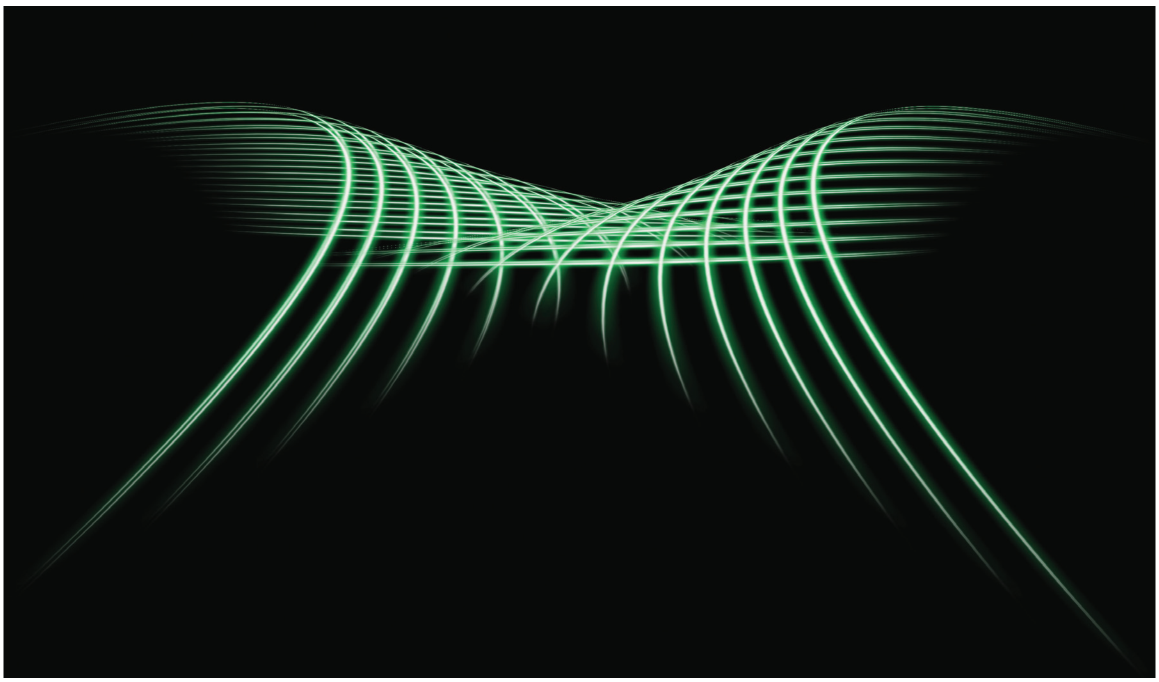
HCM2010

HIGHWAY CAPACITY MANUAL



CHAPTER 34

INTERCHANGE RAMP TERMINALS: SUPPLEMENTAL

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HCM2010

HIGHWAY CAPACITY MANUAL

The HCM 2010 significantly enhances how engineers and planners assess the traffic and environmental effects of highway projects by

- Providing an integrated multimodal approach to the analysis and evaluation of urban streets from the points of view of automobile drivers, transit passengers, bicyclists, and pedestrians;
- Addressing the proper application of microsimulation analysis and the evaluation of the results;
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CHAPTER 34
INTERCHANGE RAMP TERMINALS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. EXAMPLE PROBLEMS

INTRODUCTION

This supplemental chapter provides a complete solution of the first two problems of Chapter 22, Interchange Ramp Terminals, as well as six additional examples related to the analysis of interchange ramp terminals. Exhibit 34-1 describes each of the eight example problems included in this supplemental chapter and indicates the methodology applied.

Exhibit 34-1
Example Problem Descriptions

Example Problem	Description	Application
1	Diamond interchange	Operational
2	Parclo A-2Q interchange	Operational
3	Diamond interchange with four-phase signalization and queue spillback	Operational
4	Diamond interchange with demand starvation	Operational
5	Single-point urban interchange	Operational
6	Diamond interchange with closely spaced intersections	Operational
7	Diamond interchange with roundabouts	Operational
8	Diamond interchange analysis using simulation	Alternative tools

EXAMPLE PROBLEM 1: DIAMOND INTERCHANGE

The Interchange

The interchange of I-99 (northbound/southbound, NB/SB) and University Drive (eastbound/westbound, EB/WB) is a diamond interchange. Exhibit 34-2 provides the interchange volumes and channelization, while Exhibit 34-3 provides the signalization information. The offset is referenced to the beginning of green on the EB direction of the arterial.

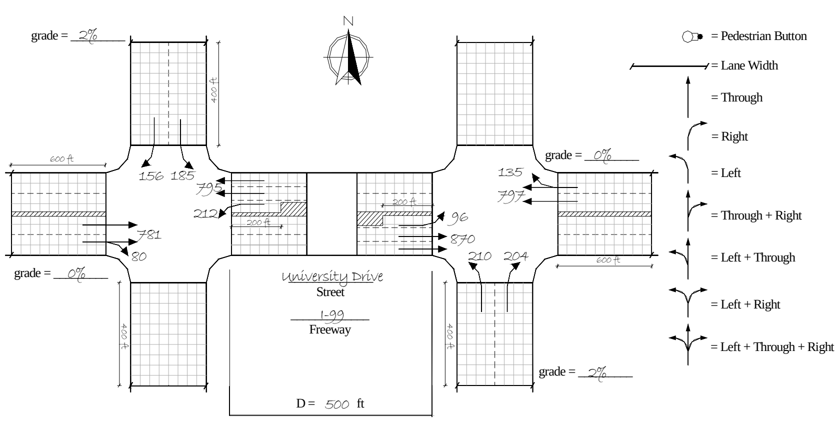


Exhibit 34-2
Example Problem 1: Interchange Volumes and Channelization

Exhibit 34-3
Example Problem 1: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (4+8)	Φ (4+7)	Φ (2+5)	Φ (4+8)	Φ (1+6)	Φ (3+8)
Green time	63	43	39	63	53	29
Yellow + all red	5	5	5	5	5	5
Offset		19			9	

The Question

What are the control delay, queue storage ratio, and level of service (LOS) for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 50 ft for all right-turning movements and 75 ft for all left-turning movements. Arrival Type 4 is assumed for all arterial movements and Arrival Type 3 for all other movements.

There are 5% heavy vehicles on both the external and the internal through movements, and the peak hour factor (PHF) for the interchange is estimated to be 0.90. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of Origin–Destination Movements

Origin–destination (O-D) movements through this diamond interchange are calculated on the basis of Exhibit 22-21(a). Since all movements utilize the signal, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 34-4.

Exhibit 34-4
Example Problem 1: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	210	233
B	204	227
C	156	173
D	185	206
E	96	107
F	80	89
G	135	150
H	212	236
I	685	761
J	585	650
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

Both external approaches to this interchange consist of a two-lane shared right and through lane group. Use of the two-lane model of Exhibit 22-16 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 34-5.

Exhibit 34-5
Example Problem 1: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	Maximum Lane Utilization	Lane Utilization Factor
Eastbound external	0.5056	0.4944	0.5056	0.9890
Westbound external	0.5181	0.4819	0.5181	0.9651

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-5. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equations 22-6 through 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated as indicated in Chapter 18, Signalized Intersections. The estimated saturation flow rates for all approaches are shown in Exhibit 34-6 and Exhibit 34-7.

Common Green and Lost Time due to Downstream Queue and Demand Starvation Calculations

Exhibit 34-8 first provides the beginning and end times of the green for each phase at the two intersections assuming that time zero is when Phase 1 of the first intersection begins. On the basis of the information provided in Exhibit 34-3, the relative offset between the two intersections is Offset 2 – Offset 1 + n × cycle length = 9 – 19 + 160 = 150 s. Next, the exhibit provides the beginning and end of green for the six pairs of movements between the two intersections and the respective common green time for each pair of movements. For example, the EB external through movement has the green between 0 and 63 s, while the EB internal through movement has the green twice during the cycle, between 150 and 53 s and between 116 and 150 s. The common green time when both movements have the green is between 0 and 53 s, for a duration of 53 s.

Exhibit 34-6
Example Problem 1: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

	Eastbound			Westbound		
	EXT-TH&R	**INT-TH**	**INT-L**	**EXT-TH&R**	**INT-TH**	**INT-L**
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	2	2	1	2	2	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	1.000	0.952	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.989	0.952	1.000	0.965	0.952	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.930	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	0.999	1.000	1.000	0.998	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.991	1.000	0.930	0.985	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	1.034	1.036	0.963	1.044	1.026	1.000
Adjusted saturation flow (s, veh/hg/ln)	3,700	3,568	1,703	3,637	3,535	1,767

Exhibit 34-7
Example Problem 1: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

	Northbound Left	Northbound Right	Southbound Left	Southbound Right
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	1	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	1.000	1.000	1.000	1.000
Grade adjustment (f_g)	0.990	0.990	0.990	0.990
Parking adjustment (f_p)	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	0.930	1.000	0.930	1.000
Right-turn adjustment (f_{RT})	1.000	0.899	1.000	0.899
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.930	0.899	0.930	0.899
Traffic pressure adjustment for lane group (f_v)	1.000	0.979	0.991	0.968
Adjusted saturation flow (s, veh/hg/ln)	1,749	1,656	1,734	1,638

Exhibit 34-8
Example Problem 1: Common Green Calculations

Phase	Intersection I Green Begin	Intersection I Green End	Intersection II Green Begin	Intersection II Green End
Phase 1	0	63	150	53
Phase 2	68	111	58	111
Phase 3	116	155	116	145

Movement	First Green Time Within Cycle Begin	First Green Time Within Cycle End	Second Green Time Within Cycle Begin	Second Green Time Within Cycle End	Common Green Time
EB EXT THRU	0	63			53
EB INT THRU	150	53	116	150	
WB EXT THRU	150	53			53
WB INT THRU	0	111			
SB RAMP	116	155			34
EB INT THRU	150	53	116	150	
NB RAMP	58	111			53
WB INT THRU	0	111			
WB INT LEFT	68	111			0
EB INT THRU	150	53			
EB INT LEFT	116	145			0
WB INT THRU	0	111			

The next step involves the calculation of lost time due to downstream queues. First, the queues at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. Exhibit 34-9 presents the calculation of these downstream queues followed by the calculation of the respective lost time due to those queues.

The lost time due to demand starvation is calculated by using Equation 22-16. The respective calculations are presented in Exhibit 34-10. As shown, in this case there is no lost time due to demand starvation (L_{DS} = 0).

Exhibit 34-9
Example Problem 1: Lost Time due to Downstream Queues

	EB EXT-TH	SB-L	WB EXT-TH	NB-L
V_R or V_A (veh/h)	206	868	233	886
N_R or N_A	1	2	1	2
G_R or G_A (s)	39	63	53	63
G_D (s)	97	97	111	111
C (s)	160	160	160	160
CG_{UD} or CG_{RD} (s)	53	34	53	53
Queue length (Q_A or Q_R) (ft)	0.0	4.1	0.0	0.0
	Lost Time due to Downstream Queue			
	EB EXT-TH	**SB-L**	**WB EXT-TH**	**NB-L**
G_R or G_A (s)	63	39	63	53
C (s)	160	160	160	160
D_{QA} or D_{QR} (ft)	500	496	500	500
CG_{UD} or CG_{RD} (s)	53	34	53	53
Additional lost time, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (s)	0.0	0.0	0.0	0.0
Total lost time, t'_L (s)	5.0	5.0	5.0	5.0
Effective green time, g' (s)	63.0	39.0	63.0	53.0

Exhibit 34-10
Example Problem 1: Lost Time due to Demand Starvation

	Lost Time due to Demand Starvation	
	EB- INT-TH	**WB- INT-TH**
$v_{\text{Ramp-}L}$ (veh/h)	206	233
v_{Arterial} (veh/h)	868	886
C (s)	160	160
$N_{\text{Ramp-}L}$	1	1
N_{Arterial}	2	2
CG_{RD} (s)	34	53
CG_{UD} (s)	53	53
H_I	2.02	2.04
Q_{initial} (ft)	0	0
CG_{DS} (s)	0	0
L_{DS} (s)	0	0
t''_L (s)	5	5
Effective green time, g'' (s)	97	111

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue to the available queue storage by using Equation 31-91. Exhibit 34-11 and Exhibit 34-12 present the calculations of the queue storage ratio for all movements in Example 1. Those exhibits also show the *v*/*c* ratio for each movement. Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-13 and Exhibit 34-14 provide the control delay for each movement of the interchange.

