors calculated in Step 4 and added to the base saturation headways to determine saturation headways as follows (eastbound illustrated):

i	h_{base}	h_{adj}	h_{si}
1	3.9	0.063	3.963
2	4.7	0.063	4.763
5	5.8	0.063	5.863
7	7.0	0.063	7.063

Step 10: Compute Departure Headways

The departure headway of the lane is the sum of the products of the adjusted probabilities and the saturation headways as follows (eastbound illustrated):

$$h_d = (0.5445)(3.963) + (0.3213)(4.763) + (0.0875)(5.863) + (0.0524)(7.063) = 4.57$$

Step 11: Check for Convergence

The calculated values of h_d are checked against the initial values assumed for h_d. After one iteration, each calculated headway differs from the initial value by more than 0.1 s. Therefore, the new calculated headway values are used as initial values in a second iteration. For this problem, four iterations are required for convergence.

	EB L1	EB L2	WB L1	WB L2	NB L1	NB L2	SB L1	SB L2
Total Lane Flow Rate	368		421				158	
hd, initial value, iteration 1	3.2		3.2				3.2	
x, initial, iteration 1	0.327		0.374				0.140	
hd, computed value, iteration 1	4.57		4.35				5.14	
Convergence?	N		N				N	
hd, initial value, iteration 2	4.57		4.35				5.14	
x, initial, iteration 2	0.468		0.509				0.225	
hd, computed value, iteration 2	4.88		4.66				5.59	
Convergence?	N		N				N	
hd, initial value, iteration 3	4.88		4.66				5.59	
x, initial, iteration 3	0.499		0.545				0.245	
hd, computed value, iteration 3	4.95		4.73				5.70	
Convergence?	Y		Y				N	
hd, initial value, iteration 4	4.88		4.66				5.70	
x, initial, iteration 4	0.499		0.545				0.250	
hd, computed value, iteration 4	4.97		4.74				5.70	
Convergence?	Y		Y				Y	

Step 12: Compute Capacity

The capacity of each lane in a subject approach is computed by increasing the given flow rate on the subject lane (assuming the flows on the opposing and conflicting approaches are constant) until the degree of utilization for the subject lane reaches 1. This level of calculation requires running an iterative procedure many times, which is practical for a spreadsheet or software implementation.

Here, the eastbound lane capacity is approximately 720 veh/h, which is lower than the value that could be estimated by dividing the lane volume by the degree of utilization (368/0.492 = 748 veh/h). The difference is due to the interaction effects among the approaches: increases in eastbound traffic volume increase the departure headways of the lanes on the other approaches, which in turn increases the departure headway of the lane(s) on the subject approach.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time by using Equation 20-29. For the eastbound lane (using a value for m of 2.0 for Geometry Group 1), the calculation is as follows:

$$t_s = h_d - m = 4.97 - 2.0 = 2.97$$

Step 14: Compute Control Delay

The control delay for each lane is computed with Equation 20-30 as follows (eastbound illustrated):

$$d = 2.97 + 900(0.25)\left[(0.508-1) + \sqrt{(0.508-1)^2 + \frac{4.97(0.508)}{450(0.25)}}\right] + 5 = 13.0\text{ s}$$

By using Exhibit 20-2, the eastbound lane (and thus approach) is assigned LOS B. A similar calculation for the westbound and southbound lanes (and thus approaches) yields 13.5 and 10.6 s, respectively.

The control delays for the approaches can be combined into an intersection control delay by using a weighted average as follows:

$$d = \frac{(13.0)(368) + (13.5)(421) + (10.6)(158)}{368 + 421 + 158} = 12.8\text{ s}$$

This value of delay is assigned LOS B.

Step 15: Compute Queue Length

The 95th percentile queue for each lane is computed with Equation 20-33 as follows (eastbound approach illustrated):

$$Q_{95} \approx \frac{900(0.25)}{4.97}\left[(0.508-1) + \sqrt{(0.508-1)^2 + \frac{4.97(0.508)}{150(0.25)}}\right] = 2.9\text{ veh}$$

This queue length would be reported as 3 vehicles.

Discussion

The results indicate that the intersection operates well with low delays.

5. REFERENCES

These references are available in the Technical Reference Library in Volume 4.

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