Exhibit 34-11
Example Problem 1: Queue Storage Ratio for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	957	107	967	1,036	236	883
s (veh/h/ln)	1,850	1,703	1,784	1,819	1,768	1,768
g (s)	63	29	97	63	43	111
g/C	0.39	0.18	0.61	0.39	0.27	0.69
I	1.00	0.71	0.71	1.00	0.62	0.62
c (veh/h/ln group)	1,459	309	2,163	1,437	475	2,452
$X = v/c$	0.66	0.35	0.45	0.72	0.50	0.36
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0	4.0	4.0	4.0
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.333	1.333	1.000	1.333	1.333
P	0.39	0.24	0.81	0.39	0.36	0.92
r (s)	97	131	63	97	117	49
t_f (s)	0.01	0.00	0.00	0.01	0.00	0.00
q (veh/s)	0.27	0.03	0.27	0.27	0.07	0.25
q_g (veh/s)	0.27	0.04	0.36	0.28	0.13	0.25
q_r (veh/s)	0.27	0.03	0.13	0.72	0.50	0.36
Q_1 (veh)	15.2	3.5	3.8	13.9	6.9	1.2
Q_2 (veh)	0.9	0.2	0.1	1.2	0.3	0.1
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00
t_A	0	0	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	16.2	3.7	4.0	15.2	7.2	1.3
L_h (ft)	25.01	25.00	25.01	25	25	25
L_a (ft)	600	200	500	600	200	500
R_Q	0.67	0.46	0.20	0.63	0.90	0.06

Exhibit 34-12
Example Problem 1: Queue Storage Ratio for Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0
v (veh/h/ln group)	233	227	206	173
s (veh/h/ln)	1,749	1,656	1,734	1,638
g (s)	53	53	39	39
g/C	0.33	0.33	0.24	0.24
I	1.00	1.00	1.00	1.00
c (veh/h/ln group)	580	549	423	399
$X = v/c$	0.40	0.41	0.49	0.43
r_a (ft/s^2)	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0	4.0
S_s (mi/h)	5	5	5	5
S_{pl} (mi/h)	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.000	1.000
P	0.33	0.33	0.24	0.24
r (s)	107.00	107.00	121.00	121.00
t_f (s)	0.00	0.00	0.00	0.00
q (veh/s)	0.06	0.06	0.06	0.05
q_g (veh/s)	0.06	0.06	0.06	0.05
q_r (veh/s)	0.06	0.06	0.06	0.05
Q_1 (veh)	7.1	6.9	7.1	5.9
Q_2 (veh)	0.3	0.3	0.5	0.4
T	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00
t_A	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00	0.00
Q_3 (veh)	0.0	0.0	0.0	0.0
Q (veh)	7.4	7.3	7.5	6.2
L_h (ft)	25	25	25	25
L_a (ft)	400	400	400	400
R_Q	0.46	0.45	0.47	0.39

Exhibit 34-13
Example Problem 1: Control Delay for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	**INT-L**	**INT-TH**	**EXT-TH&R**	**INT-L**	**INT-TH**
g (s)	-	29	97	-	43	111
g' (s)	63	-	-	63	-	-
g/C or g'/C	0.39	0.18	0.61	0.39	0.27	0.69
c (veh/h)	1,459	309	2,163	1,437	475	2,452
$X = v/c$	0.66	0.35	0.45	0.72	0.50	0.36
d_1 (s/veh)	39.6	52.8	7.3	31.3	42.9	2.0
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	4.6	2.2	0.5	6.2	2.3	0.3
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	0.560	1.000	1.000	0.283
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	44.1	55.0	7.8	37.5	45.2	2.3

Exhibit 34-14
Example Problem 1: Control Delay for Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	**Right**	**Left**	**Right**
g (s)	-	53	-	39
g' (s)	53	-	39	-
g/C or g'/C	0.33	0.33	0.24	0.24
c (veh/h)	580	549	423	399
$X = v/c$	0.42	0.41	0.49	0.43
d_1 (s/veh)	41.3	41.5	51.9	51.2
k	0.5	0.5	0.5	0.5
d_2 (s/veh)	2.1	2.1	4.0	3.4
d_3 (s/veh)	0.0	0.0	0.0	0.0
PF	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04
u	0	0	0	0
t	0	0	0	0
d (s/veh)	43.4	43.4	55.9	54.6

Results

This section discusses the estimation of the delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as the sum of the movement delays for each movement utilized by the O-D, as indicated in Equation 22-1. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement is F. Exhibit 34-15 summarizes the results for all O-D movements at this interchange. As shown, all the movements have v/c and queue storage ratios less than 1; for these O-D movements, the LOS is determined by using Exhibit 22-11.

Exhibit 34-15
Example Problem 1: O-D Movement LOS

O-D Movement	Control Delay (s)	*v/c* > 1?	R_Q > 1?	LOS
A	45.6	No	No	C
B	43.7	No	No	C
C	54.6	No	No	C
D	63.6	No	No	D
E	99.2	No	No	E
F	44.2	No	No	C
G	37.5	No	No	C
H	82.7	No	No	D
I	52.0	No	No	C
J	39.8	No	No	C
K	-	-	-	-
L	-	-	-	-
M	88.5	No	No	E
N	110.8	No	No	E

EXAMPLE PROBLEM 2: PARCLO A-2Q INTERCHANGE

The Interchange

The interchange of I-75 (NB/SB) and Newberry Avenue (EB/WB) is a Parclo A-2Q interchange. Exhibit 34-16 provides the interchange volumes and channelization, while Exhibit 34-17 provides the signalization information. The offset is referenced to the beginning of green on the EB direction of the arterial.

Exhibit 34-16
Example Problem 2: Intersection Plan View

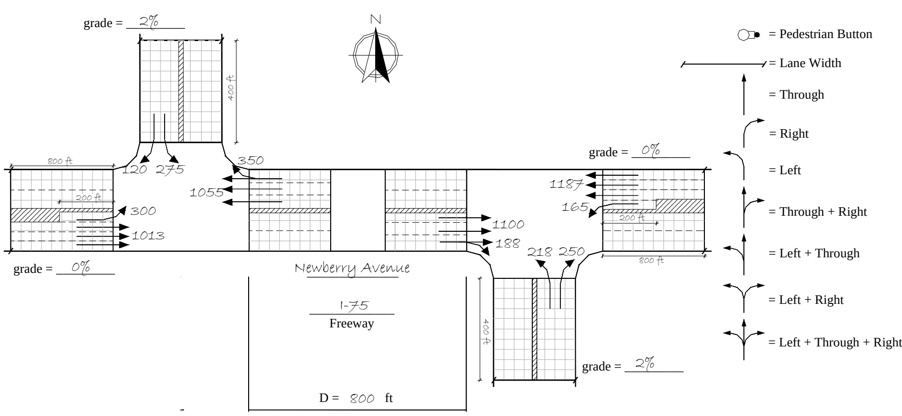


Exhibit 34-17
Example Problem 2: Signalization Information

	Intersection I			Intersection II		
Phase	**1**	**2**	**3**	**1**	**2**	**3**
NEMA	Φ (3+8)	Φ (4+8)	Φ (2+5)	Φ (4+7)	Φ (1+6)	Φ (4+8)
Green time	25	60	40	25	35	65
Yellow + all red	5	5	5	5	5	5
Offset		0			0	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. The eastbound and westbound left-turn radii are 80 ft, while all remaining turning movements have radii of 50 ft. The arrival type is assumed to be 4 for all arterial movements and 3 for all other movements.

There are 10% heavy vehicles on both the external and the internal through movements, and the PHF for the interchange is estimated to be 0.95. Start-up lost time is 3 s for all approaches, while the extension of effective green is 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of O-Ds

O-Ds through this parclo interchange are calculated on the basis of Exhibit 22-21(b). Since all movements utilize the signal, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the PHF-adjusted values are presented in Exhibit 34-18.

Exhibit 34-18
Example Problem 2: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	218	229
B	250	263
C	120	126
D	275	289
E	188	198
F	300	316
G	165	174
H	350	368
I	825	868
J	837	881
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

The external approaches to this interchange consist of a three-lane through lane group. Use of the three-lane model from Exhibit 22-17 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 34-19.

Exhibit 34-19
Example Problem 2: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	V_3	Maximum Lane Utilization	Lane Utilization Factor
Eastbound external	0.2660	0.2791	0.4549	0.4549	0.7328
Westbound external	0.2263	0.2472	0.5265	0.5265	0.6332

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-19. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through

Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated according to Chapter 18, Signalized Intersections. The results of these calculations for all approaches are presented in Exhibit 34-20 and Exhibit 34-21.

Exhibit 34-20
Example Problem 2: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

	Eastbound			Westbound		
	EXT-TH	EXT-L	INT-TH&R	EXT-TH	EXT-L	INT-TH&R
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	3	1	3	3	1	3
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.909	1.000	0.909	0.909	1.000	0.909
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.733	1.000	1.000	0.633	1.000	1.000
Left-turn adjustment (f_{LT})	1.000	0.934	1.000	1.000	0.934	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	0.998	1.000	1.000	0.994
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	1.000	0.934	0.985	1.000	0.934	0.975
Traffic pressure adjustment for lane group (f_v)	0.997	1.013	1.016	1.009	0.976	1.024
Adjusted saturation flow (s, veh/hg/ln)	3,786	1,798	5,253	3,310	1,733	5,271

Exhibit 34-21
Example Problem 2: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

	Northbound		Southbound	
	Left	Right	Left	Right
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	1	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	1.000	1.000	1.000	1.000
Grade adjustment (f_g)	0.990	0.990	0.990	0.990
Parking adjustment (f_p)	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	0.899	1.000	0.899	1.000
Right-turn adjustment (f_{RT})	1.000	0.899	1.000	0.899
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.899	0.899	0.899	0.899
Traffic pressure adjustment for lane group (f_v)	0.990	0.980	1.006	0.956
Adjusted saturation flow (s, veh/hg/ln)	1,674	1,658	1,701	1,617

Common Green and Lost Time due to Downstream Queue and Demand Starvation Calculations

Exhibit 34-22 provides the beginning and end times of the green for each phase followed by the beginning and end of green for the six pairs of movements at the two intersections. It is assumed that time zero is when Phase 1 of the first intersection begins (in this case the offset for both intersections is zero, and therefore the beginning of Phase 1 for the second intersection is also zero).

Exhibit 34-22
Example Problem 2: Common Green Calculations

Phase	Intersection I Green Begin	Intersection I Green End	Intersection II Green Begin	Intersection II Green End	
Phase 1	0	25	0	25	
Phase 2	30	90	30	65	
Phase 3	95	135	70	135	
Movement	**First Green Time Within Cycle Begin**	**First Green Time Within Cycle End**	**Second Green Time Within Cycle Begin**	**Second Green Time Within Cycle End**	**Common Green Time**
EB EXT THRU	0	90			20
EB INT THRU	70	135			
WB EXT THRU	0	25	70	135	20
WB INT THRU	30	90			
SB RAMP	95	135			40
EB INT THRU	70	135			
NB RAMP	30	65			35
WB INT THRU	30	90			

The next step involves the calculation of lost time due to downstream queues. First, the queues at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. Exhibit 34-23 presents the calculation of these downstream queues followed by the calculation of the respective lost time due to those queues.

Exhibit 34-23
Example Problem 2: Lost Time due to Downstream Queues

	EB EXT-TH	SB-L	WB EXT-TH	NB-L
V_R or V_A (veh/h)	289	1,066	229	1,249
N_R or N_A	1	3	1	3
G_R or G_A (s)	40	90	35	95
G_D (s)	65	65	60	60
C (s)	140	140	140	140
CG_{UD} or CG_{RD} (s)	20	40	20	35
Queue length (Q_A or Q_R) (ft)	0.9	48.6	0.0	89.4
	Lost Time due to Downstream Queue			
	EB EXT-TH	**SB-L**	**WB EXT-TH**	**NB-L**
G_R or G_A (s)	90	40	95	35
C (s)	140	140	140	140
D_{QA} or D_{QR} (ft)	799	751	800	711
CG_{UD} or CG_{RD} (s)	20	40	20	35
Additional lost time, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (s)	0	0	0	0
Total lost time, t'_L (s)	6	6	6	6
Effective green time, g' (s)	89	39	94	34

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue to the available queue storage by using Equation 31-91. Exhibit 34-24 and Exhibit 34-25 present the calculation of the queue storage ratio for all movements in Example Problem 2. The exhibit also shows the *v/c* ratio for each movement. Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-26 and Exhibit 34-27 provide the control delay for each movement of this interchange.

Exhibit 34-24
Example Problem 2: Queue Storage Ratio for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH	EXT-L	INT-TH&R	EXT-TH	EXT-L	INT-TH&R
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	1,066	316	1,282	1,249	174	1,479
s (veh/h/ln)	1,262	1,798	1,751	1,103	1,733	1,757
g (s)	89	24	64	94	24	59
g/C	0.64	0.17	0.46	0.67	0.17	0.42
I	1.00	1.00	0.90	1.00	1.00	0.81
c (veh/h/ln group)	2,407	308	2,401	2,222	297	2,221
$X = v/c$	0.44	1.02	0.54	0.56	0.58	0.67
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0	4.0	4.0	4.0
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.333	1.000	1.000	1.333
P	0.636	0.171	0.609	0.671	0.171	0.562
r (s)	51	116	76	46	116	81
t_f (s)	0.00	0.01	0.00	0.01	0.00	0.01
q (veh/s)	0.30	0.09	0.38	0.35	0.05	0.41
q_g (veh/s)	0.30	0.09	0.50	0.35	0.05	0.55
q_r (veh/s)	0.30	0.09	0.27	0.35	0.05	0.31
Q_1 (veh)	5.4	10.7	6.9	6.3	5.6	10.4
Q_2 (veh)	0.1	4.9	0.3	0.2	0.7	0.5
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00
t_A	0	0	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	5.5	15.7	7.2	6.5	6.3	10.9
L_h (ft)	25.02	25.00	25.02	25.02	25.00	25.02
L_a (ft)	800	200	800	800	200	800
R_Q	0.17	1.96	0.23	0.20	0.78	0.34

Exhibit 34-25
Example Problem 2: Queue Storage Ratio for Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0
v (veh/h/ln group)	229	263	289	126
s (veh/h/ln)	1,674	1,658	1,701	1,617
g (s)	34	34	39	39
g/C	0.24	0.24	0.28	0.28
I	1.00	1.00	1.00	1.00
c (veh/h/ln group)	407	403	474	450
$X = v/c$	0.56	0.65	0.61	0.28
r_a (ft/s^2)	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0	4.0
S_s (mi/h)	5	5	5	5
S_{pl} (mi/h)	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.000	1.000
P	0.243	0.243	0.279	0.279
r (s)	106	106	101	101
t_f (s)	0.00	0.00	0.01	0.00
q (veh/s)	0.06	0.07	0.08	0.04
q_g (veh/s)	0.06	0.07	0.08	0.04
q_r (veh/s)	0.06	0.07	0.08	0.04
Q_1 (veh)	7.8	9.2	9.8	3.4
Q_2 (veh)	0.6	0.9	0.8	0.2
T	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00
t_A	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00	0.00
Q_3 (veh)	0.0	0.0	0.0	0.0
Q (veh)	8.5	10.1	10.5	3.6
L_h (ft)	25	25	25	25
L_a (ft)	400	400	400	400
R_Q	0.53	0.63	0.66	0.22

	Eastbound Movements			Westbound Movements		
	EXT-TH	EXT-L	INT-TH&R	EXT-TH	EXT-L	INT-TH&R
g (s)	-	24	64	-	24	59
g' (s)	89	-	-	94	-	-
g/C or g'/C	0.64	0.17	0.46	0.67	0.17	0.42
c (veh/h)	2,407	308	2,401	2,222	297	2,221
$X = v/c$	0.44	1.02	0.56	0.56	0.58	0.67
d_1 (s/veh)	12.9	58.0	18.8	12.1	53.4	24.1
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	0.6	57.7	1.5	1.0	8.2	2.6
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	0.827	1.000	1.000	0.871
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	13.5	115.7	20.3	13.2	61.6	26.8

Exhibit 34-26
Example Problem 2: Control Delay for Eastbound and Westbound Movements

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
g (s)	-	34	39	-
g' (s)	34	-	-	39
g/C or g'/C	0.24	0.24	0.28	0.28
c (veh/h)	407	403	474	450
$X = v/c$	0.56	0.65	0.61	0.28
d_1 (s/veh)	46.5	47.7	43.9	39.5
k	0.5	0.5	0.5	0.5
d_2 (s/veh)	5.6	8.0	5.8	1.6
d_3 (s/veh)	0.0	0.0	0.0	0.0
PF	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04
u	0	0	0	0
t	0	0	0	0
d (s/veh)	52.1	55.7	49.7	41.1

Exhibit 34-27
Example Problem 2: Control Delay for Northbound and Southbound Movements

Results

Delay for each O-D is estimated as the sum of the movement delays for each movement utilized by the O-D, as indicated in Equation 22-1. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement is F. Exhibit 34-28 presents the resulting delay, v/c ratio, and R_Q for each O-D movement. As shown, O-D Movement F (which consists of the EB external left movement) has v/c and R_Q ratios greater than 1, resulting in LOS F. For the remaining movements, the LOS is determined by using Exhibit 22-11.

O-D Movement	Control Delay (s)	v/c > 1?	R_Q > 1?	LOS
A	78.9	No	No	D
B	55.7	No	No	D
C	41.1	No	No	C
D	70.0	No	No	D
E	33.8	No	No	C
F	115.7	Yes	Yes	F
G	61.6	No	No	D
H	40.0	No	No	C
I	33.8	No	No	C
J	40.0	No	No	C
K	-	-	-	-
L	-	-	-	-
M	78.9	No	No	D
N	70.0	No	No	D

Exhibit 34-28
Example Problem 2: O-D Movement LOS

EXAMPLE PROBLEM 3: DIAMOND INTERCHANGE WITH QUEUE SPILLBACK

The Interchange

The interchange of I-95 (NB/SB) and 22nd Avenue (EB/WB) is a diamond interchange. The traffic, geometric, and signalization conditions for this study site are provided in Exhibit 34-29 and Exhibit 34-30. The offset is referenced to the beginning of green on the EB direction of the arterial.

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 50 ft for all turning movements except the eastbound and westbound left movements, which are 75 ft.

There are 5% heavy vehicles on both the external and the internal through movements, and the PHF for the interchange is 0.97. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Exhibit 34-29
Example Problem 3:
Intersection Plan View

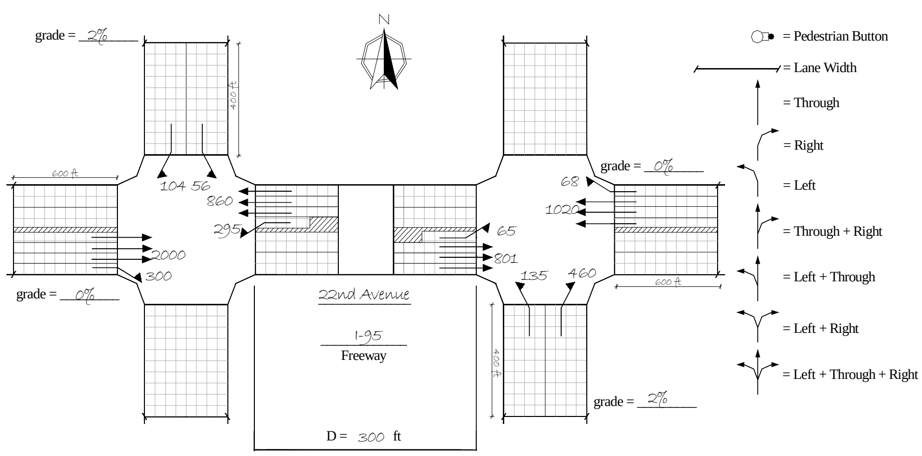


Exhibit 34-30
Example Problem 3:
Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (2+5)	Φ (4+8)	Φ (4+7)	Φ (3+8)	Φ (4+8)	Φ (1+6)
Green time	27	59	19	27	39	39
Yellow + all red	5	5	5	5	5	5
Offset		0			0	

Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). Since all movements utilize the signal, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 34-31.

Exhibit 34-31
Example Problem 3: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	135	139
B	460	474
C	104	107
D	56	58
E	1,255	1,294
F	300	309
G	68	70
H	295	304
I	745	768
J	725	747
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

This interchange consists of external approaches with three through lanes and an exclusive right-turn lane. The lane utilization for Lane 1 is predicted by using the three-lane model of Exhibit 22-16. Since there is an exclusive right-turn lane for both external approaches, according to the second note of Exhibit 22-16 the lane utilization for Lane 3 should be estimated by assuming that the right-turning O-D (v_F, v_G) is zero. Exhibit 34-32 presents the calculation results and the lane utilization factor for each approach.

Exhibit 34-32
Example Problem 3: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	V_3	Maximum Lane Utilization	Lane Utilization Factor
3-lane EB	0.5551	0.2224	0.2224	0.5551	0.6005
3-lane WB	0.4441	0.2779	0.2779	0.4441	0.7506

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hr/ln by using Equation 22-3. The lane utilization of the external through movements is obtained as shown above in Exhibit 34-32. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated as indicated in Chapter 18, Signalized Intersections. The results of these calculations for all approaches are presented in Exhibit 34-33 and Exhibit 34-34.

Exhibit 34-33
Example Problem 3:
Saturation Flow Rate
Calculation for Eastbound
and Westbound Approaches

	Eastbound				Westbound			
	EXT-TH	**EXT-R**	**INT-TH**	**INT-L**	**EXT-TH**	**EXT-R**	**INT-TH**	**INT-L**
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	3	1	3	1	3	1	3	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	1.000	0.952	1.000	0.952	1.000	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.600	1.000	0.908	1.000	0.751	1.000	0.908	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	1.000	0.930	1.000	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	1.000	0.899	1.000	1.000	1.000	0.899	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	1.000	0.899	1.000	0.930	1.000	0.899	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	1.043	0.980	0.975	0.948	0.987	0.945	0.978	0.998
Adjusted saturation flow (s, veh/hg/ln)	3,400	1,675	4,807	1,676	4,021	1,614	4,822	1,764

Exhibit 34-34
Example Problem 3:
Saturation Flow Rate
Calculation for Northbound
and Southbound Approaches

	Northbound		Southbound	
	Left	**Right**	**Left**	**Right**
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	1.000	1.000
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	1.000	1.000	1.000	1.000
Grade adjustment (f_g)	0.990	0.990	0.990	0.990
Parking adjustment (f_p)	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	0.899	1.000	0.899	1.000
Right-turn adjustment (f_{RT})	1.000	0.899	1.000	0.899
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.899	0.899	0.899	0.899
Traffic pressure adjustment for lane group (f_v)	0.963	1.007	0.946	0.950
Adjusted saturation flow (s, veh/hg/ln)	1,628	1,703	1,600	1,606

Common Green and Lost Time due to Downstream Queue and Demand Starvation Calculations

Exhibit 34-35 first provides the beginning and ending of the green time for each phase at the two intersections, assuming that time zero is when Phase 1 of the first intersection begins. In this case, the offset for both intersections is zero; therefore the beginning of Phase 1 for the second intersection is also zero.

Phase	Intersection I Green Begin	Intersection I Green End	Intersection II Green Begin	Intersection II Green End	
Phase 1	0	27	0	27	
Phase 2	32	91	32	71	
Phase 3	96	115	76	115	
Movement	**First Green Time Within Cycle Begin**	**First Green Time Within Cycle End**	**Second Green Time Within Cycle Begin**	**Second Green Time Within Cycle End**	**Common Green Time**
EB EXT THRU	32.0	91.0			39
EB INT THRU	0.0	71.0			
WB EXT THRU	32.0	71.0			39
WB INT THRU	32.0	115.0			
SB RAMP	0.0	27.0			27
EB INT THRU	0.0	71.0			
NB RAMP	76.0	115.0			39
WB INT THRU	32.0	115.0			
WB INT LEFT	96.0	115.0			0
EB INT THRU	0.0	71.0			
EB INT LEFT	0.0	27.0			0
WB INT THRU	32.0	115.0			

Exhibit 34-35
Example Problem 3: Common Green Calculations

The next step involves the calculation of lost time due to downstream queues. First, the queues at the beginning of the upstream arterial phase and the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. Exhibit 34-36 presents the calculation of these downstream queues followed by the calculation of the respective lost time due to those queues. As shown, the SB-L movement has additional lost time of 5.5 s due to the downstream queue.

The lost time due to demand starvation is calculated by using Equation 22-16. The respective calculations are presented in Exhibit 34-37. As shown, in this case there is no lost time due to demand starvation.

	EB EXT-TH	SB-L	WB EXT-TH	NB-L
V_R or V_A (veh/h)	58	2,062	139	1,052
N_R or N_A	1	3	1	3
G_R or G_A (s)	27	59	39	39
G_D (s)	71	71	83	83
C (s)	120	120	120	120
CG_{UD} or CG_{RD} (s)	39.0	27.0	39.0	39.0
Queue length (Q_A or Q_R) (ft)	0.0	108.60	0.0	0.0
	Lost Time due to Downstream Queue			
	EB EXT-TH	**SB-L**	**WB EXT-TH**	**NB-L**
G_R or G_A (s)	59	27	39	39
C (s)	120	120	120	120
D_{QA} or D_{QR} (ft)	300	191	300	300
CG_{UD} or CG_{RD} (s)	39.0	27	39	39
Additional lost time, L_{D-A} or L_{D-R} (s)	0.0	5.5	0.0	0.0
Total lost time, t'_L (s)	5.0	10.5	5.0	5.0
Effective green time, g' (s)	59.0	21.5	39.0	39.0

Exhibit 34-36
Example Problem 3: Lost Time due to Downstream Queues

Exhibit 34-37
Example Problem 3: Lost Time due to Demand Starvation Calculations

	EB- INT-TH	WB- INT-TH
$v_{\text{Ramp-}L}$ (veh/h)	58	139
v_{Arterial} (veh/h)	2062	1052
C (s)	120	120
$N_{\text{Ramp-}L}$	1	1
N_{Arterial}	3	3
CG_{RD} (s)	27	39
CG_{UD} (s)	39	39
H_I	2.25	2.24
Q_{initial} (ft)	0	0
CG_{DS} (s)	0	0
L_{DS} (s)	0	0
t''_L (s)	5	5
Effective green time, g'' (s)	71	83

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue to the available queue storage by using Equation 31-91. Exhibit 34-38 and Exhibit 34-39 present the calculations of the queue storage ratio for all movements. Those exhibits also provide the *v/c* ratio for each movement. Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-40 and Exhibit 34-41 provide the control delay for each movement of the interchange.

Exhibit 34-38
Example Problem 3: Queue Storage Ratio for Eastbound and Westbound Movements

	Eastbound Movements				Westbound Movements			
	EXT-TH	EXT-R	INT-L	INT-TH	EXT-TH	EXT-R	INT-L	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	2,062	309	67	826	1,052	70	304	887
s (veh/h/ln)	1,133	1,675	1,676	1,602	1,340	1,614	1,764	1,607
g (s)	59.0	59.0	27.0	71.0	39.0	39.0	19.0	83.0
g/C	0.49	0.49	0.23	0.59	0.33	0.33	0.16	0.69
I	1.00	1.00	0.09	0.09	1.00	1.00	0.49	0.49
c (veh/h/ln group)	1,672	824	377	2,844	1,307	524	279	3,336
$X = v/c$	1.23	0.38	0.18	0.29	0.80	0.13	1.09	0.27
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1	1	1	1.333	1	1	1	1.333
P	0.49	0.49	0.23	0.79	0.33	0.33	0.16	0.92
r (s)	61.00	61.00	93.00	49.00	81.00	81.00	101.00	37.00
t_r (s)	0.02	0.00	0.00	0.00	0.01	0.00	0.01	0.00
q (veh/s)	0.57	0.09	0.02	0.23	0.29	0.02	0.08	0.25
q_g (veh/s)	0.57	0.09	0.02	0.31	0.29	0.02	0.08	0.33
q_r (veh/s)	0.57	0.09	0.02	0.12	0.29	0.02	0.08	0.06
Q_1 (veh)	14.9	5.2	1.6	1.6	9.1	1.4	8.2	0.5
Q_2 (veh)	17.1	0.3	0.0	0.0	0.6	0.1	5.0	0.1
T	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	97.50	0.00	0.00	0.00	0.00	0.00	6.22	0.00
t_A	0.25	0	0	0	0	0	0.25	0
Q_e (veh)	97.50	0.00	0.00	0.00	0.00	0.00	6.22	0.00
Q_b (veh)	0	0	0	0	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	32.0	5.5	1.6	1.6	9.7	1.5	13.2	0.6
L_h (ft)	25	25	25	25	25	25	25	25
L_a (ft)	600	600	200	300	600	600	200	300
R_Q	1.33	0.23	0.20	0.13	0.41	0.06	1.65	0.12

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0
v (veh/h/ln group)	139	474	58	107
s (veh/h/ln)	1,628	1,703	1,600	1,607
g (s)	39.0	39.0	22.0	27.0
g/C	0.33	0.33	0.18	0.23
I	1.00	1.00	1.00	1.00
c (veh/h/ln group)	529	553	287	362
$X = v/c$	0.26	0.86	0.20	0.30
r_a (ft/s^2)	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4
S_s (mi/h)	5	5	5	5
S_{pl} (mi/h)	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04
Rp	1	1	1	1
P	0.33	0.33	0.18	0.23
r (s)	81.00	81.00	98.50	93.00
t_f (s)	0.00	0.01	0.00	0.00
q (veh/s)	0.04	0.13	0.02	0.03
q_g (veh/s)	0.04	0.13	0.02	0.03
q_r (veh/s)	0.04	0.13	0.02	0.03
Q_1 (veh)	2.9	12.6	1.4	2.6
Q_2 (veh)	0.2	2.4	0.1	0.2
T	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.0	0.0	0.0	0.0
t_A	0.0	0.0	0.0	0.0
Q_e (veh)	0.0	0.0	0.0	0.0
Q_b (veh)	0.0	0.0	0.0	0.0
Q_3 (veh)	0.0	0.0	0.0	0.0
Q (veh)	3.1	15.0	1.6	2.8
L_h (ft)	25	25	25	25
L_a (ft)	400	400	400	400
R_Q	0.19	0.94	0.10	0.17

Exhibit 34-39
Example Problem 3: Queue Storage Ratio for Northbound and Southbound Movements

	Eastbound Movements				Westbound Movements			
	EXT-TH	EXT-R	INT-L	INT-TH	EXT-TH	EXT-R	INT-L	INT-TH
g (s)	-	59	27	71	-	39	19	83
g' (s)	59	-	-	-	39	-	-	-
g/C or g'/C	0.49	0.49	0.23	0.59	0.33	0.33	0.16	0.69
c (veh/h)	1,672	824	377	2,844	1,307	524	279	3,336
$X = v/c$	1.23	0.38	0.18	0.29	0.80	0.13	1.09	0.27
d_1 (s/veh)	30.5	19.0	37.5	5.8	37.0	28.6	50.5	1.5
k	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	110.5	1.3	0.1	0.0	5.4	0.5	64.1	0.1
d_3 (s/veh)	0	0	0	0	0	0	0	0
PF	1.000	1.000	1.000	0.595	1.000	1.000	1.000	0.291
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0	0	0
t	0	0	0	0	0	0	0	0
d (s/veh)	141.0	20.3	37.6	5.8	42.4	29.1	114.6	1.6

Exhibit 34-40
Example Problem 3: Control Delay for Eastbound and Westbound Movements

Exhibit 34-41
Example Problem 3: Control Delay for Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	**Right**	**Left**	**Right**
g (s)	-	39	-	27
g' (s)	39	-	21.5	-
g/C or g'/C	0.33	0.33	0.18	0.23
c (veh/h)	529	553	287	361
$X = v/c$	0.26	0.86	0.20	0.30
d_1 (s/veh)	29.9	37.9	41.9	38.6
k	0.5	0.5	0.5	0.5
d_2 (s/veh)	1.2	15.7	1.6	2.1
d_3 (s/veh)	0.0	0.0	0.0	0.0
PF	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04
u	0	0	0	0
t	0	0	0	0
d (s/veh)	31.1	53.6	43.5	40.7

Results

Delay for each O-D is estimated as the sum of the movement delays for each movement utilized by the O-D, as indicated in Equation 22-1. Next, the v/c ratio and queue storage ratio are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement is F. Exhibit 34-42 presents a summary of the results for all O-D movements at this interchange. As shown, v/c and R_Q for parts of O-Ds E, H, I, and M exceed 1; therefore, these O-Ds operate in LOS F. O-D E and O-D I include the EB external through movement, while O-D H and O-D M include the WB internal left. These movements have v/c ratios exceeding 1. The remaining movements have v/c and queue storage ratios less than 1; the LOS for these O-D movements is determined by using Exhibit 22-11.

Exhibit 34-42
Example Problem 3: O-D Movement LOS

O-D Movement	Control Delay (s)	v/c > 1?	R_Q > 1?	LOS
A	32.7	No	No	C
B	53.6	No	No	C
C	40.7	No	No	C
D	49.3	No	No	C
E	178.6	Yes	Yes	F
F	20.3	No	No	B
G	29.1	No	No	B
H	157.0	Yes	Yes	F
I	146.8	Yes	Yes	F
J	44.0	No	No	C
K	-	-	-	-
L	-	-	-	-
M	145.7	Yes	Yes	F
N	81.1	No	No	D

EXAMPLE PROBLEM 4: DIAMOND INTERCHANGE WITH DEMAND STARVATION

The Interchange

The interchange of I-75 (NB/SB) and Archer Road (EB/WB) is a diamond interchange. The traffic, geometric, and signalization conditions for this interchange are provided in Exhibit 34-43 and Exhibit 34-44. The offset is referenced to the beginning of green on the EB direction of the arterial.

Exhibit 34-43
Example Problem 4: Intersection Plan View

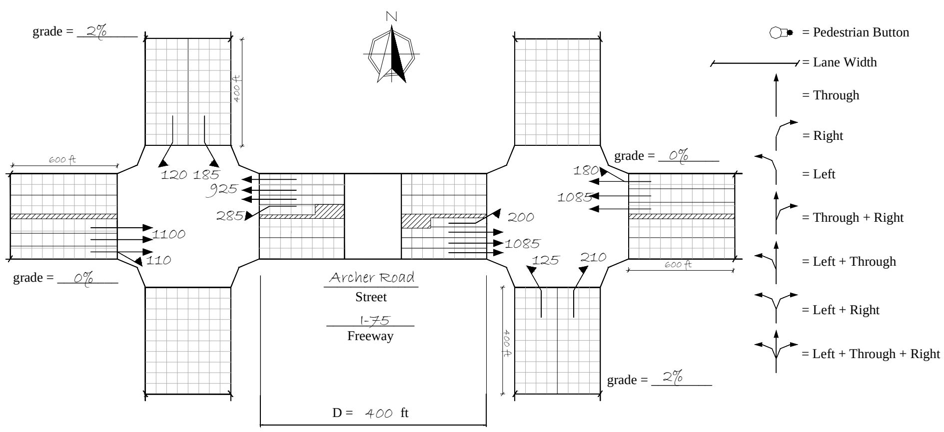


Exhibit 34-44
Example Problem 4: Signalization Information

	Intersection I			Intersection II		
Phase	**1**	**2**	**3**	**1**	**2**	**3**
NEMA	Φ (4+7)	Φ (4+8)	Φ (2+5)	Φ (4+8)	Φ (3+8)	Φ (1+6)
Green time	30	25	30	30	25	30
Yellow + all red	5	5	5	5	5	5
Offset		0			0	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 50 ft for all turning movements except the eastbound and westbound left, which are 75 ft.

There are 5% heavy vehicles on both external and internal through movements, and the PHF for the interchange is estimated to be 0.97. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis interval there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). Since all movements utilize the signal, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these O-D calculations and the resulting PHF-adjusted values are presented in Exhibit 34-45.

Exhibit 34-45
Example Problem 4:
Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	125	129
B	210	216
C	120	124
D	185	191
E	200	206
F	110	113
G	180	186
H	285	294
I	900	928
J	800	825
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

This interchange consists of a three-lane shared right and through lane group for the external approaches. Use of the three-lane model from Exhibit 22-16 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 34-46.

Exhibit 34-46
Example Problem 4: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	V_3	Maximum Lane Utilization	Lane Utilization Factor
3-lane EB	0.3879	0.2773	0.3348	0.3879	0.8593
3-lane WB	0.4032	0.2502	0.3465	0.4032	0.8266

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hr/ln by using Equation 22-3. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-46. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equations 22-6 through 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated as indicated in Chapter 18, Signalized Intersections. The results of the saturation flow rate calculations for all approaches are presented in Exhibit 34-47 and Exhibit 34-48.

	Eastbound			Westbound		
	EXT-TH&R	INT-TH	INT-L	EXT-TH&R	INT-TH	INT-L
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	3	3	1	3	3	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	1.000	0.952	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.859	0.908	1.000	0.827	0.908	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.930	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	0.999	1.000	1.000	0.998	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.991	1.000	0.930	0.986	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	0.986	0.981	0.969	0.989	0.974	0.985
Adjusted saturation flow (s, veh/hg/ln)	4,597	4,834	1,714	4,428	4,799	1,741

Exhibit 34-47
Example Problem 4: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

	Northbound		Southbound	
	Left	Right	Left	Right
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	1	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	1.000	1.000	1.000	1.000
Grade adjustment (f_g)	0.990	0.990	0.990	0.990
Parking adjustment (f_p)	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	0.899	1.000	0.899	1.000
Right-turn adjustment (f_{RT})	1.000	0.899	1.000	0.899
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.899	0.899	0.899	0.899
Traffic pressure adjustment for lane group (f_v)	0.956	0.961	0.967	0.949
Adjusted saturation flow (s, veh/hg/ln)	1,617	1,625	1,635	1,605

Exhibit 34-48
Example Problem 4: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

Common Green and Lost Time due to Downstream Queue and Demand Starvation Calculations

Exhibit 34-49 presents the beginning and end times of the green for each phase at the two intersections. It is assumed that time zero is when Phase 1 of the first intersection begins. In this case the offset for both intersections is zero; therefore the beginning of Phase 1 for the second intersection is also zero.

Exhibit 34-49
Example Problem 4:
Common Green Calculations

Phase	Intersection I Green Begin	Intersection I Green End	Intersection II Green Begin	Intersection II Green End	
Phase 1	0	30	0	30	
Phase 2	35	60	35	60	
Phase 3	65	95	65	95	

Movement	First Green Time Within Cycle Begin	First Green Time Within Cycle End	Second Green Time Within Cycle Begin	Second Green Time Within Cycle End	Common Green Time
EB EXT THRU	35	60			25
EB INT THRU	0	60			
WB EXT THRU	0	30			30
WB INT THRU	0	60			
SB RAMP	65	95			0
EB INT THRU	35	60			
NB RAMP	65	95			0
WB INT THRU	0	60			
WB INT LEFT	0	30			30
EB INT THRU	0	60			
EB INT LEFT	35	60			25
WB INT THRU	0	60			

The next step involves the calculation of lost time due to downstream queues. First, the queues at the beginning of the upstream arterial phase and the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. Exhibit 34-50 presents the calculation of these downstream queues followed by the calculation of the respective lost time due to those queues. As shown, there is no additional lost time due to downstream queues.

The lost time due to demand starvation is calculated by using Equation 22-16. The respective calculations are presented in Exhibit 34-51. As shown, both internal through movements experience lost time due to demand starvation.

Exhibit 34-50
Example Problem 4: Lost Time due to Downstream Queues

	EB EXT-TH	SB-L	WB EXT-TH	NB-L
V_R or V_A (veh/h)	191	1134	129	1119
N_R or N_A	1	3	1	3
G_R or G_A (s)	30	25	30	30
G_D (s)	60	60	60	60
C (s)	100	100	100	100
CG_{UD} or CG_{RD} (s)	25	0	30	0
Queue length (Q_A or Q_R) (ft)	0.0	31.5	0.0	40
	Lost Time due to Downstream Queue			
	EB EXT-TH	**SB-L**	**WB EXT-TH**	**NB-L**
G_R or G_A (s)	25	30	30	30
C (s)	100	100	100	100
D_{QA} or D_{QR} (ft)	400	369	400	360
CG_{UD} or CG_{RD} (s)	25	0	30	0
Additional lost time, L_{D-A} or L_{D-R} (s)	0	0	0	0
Total lost time, t'_L (s)	5	5	5	5
Effective green time, g' (s)	25	30	30	30

Exhibit 34-51
Example Problem 4: Lost Time due to Demand Starvation Calculations

	EB- INT-TH	WB- INT-TH
v_{Ramp-L} (veh/h)	191	129
$v_{Arterial}$ (veh/h)	1134	1119
C (s)	100	100
N_{Ramp-L}	1	1
$N_{Arterial}$	3	3
CG_{RD} (s)	5	5
CG_{UD} (s)	25	30
H_I	2.23	2.25
$Q_{initial}$ (ft)	6.8	2.8
CG_{DS} (s)	30	25
L_{DS} (s)	14.7	18.6
t''_L (s)	19.7	23.6
Effective green time, g'' (s)	45.3	41.4

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue to the available queue storage by using Equation 31-91. Exhibit 34-52 and Exhibit 34-53 present the calculations of the queue storage ratio for all movements. These exhibits also provide the *v*/c ratios for all movements. Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-54 and Exhibit 34-55 provide the control delay for each movement of the interchange.

Exhibit 34-52
Example Problem 4: Queue Storage Ratio for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	1,247	206	1,119	1,304	294	954
s (veh/h/ln)	1,532	1,714	1,611	1,476	1,741	1,600
g (s)	25	25	45	30	30	41
g/C	0.25	0.25	0.45	0.30	0.30	0.41
I	1.00	0.09	0.09	1.00	0.13	0.13
c (veh/h/ln group)	1,198	428	2,190	1,383	522	1,987
$X = v/c$	1.04	0.48	0.51	0.94	0.56	0.48
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.333	1.000	1.000	1.333
P	0.25	0.25	0.60	0.30	0.30	0.55
r (s)	75.00	75.00	54.71	70.00	70.00	58.64
t_r (s)	0.01	0.00	0.00	0.01	0.00	0.00
q (veh/s)	0.35	0.06	0.31	0.36	0.08	0.26
q_g (veh/s)	0.35	0.06	0.41	0.36	0.08	0.35
q_r (veh/s)	0.35	0.06	0.23	0.36	0.08	0.20
Q_1 (veh)	9.2	4.1	3.8	9.8	5.7	3.7
Q_2 (veh)	5.5	0.0	0.0	3.0	0.1	0.0
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	24.54	0.00	0.00	0.00	0.00	0.00
t_A	0.25	0	0	0	0	0
Q_e (veh)	24.54	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	14.7	4.1	3.9	12.8	5.8	3.7
L_h (ft)	25	25	25	25	25	25
L_a (ft)	600	200	400	600	200	400
R_Q	0.61	0.52	0.24	0.53	0.72	0.23

Exhibit 34-53
Example Problem 4: Queue Storage Ratio for Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0
v (veh/h/ln group)	129	216	191	124
s (veh/h/ln)	1,617	1,625	1,635	1,606
g (s)	30	30	30	30
g/C	0.30	0.30	0.30	0.30
I	1.00	1.00	1.00	1.00
c (veh/h/ln group)	485	487	491	482
$X = v/c$	0.27	0.44	0.39	0.26
r_a (ft/s^2)	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4
S_s (mi/h)	5	5	5	5
S_{pl} (mi/h)	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04
Rp	1.00	1.00	1.00	1.00
P	0.30	0.30	0.30	0.30
r (s)	70.00	70.00	70.00	70.00
t_f (s)	0.00	0.00	0.00	0.00
q (veh/s)	0.04	0.06	0.05	0.03
q_g (veh/s)	0.04	0.06	0.05	0.03
q_r (veh/s)	0.04	0.06	0.05	0.03
Q_1 (veh)	2.3	4.0	3.5	2.2
Q_2 (veh)	0.2	0.4	0.3	0.2
T	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00
t_A	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0
Q (veh)	2.4	4.4	3.8	2.3
L_h (ft)	25	25	25	25
L_a (ft)	400	400	400	400
R_Q	0.15	0.28	0.24	0.15

Exhibit 34-54
Example Problem 4: Control Delay for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
g (s)	-	25	45	-	30	41
g' (s)	25	-	-	30	-	-
g/C or g'/C	0.25	0.25	0.45	0.30	0.30	0.41
c (veh/h)	1,198	428	2,190	1,385	522	1,985
$X = v/c$	1.04	0.48	0.51	0.94	0.56	0.48
d_1 (s/veh)	37.4	32.0	13.5	34.2	29.5	15.8
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	50.3	0.3	0.1	23.7	0.6	0.1
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	0.863	1.000	1.000	0.902
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	87.6	32.3	13.5	57.9	30.1	16.0

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
g (s)	-	30	-	30
g' (s)	30	-	30	-
g/C or g'/C	0.30	0.30	0.30	0.30
c (veh/h)	485	487	490	482
$X = v/c$	0.27	0.44	0.39	0.26
d_1 (s/veh)	26.6	28.3	27.7	26.5
k	0.5	0.5	0.5	0.5
d_2 (s/veh)	1.3	2.9	2.3	1.3
d_3 (s/veh)	0.0	0.0	0.0	0.0
PF	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04
u	0	0	0	0
t	0	0	0	0
d (s/veh)	28.0	31.2	30.1	27.8

Exhibit 34-55
Example Problem 4: Control Delay for Northbound and Southbound Movements

Results

Delay for each O-D is estimated as the sum of the movement delays for each movement utilized by the O-D, as indicated in Equation 22-1. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement is F. Exhibit 34-56 summarizes the results for all O-D movements at this interchange. As shown, the v/c ratio exceeds 1 for O-D Movements E, F, and I, all of which include the EB external through and right movements. Therefore, these O-D movements operate in LOS F. The remaining movements have v/c and queue storage ratios less than 1; the LOS is determined by using Exhibit 22-11 for these movements.

O-D Movement	Control Delay (s)	v/c > 1?	R_Q > 1?	LOS
A	43.9	No	No	C
B	31.2	No	No	C
C	27.8	No	No	B
D	43.6	No	No	C
E	119.9	Yes	No	F
F	87.6	Yes	No	F
G	57.9	No	No	D
H	88.0	No	No	E
I	101.1	Yes	No	F
J	73.9	No	No	D
K	-	-	-	-
L	-	-	-	-
M	58.0	No	No	D
N	62.4	No	No	D

Exhibit 34-56
Example Problem 4: O-D Movement LOS

EXAMPLE PROBLEM 5: SINGLE-POINT URBAN INTERCHANGE

The Interchange

The interchange of I-95 (NB/SB) and University Drive (EB/WB) is a single-point urban interchange (SPUI). The traffic, geometric, and signalization conditions of the interchange are provided in Exhibit 34-57 and Exhibit 34-58.

Exhibit 34-57
Example Problem 5:
Intersection Plan View

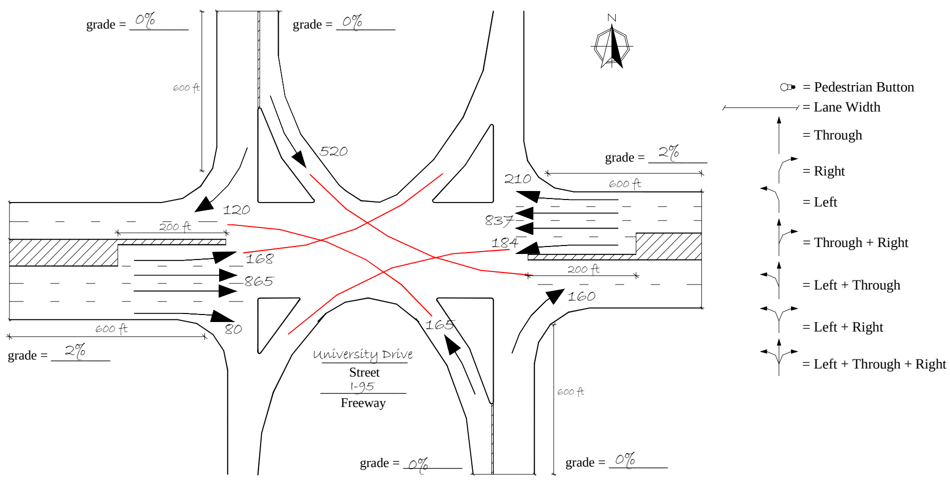


Exhibit 34-58
Example Problem 5:
Signalization Information

Phase	SPUI Interchange		
	1	**2**	**3**
NEMA	Φ (3+7+2R+6R)	Φ (3+8+4+7)	Φ (1+5+4R+8R)
Green time	16	32	38
Yellow + all red	8	8	8

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 75 ft for all turn movements.

There are 3% heavy vehicles on all eastbound and westbound movements, and the PHF for the interchange is 0.95. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of O-Ds

O-Ds through this SPUI are calculated on the basis of Exhibit 22-5. O-Ds can be calculated directly from the turning movements at a SPUI because it has only one intersection. The O-Ds and the corresponding PHF-adjusted values are presented in Exhibit 34-59.

Exhibit 34-59
Example Problem 5: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	165	174
B	160	168
C	120	126
D	520	547
E	168	177
F	80	84
G	210	221
H	184	194
I	865	911
J	837	881
K	0	0
L	0	0
M	0	0
N	0	0

Saturation Flow Rate Calculations

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hr/ln by using Equation 22-3. Traffic pressure is calculated by using Equation 22-4, while left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The left-turn permissive phases as well as the remaining adjustment factors are treated as specified in Chapter 18, Signalized Intersections. The results of the saturation flow rate calculations for all approaches are presented in Exhibit 34-60 and Exhibit 34-61.

Exhibit 34-60
Example Problem 5: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

	Eastbound				Westbound			
	Left Prot.	Left Perm.	Through	Right	Left Prot.	Left Perm.	Through	Right
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	2	1	1	1	2	1
Lane width adjustment (f_w)	0.967	0.967	0.967	0.967	0.967	0.967	0.967	0.967
Heavy-vehicle adjustment (f_{hv})	0.971	0.971	0.971	0.971	0.971	0.971	0.971	0.971
Grade adjustment (f_g)	0.990	0.990	0.990	0.990	0.990	0.990	0.990	0.990
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	0.952	1.000	1.000	1.000	0.952	1.000
Left-turn adjustment (f_{LT})	0.930	0.136	1.000	1.000	0.930	0.125	1.000	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	1.000	0.994	1.000	1.000	1.000	0.983
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.930	0.930	1.000	0.899	0.930	0.930	1.000	0.899
Traffic pressure adjustment for lane group (f_v)	0.950	0.951	0.998	0.946	0.950	0.954	0.995	0.964
Adjusted saturation flow (s, veh/hg/ln)	1,560	228	3,353	1,659	1,561	211	3,346	1,673

Note: Prot. = protected, Perm. = permitted.

Exhibit 34-61
Example Problem 5: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

	Northbound			Southbound		
	Left	Through	Right	Left	Through	Right
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	1	1	1	1
Lane width adjustment (f_w)	0.967	0.967	0.967	0.967	0.967	0.967
Heavy-vehicle adjustment (f_{hv})	1.000	1.000	1.000	1.000	1.000	1.000
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	0.899	1.000	1.000	0.899	1.000	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	0.899	1.000	1.000	0.899
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.899	1.000	0.899	0.899	1.000	0.899
Traffic pressure adjustment for lane group (f_v)	0.967	0.935	0.957	1.044	0.935	0.951
Adjusted saturation flow (s, veh/hg/ln)	1,597	1,717	1,580	1,724	1,717	1,571

Supplemental Uniform Delay Worksheet for Left Turns from Exclusive Lanes with Protected and Permitted Phases

Uniform delay for the eastbound and westbound left-turn movements must be calculated with a supplemental worksheet since both of these exclusive left-turn lanes have both protected and permitted movements. The intermediate calculations and uniform delay for the eastbound and westbound left turns are completed according to the methodology of Chapter 18, Signalized Intersections, and are shown in Exhibit 34-62.

Exhibit 34-62
Example Problem 5: Uniform Delay Calculations for Left Turns Featuring Both Permissive and Protected Phasing

	Eastbound Left	Westbound Left
C (s)	110	110
Leading left?	Yes	Yes
g (s)	16	16
g_q (s)	17	20
g_u (s)	13.01	11.78
r (s)	62.00	62.00
$X = v/c$	0.60	0.67
q_a (veh/s)	0.05	0.05
s_p (veh/s)	0.43	0.43
s_s (veh/s)	0.16	0.16
X_{perm}	0.78	0.92
X_{prot}	0.55	0.60
Case	1	1
Q_A (ft)	3.0	3.3
Q_u (ft)	0.9	1.1
Q_r (ft)	0.0	0.0
d_1 (s/veh)	22.1	22.7

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue to the available queue storage by using Equation 31-91. Exhibit 34-63 and Exhibit 34-64 present the calculations of the queue storage ratio for all movements. These exhibits also show the *v/c* ratio for each movement. Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-65 and Exhibit 34-66 provide the control delay for each movement of the interchange. The eastbound left turns for the permissive and protected phases are treated in combination in these calculations.

Exhibit 34-63
Example Problem 5: Queue Storage Ratio for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	Left	Through	Right	Left	Through	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	177	911	84	194	881	221
s (veh/h/ln)	672	1,676	1,659	661	1,673	1,673
g (s)	48.0	32.0	38.0	48.0	32.0	38.0
g/C	0.44	0.29	0.35	0.44	0.29	0.35
I	1.0	1.0	1.0	1.0	1.0	1.0
c (veh/h/ln group)	293	975	573	288	973	578
$X = v/c$	0.60	0.93	0.15	0.67	0.91	0.38
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1	1	1	1	1	1
P	0.44	0.29	0.35	0.44	0.29	0.35
r (s)	62.0	78.0	72.0	62.0	78.0	72.0
t_f (s)	0.00	0.01	0.00	0.01	0.01	0.00
q (veh/s)	0.05	0.25	0.02	0.05	0.24	0.06
q_g (veh/s)	0.05	0.25	0.02	0.05	0.24	0.06
q_r (veh/s)	0.05	0.25	0.02	0.05	0.24	0.06
Q_1 (veh)	4.1	14.2	1.8	4.7	13.6	5.1
Q_2 (veh)	0.7	2.3	0.1	0.9	1.9	0.3
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00
t_A	0	0	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0	0	0
Q_3 (veh)	0	0	0	0	0	0
Q (veh)	4.9	16.5	1.9	5.7	15.4	5.4
L_h (ft)	25.006	25.006	25.006	25.006	25.006	25.006
L_a (ft)	200	600	600	200	600	600
R_Q	0.61	0.69	0.08	0.71	0.64	0.23

Exhibit 34-64
Example Problem 5: Queue Storage Ratio for Northbound and Southbound Movements

	Northbound Movements			Southbound Movements		
	Left	Through	Right	Left	Through	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	174	0	168	547	0	126
s (veh/h/ln)	1,597	1,717	1,580	1,724	1,717	1,571
g (s)	38.0	38.0	16.0	38.0	38.0	16.0
g/C	0.35	0.35	0.15	0.35	0.35	0.15
I	1.0	1.0	1.0	1.0	1.0	1.0
c (veh/h/ln group)	552	593	230	596	593	228
$X = v/c$	0.31	0.00	0.73	0.92	0.00	0.55
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1	1	1	1	1	1
P	0.35	0.35	0.15	0.35	0.35	0.15
r (s)	72.0	72.0	94.0	72.0	72.0	94.0
t_f (s)	0.00	0.00	0.00	0.01	0.00	0.00
q (veh/s)	0.05	0.00	0.05	0.15	0.00	0.04
q_g (veh/s)	0.05	0.00	0.05	0.15	0.00	0.04
q_r (veh/s)	0.05	0.00	0.05	0.15	0.00	0.04
Q_1 (veh)	3.9	0.0	4.9	16.0	0.0	3.6
Q_2 (veh)	0.2	0.0	1.2	3.6	0.0	0.6
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00
t_A	0	0	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0	0	0
Q_3 (veh)	0	0	0	0	0	0
Q (veh)	4.1	0.0	6.1	19.6	0.0	4.2
L_h (ft)	25	25	25	25	25	25
L_a (ft)	600	600	600	600	600	600
R_Q	0.17	0.00	0.25	0.82	0.00	0.17

Exhibit 34-65
Example Problem 5: Control Delay for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	Left	Through	Right	Left	Through	Right
g (s)	-	32	38	-	32	38
g' (s)	48	-	-	48	-	-
g/C or g'/C	0.44	0.29	0.35	0.44	0.29	0.35
c (veh/h)	293	975	573	288	973	578
$X = v/c$	0.60	0.93	0.15	0.67	0.91	0.38
d_1 (s/veh)	22.1	38.0	24.8	22.8	37.5	27.2
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	8.9	16.6	0.5	11.8	13.4	1.9
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	31.0	54.6	25.4	34.6	51.0	29.1

	Northbound Movements			Southbound Movements		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**
g (s)	-	38	16	-	38	16
g' (s)	38	-	-	38	-	-
g/C or g'/C	0.35	0.35	0.15	0.35	0.35	0.15
c (veh/h)	552	593	230	596	593	228
$X = v/c$	0.31	0.00	0.73	0.92	0.00	0.55
d_1 (s/veh)	26.4	23.6	45.0	34.5	23.6	43.7
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	1.5	0.0	18.6	21.5	0.0	9.3
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	27.9	23.6	63.6	56.0	23.6	53.0

Exhibit 34-66
Example Problem 5: Control Delay for Northbound and Southbound Movements

Results

Delay for each O-D is estimated as the movement delay for the corresponding movement, as shown in Exhibit 34-67. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for the respective O-D is F. Exhibit 34-67 summarizes the results for each O-D of the interchange. As shown, no movements have a v/c ratio or R_Q exceeding 1, and therefore the LOS result is based on Exhibit 22-11.

O-D Movement	Control Delay (s)	v/c > 1?	R_Q > 1?	LOS
A	27.9	No	No	B
B	63.6	No	No	D
C	53.0	No	No	C
D	56.0	No	No	D
E	31.0	No	No	C
F	25.4	No	No	B
G	29.1	No	No	B
H	34.6	No	No	C
I	54.6	No	No	C
J	51.0	No	No	C
K	23.6	No	No	B
L	23.6	No	No	B
M	-	-	-	-
N	-	-	-	-

Exhibit 34-67
Example Problem 5: O-D Movement LOS

EXAMPLE PROBLEM 6: DIAMOND INTERCHANGE WITH ADJACENT INTERSECTION

The Interchange

At the diamond interchange described in Example Problem 1 (I-99 and University Drive), an adjacent intersection was built 300 ft to the west of the interchange (Spring Street, NB/SB, and University Drive, EB/WB). The traffic, geometric, and signalization conditions are shown in Exhibit 34-68 and Exhibit 34-69. The offset is referenced to the beginning of the green for the respective EB arterial approach.

Exhibit 34-68
Example Problem 6: Intersection Plan View

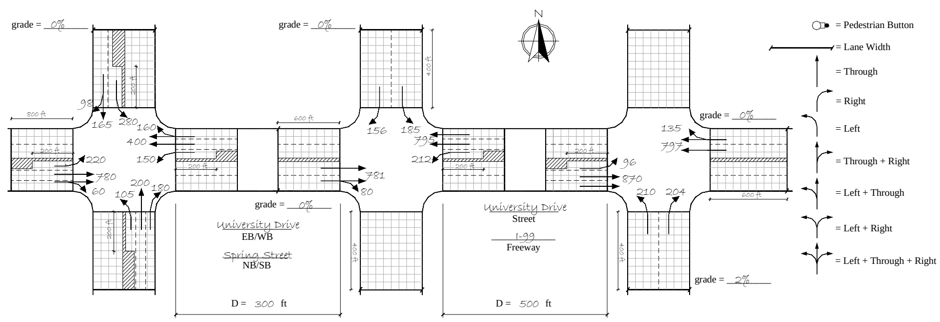


Exhibit 34-69
Example Problem 6: Signalization Information

Phase	Intersection I: 1	Intersection I: 2	Intersection I: 3	Intersection II: 1	Intersection II: 2	Intersection II: 3
NEMA	Φ (4+8)	Φ (4+7)	Φ (2+5)	Φ (4+8)	Φ (1+6)	Φ (3+8)
Green time	63	43	39	63	53	29
Yellow + all red	5	5	5	5	5	5
Offset		19			9	

Phase	Adjacent Intersection: 1	2	3	4
NEMA	Φ (1+5)	Φ (2+6)	Φ (4+7)	Φ (3+8)
Green time	33	59	24	24
Yellow + all red	5	5	5	5
Offset		19		

The Question

What are the control delay, queue storage ratio, and LOS for this interchange and the adjacent intersection?

The Facts

The closely spaced intersection operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii at the interchange are 50 ft for all right-turning movements and 75 ft for all left-turning movements.

There are 5% heavy vehicles on eastbound and westbound through movements of the interchange and all movements of the adjacent intersection. The PHF for the interchange–intersection system is 0.97. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of O-Ds

The O-Ds of the interchange are obtained as explained in Example Problem 1 and were presented in Exhibit 34-4.

Lane Utilization and Saturation Flow Rate Calculations

The adjacent intersection has a two-lane shared right and through lane group for both the inbound (arriving to the interchange) and outbound (leaving the interchange) approaches. The lane utilization factors for the inbound and outbound approaches of the closely spaced intersection are estimated by obtaining the respective lane utilization values (through or shared) from Exhibit 18-30 and subtracting 0.05. The resulting lane utilization factors are shown in Exhibit 34-70.

Exhibit 34-70
Example Problem 6: Lane Utilization Adjustment Calculations

Lane Group	Lane Utilization Factor
2-lane group EB (inbound)	0.902
2-lane group WB (outbound)	0.902

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hr/ln by using Equation 22-3. The saturation flows for each lane group of the adjacent intersection are estimated according to Chapter 18, Signalized Intersections. The results of the saturation flow rate calculations for all movements of the adjacent intersection and the interchange are presented in Exhibit 34-71 through Exhibit 34-73. Note that turn radius and traffic pressure adjustments are not considered in the adjacent intersection.

Exhibit 34-71
Example Problem 6: Saturation Flow Rate Calculation for Interchange Eastbound and Westbound Approaches

	Eastbound			Westbound		
	EXT-TH&R	INT-TH	INT-L	EXT-TH&R	INT-TH	INT-L
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	2	2	1	2	2	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	1.000	0.952	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.989	0.952	1.000	0.965	0.952	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.930	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	0.999	1.000	1.000	0.998	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.991	1.000	0.930	0.985	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	1.027	1.028	0.961	1.044	1.019	0.995
Adjusted saturation flow (s, veh/hg/ln)	3,670	3,540	1,698	3,637	3,510	1,759

Exhibit 34-72
Example Problem 6: Saturation Flow Rate Calculation for Interchange Northbound and Southbound Approaches

	Northbound		Southbound	
	Left	Right	Left	Right
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900
Number of lanes (N)	1	1	1	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	1.000	1.000	1.000	1.000
Grade adjustment (f_g)	0.990	0.990	0.990	0.990
Parking adjustment (f_p)	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	0.899	1.000	0.899	1.000
Right-turn adjustment (f_{RT})	1.000	0.899	1.000	0.899
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.899	0.899	0.899	0.899
Traffic pressure adjustment for lane group (f_v)	0.995	0.971	0.987	0.966
Adjusted saturation flow (s, veh/hg/ln)	1,682	1,650	1,669	1,633

Exhibit 34-73
Example Problem 6: Saturation Flow Rate Calculation for Adjacent Intersection

	Eastbound		Westbound		Northbound			Southbound	
	Through & Right	Left	Through & Right	Left	Through	Right	Left	Through & Right	Left
Base saturation flow (s_0, pc/hg/ln)	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
Number of lanes (N)	2	1	2	1	1	1	1	2	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	0.952	0.952	0.952	0.952	0.952	0.952	0.952
Grade adjustment (f_g)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.902	1.000	0.902	1.000	1.000	1.000	1.000	1.000	1.000
Left-turn adjustment (f_{LT})	1.000	0.930	1.000	0.930	1.000	1.000	0.899	1.000	0.899
Right-turn adjustment (f_{RT})	1.000	1.000	1.000	1.000	1.000	0.899	1.000	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted saturation flow (s, veh/hg/ln)	3,359	1,680	3,251	1,645	1,765	1,580	1,568	3,434	1,654

Common Green and Lost Time due to Downstream Queue Calculations

Common green is calculated between certain movements that can contribute to excessive downstream queues or demand starvation, depending on the signal-phasing sequence. The adjacent intersection is offset by 10 s from Intersection 2 and by 0 s from Intersection 1. Exhibit 34-74 presents the beginning and end of each phase at the three intersections and the calculations of common green between the relevant movements at the three intersections.

Exhibit 34-74
Example Problem 6: Common Green Calculations

Phase	Intersection I Green Begin	Intersection I Green End	Intersection II Green Begin	Intersection II Green End
Phase 1	0	63	150	53
Phase 2	68	111	58	111
Phase 3	116	155	116	145

Phase	Adjacent Intersection Phase Begin	Adjacent Intersection Phase End
Phase 1	0	33
Phase 2	38	62
Phase 3	67	96
Phase 4	96	155

Movement	First Green Time Within Cycle Begin	First Green Time Within Cycle End	Second Green Time Within Cycle Begin	Second Green Time Within Cycle End	Common Green Time
EB EXT THRU	0	63			53
EB INT THRU	150	53	116	150	
WB EXT THRU	150	53			53
WB INT THRU	0	111			
SB RAMP	116	155			34
EB INT THRU	150	53	116	150	
NB RAMP	58	111			53
WB INT THRU	0	111			
WB INT LEFT	68	111			0
EB INT THRU	150	53			
EB INT LEFT	116	145			0
WB INT THRU	0	111			
EB EXT THRU	0	63			25
ADJ EB THRU	38	97			
EB EXT THRU	0	63			0
ADJ SB LEFT	102	126			
EB EXT THRU	0	63			0
ADJ NB Right	131	155			
ADJ WB THRU	38	97			59
WB INT THRU	0	111			
ADJ WB THRU	38	97			0
SB RAMP	116	155			

The next step is the calculation of lost time due to downstream queues. At an adjacent intersection, additional lost time due to interchange operations may occur at the intersection's eastbound, southbound left-turn, and northbound right-turn approaches. Furthermore, the interchange westbound internal link and southbound ramp may experience additional lost time due to operations at the adjacent closely spaced intersection.

To estimate whether these approaches experience additional lost time, the procedure estimates the queue at the beginning of the intersection's eastbound through arterial phase, southbound left-turn phase, and northbound right-turn phase. They are calculated by using Equations 22-14 and 22-15. The resulting queues are subtracted from the downstream link length (link between the closely spaced intersection and the interchange) to determine the storage at the beginning of each phase. Exhibit 34-75 presents the calculation of lost time due to downstream queues. The results indicate that the southbound left-turn and northbound right-turn movements of the adjacent intersection experience additional lost time of 2.10 and 3.07 s, respectively.

Exhibit 34-75
Example Problem 6: Lost Time due to Downstream Queues

Movement	Interchange EB EXT-TH	Interchange SB-L	Interchange WB EXT-TH	Interchange NB-L
V_R or V_A (veh/h)	191	805	216	822
N_R or N_A	1	2	1	2
G_R or G_A (s)	39	63	53	63
G_D (s)	97	97	111	111
C (s)	160	160	160	160
CG_{UD} or CG_{RD} (s)	53	34	53	53
Queue length (Q_A or Q_R) (ft)	0.0	0.0	0.0	0.0

Effective Green Adjustment	Lost Time due to Downstream Queue — Interchange EB EXT-TH	SB-L	WB EXT-TH	NB-L
G_R or G_A (s)	63	39	63	53
C (s)	160	160	160	160
D_{QA} or D_{QR} (ft)	500	500	500	500
CG_{UD} or CG_{RD} (s)	53	34	53	53
Additional lost time, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (s)	0.0	0.0	0.0	0.0
Total lost time, t'_L (s)	5.0	5.0	5.0	5.0
Effective green time, g' (s)	63	39	63	53

Movement	Adjacent Intersection EB-TH	Adjacent Intersection SB-L	Adjacent Intersection NB-R	Interchange WB INT-TH	Interchange SB-R
V_R or V_A (veh/h)	474	804	804	156	795
N_R or N_A	1	2	2	1	2
G_R or G_A (s)	48	59	59	39	111
G_D (s)	63	63	63	59	59
C (s)	160	160	160	160	160
CG_{UD} or CG_{RD} (s)	25.0	0.0	0.0	15	39
Queue length (Q_A or Q_R) (ft)	56.9	102.6	102.6	0.0	91.1

Effective Green Adjustment	Lost Time due to Downstream Queue — Adjacent Intersection EB-TH	Adjacent Intersection SB-L	Adjacent Intersection NB-R	Interchange WB INT-TH	Interchange SB-R
G_R or G_A (s)	59	24	24	119	39
C (s)	160	160	160	160	160
D_{QA} or D_{QR} (ft)	243	197	197	300	209
CG_{UD} or CG_{RD} (s)	25.0	29	0	15	39
Additional lost time, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (s)	0.00	2.10	3.07	0.0	0.0
Total lost time, t'_L (s)	5.00	7.10	8.07	5.0	5.0
Effective green time, g' (s)	59.0	21.9	20.9	119	39

Queue Storage and Control Delay

The queue storage ratio is estimated as the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 34-76 and Exhibit 34-77 present the calculations of the queue storage ratio for all approaches of the interchange, while Exhibit 34-78 gives the results of all approaches of the adjacent intersection. The v/c ratio for the respective movements is also provided in these exhibits.

Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-79 through Exhibit 34-81 summarize the control delay estimates for all approaches of the interchange and adjacent signalized intersection.

Exhibit 34-76
Example Problem 6: Queue Storage Ratio for Interchange Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	888	99	897	961	219	820
s (veh/h/ln)	1,835	1,699	1,770	1,819	1,759	1,755
g (s)	63	29	97	63	43	111
g/C	0.39	0.18	0.61	0.39	0.27	0.69
I	1.00	0.75	0.75	1.00	0.68	0.68
c (veh/h/ln group)	1,448	308	2,146	1,448	473	2,435
$X = v/c$	0.61	0.32	0.42	0.66	0.46	0.34
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.333	1.000	1.000	1.333
P	0.39	0.18	0.81	0.39	0.27	0.92
r (s)	97.00	131.00	63.00	97.00	117.00	49.00
t_f (s)	0.01	0.00	0.00	0.01	0.00	0.00
q (veh/s)	0.25	0.03	0.25	0.27	0.06	0.23
q_g (veh/s)	0.25	0.03	0.33	0.27	0.06	0.30
q_r (veh/s)	0.25	0.03	0.12	0.27	0.06	0.06
Q_1 (veh)	13.8	3.5	8.5	13.0	7.3	1.1
Q_2 (veh)	0.8	0.2	0.1	1.1	0.3	0.1
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00
t_A	0	0	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	14.6	3.7	8.6	14.1	7.6	1.2
L_h (ft)	25	25	25	25	25	25
L_a (ft)	600	200	500	600	200	500
R_Q	0.61	0.46	0.43	0.59	0.95	0.06

Exhibit 34-77
Example Problem 6: Queue Storage Ratio for Interchange Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0
v (veh/h/ln group)	216	210	191	161
s (veh/h/ln)	1,682	1,651	1,669	1,634
g (s)	53	53	39	39
g/C	0.33	0.33	0.24	0.24
I	1.00	1.00	1.00	1.00
c (veh/h/ln group)	557	547	407	398
$X = v/c$	0.39	0.38	0.47	0.40
r_a (ft/s^2)	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4
S_s (mi/h)	5	5	5	5
S_{pl} (mi/h)	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.000	1.000
P	0.33	0.33	0.24	0.24
r (s)	107.00	107.00	121.00	121.00
t_f (s)	0.00	0.00	0.00	0.00
q (veh/s)	0.06	0.06	0.05	0.04
q_g (veh/s)	0.06	0.06	0.05	0.04
q_r (veh/s)	0.06	0.06	0.05	0.04
Q_1 (veh)	6.6	6.4	6.5	5.4
Q_2 (veh)	0.3	0.3	0.4	0.3
T	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00
t_A	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0
Q (veh)	6.9	6.7	7.0	5.7
L_h (ft)	25	25	25	25
L_a (ft)	400	400	400	400
R_Q	0.43	0.42	0.43	0.36

Exhibit 34-78
Example Problem 6: Queue Storage Ratio for Adjacent Intersection Movements

	Eastbound		Westbound		Northbound			Southbound	
	Through & Right	Left	Through & Right	Left	Through	Right	Left	Through & Right	Left
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	866	227	577	309	206	186	108	542	289
s (veh/h/ln)	1,679	1,680	1,650	1,722	1,765	1,580	1,568	1,717	1,654
g (s)	59.0	33	59	33	24.0	20.9	24.0	24	21.9
g/C	0.37	0.21	0.37	0.21	0.15	0.13	0.15	0.15	0.14
I	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
c (veh/h/ln group)	1,288	346	1,218	355	265	237	235	515	248
$X = v/c$	0.67	0.65	0.47	0.46	0.78	0.90	0.46	1.05	1.28
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
P	0.37	0.21	0.37	0.21	0.15	0.13	0.15	0.15	0.14
r (s)	101.00	127.00	101.00	127.00	136.00	139.07	136.00	136.00	138.10
t_f (s)	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.01	0.01
q (veh/s)	0.24	0.06	0.16	0.04	0.06	0.05	0.03	0.08	0.08
q_g (veh/s)	0.24	0.06	0.16	0.04	0.06	0.05	0.03	0.08	0.08
q_r (veh/s)	0.24	0.06	0.16	0.04	0.06	0.05	0.03	0.08	0.08
Q_1 (veh)	14.3	8.4	8.7	5.5	8.0	7.4	4.0	10.4	9.2
Q_2 (veh)	1.1	0.9	0.5	0.4	1.5	2.3	0.4	5.0	9.7
T	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.39	15.6
t_A	0	0	0	0	0	0	0	0.25	0.25
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.39	15.6
Q_b (veh)	0	0	0	0	0	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	15.4	9.3	9.1	5.9	9.5	9.8	4.4	15.5	18.8
L_h (ft)	25	25	25	25	25	25	25	25	25
L_a (ft)	800	200	300	200	800	800	200	800	200
R_Q	0.48	1.16	0.76	0.73	0.30	0.30	0.55	0.48	2.36

Exhibit 34-79
Example Problem 6: Control Delay for Interchange Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
g (s)	-	29	97	-	43	111
g' (s)	63	-	-	63	-	-
g/C or g'/C	0.39	0.18	0.61	0.39	0.27	0.69
c (veh/h)	1,448	308	2,146	1,448	473	2,435
$X = v/c$	0.61	0.32	0.42	0.68	0.46	0.34
d_1 (s/veh)	38.8	56.9	16.6	30.6	48.8	2.0
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	3.9	2.1	0.5	5.4	2.2	0.3
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	0.560	1.000	1.000	0.283
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	42.6	59.0	17.1	36.0	51.0	2.2

Exhibit 34-80
Example Problem 6: Control Delay for Interchange Northbound and Southbound Movements

	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements Right
g (s)	-	53	-	39
g' (s)	53	-	39	-
g/C or *g'/C*	0.33	0.33	0.24	0.24
c (veh/h)	557	547	407	398
X = *v/c*	0.39	0.38	0.47	0.40
d_1 (s/veh)	41.1	41.0	51.7	50.7
k	0.5	0.5	0.5	0.5
d_2 (s/veh)	2.0	2.0	3.8	3.0
d_3 (s/veh)	0.0	0.0	0.0	0.0
PF	1	1	1	1
k_{min}	0.04	0.04	0.04	0.04
u	0	0	0	0
t	0	0	0	0
d (s/veh)	43.1	43.0	55.5	53.8

Exhibit 34-81
Example Problem 6: Control Delay for Adjacent Intersection Movements

	Eastbound		Westbound		Northbound			Southbound	
	Through & Right	Left	Through & Right	Left	Through	Right	Left	Through & Right	Left
g (s)	-	33.0	59.0	33.0	24.0	-	24.0	24.0	-
g' (s)	59.0	-	-	-	-	20.9	-	-	21.9
g/C or *g'/C*	0.37	0.21	0.37	0.21	0.15	0.13	0.15	0.15	0.14
c (veh/h)	1,288	346	1,218	355	265	237	235	258	248
X = *v/c*	0.67	0.65	0.47	0.87	0.78	0.78	0.46	1.05	1.28
d_1 (s/veh)	42.5	58.3	38.7	55.6	65.4	68.5	62.1	68.0	69.0
k	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	6.0	9.3	2.7	4.4	20.0	40.7	6.4	70.6	153.6
d_3 (s/veh)	0	0	0	0	0	0	0	0	0
PF	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0	0	0	0
t	0	0	0	0	0	0	0	0	0
d (s/veh)	48.5	67.6	41.4	60.0	85.4	109.1	68.4	138.6	226.6

Results

Delay for each O-D is estimated as the sum of the movement delays for each movement utilized by the O-D, as indicated in Equation 22-1. The *v/c* and queue storage ratios are checked next. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement is F. The final delay calculations and resulting LOS for each O-D and each lane group are presented in Exhibit 34-82 and Exhibit 34-83. As shown, the *v/c* ratio and R_Q for all O-Ds are all below 1, and therefore the LOS for all O-Ds is determined by using Exhibit 22-11. The LOS for each lane group at the adjacent intersection is assigned on the basis of Chapter 18, Signalized Intersections.

Exhibit 34-82
Example Problem 6:
Interchange O-D Movement LOS

O-D Movement	Control Delay (s)	*v/c* > 1?	R_Q > 1?	LOS
A	45.3	No	No	C
B	43.0	No	No	C
C	53.8	No	No	C
D	72.6	No	No	D
E	98.1	No	No	E
F	39.1	No	No	C
G	36.0	No	No	C
H	87.0	No	No	E
I	56.2	No	No	D
J	38.2	No	No	C
K	-	-	-	-
L	-	-	-	-
M	94.1	No	No	E
N	114.5	No	No	E

Exhibit 34-83
Example Problem 6:
Adjacent Intersection Movement LOS

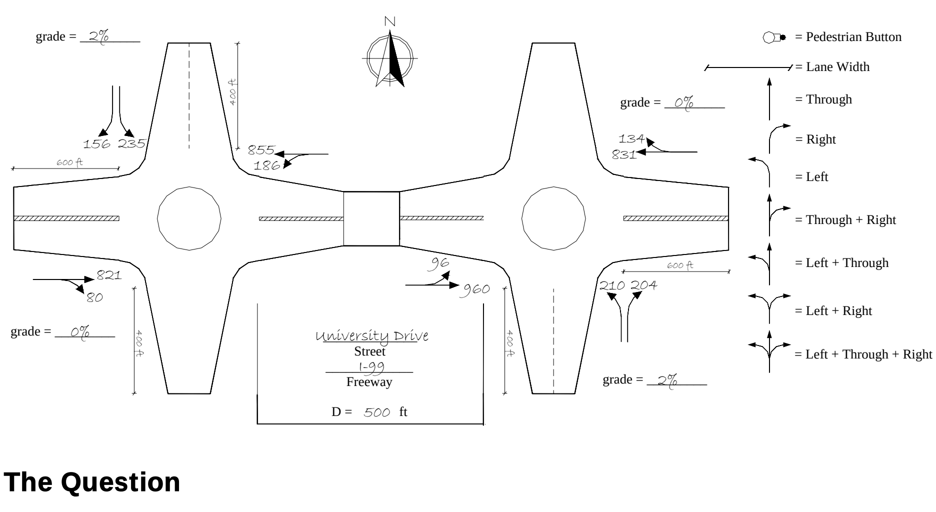
Approach	Lane Group	Control Delay (s)	LOS
EB	Through and right	48.5	C
	Left	67.6	D
WB	Through and right	41.4	C
	Left	60.0	D
NB	Through	85.4	E
	Right	109.1	E
	Left	68.4	D
SB	Through and right	138.6	F
	Left	226.6	F

EXAMPLE PROBLEM 7: DIAMOND INTERCHANGE WITH ROUNDABOUTS

The Interchange

The interchange of I-99 (NB/SB) and University Drive (EB/WB) is a diamond interchange featuring roundabouts. The traffic conditions of the interchange are provided in Exhibit 34-84.

Exhibit 34-84
Example Problem 7:
Intersection Plan View



The Question

What are the control delay and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange. This interchange has 3% heavy vehicles, and the PHF of the interchange is 0.97. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). The results of the O-D calculations and the resulting PHF-adjusted values are presented in Exhibit 34-85.

Exhibit 34-85
Example Problem 7: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)	HV-Adjusted Demand (pc/h)
A	179	185	191
B	169	174	179
C	122	126	130
D	228	235	242
E	93	96	99
F	78	80	82
G	94	97	100
H	119	123	127
I	509	525	541
J	529	545	561
K	0	0	0
L	0	0	0
M	0	0	0
N	0	0	0

Calculation of Approach Capacity and Control Delay

To estimate the delay of each approach to the roundabout, the procedures outlined in Chapter 21, Roundabouts, are used to estimate the entering and conflicting flow rates and the resulting capacity of each approach.

Exhibit 22-26 and Exhibit 22-27 are used to determine the entering and conflicting flow rates for each approach of the interchange. For example, the northbound ramp movement (Number 13 in Exhibit 22-26) consists of O-D Movements A, B, K, and M at a diamond interchange (Exhibit 22-27). The conflicting flow (Number 12) consists of O-D Movements D, E, I, and N. Exhibit 34-86 shows the entering and conflicting flow for each approach, along with the corresponding capacity and delay.

Exhibit 34-86
Example Problem 7: Approach Capacity and Delay Calculations

Approach	Entering Flow (pc/h)	Conflicting Flow (pc/h)	Capacity (pc/h)	Control Delay (s/veh)
EB EXT	722	369	782	34.5
EB INT	882	0	1,130	13.4
WB EXT	788	289	846	33.8
WB INT	879	0	1,130	13.3
NB RAMP	370	882	468	30.9
SB RAMP	372	879	469	31.1

O-D Movement Control Delay and LOS

Delay for each O-D is estimated as the sum of the approach delays for each approach utilized by the O-D. For example, O-D Movement A will utilize the northbound ramp approach and the westbound internal through approach. The control delays for these approaches are then summed to estimate the control delay for O-D Movement A. LOS for each O-D is assigned on the basis of Exhibit 22-13. The resulting control delay and LOS for all O-D movements are shown in Exhibit 34-87.

Exhibit 34-87
Example Problem 7: Control Delay and LOS for Each O-D Movement

O-D	Control Delay (s/veh)	LOS
A	44.2	D
B	30.9	C
C	31.1	C
D	44.5	D
E	47.9	D
F	34.5	C
G	33.8	C
H	47.1	D
I	47.9	D
J	47.1	D
K	30.9	C
L	31.1	C
M	44.2	D
N	44.5	D

EXAMPLE PROBLEM 8: ALTERNATIVE ANALYSIS TOOL

This example presents a simulation analysis of the basic diamond interchange configuration originally described in Example Problem 1 of Chapter 22. A few changes were made to introduce elements that are beyond the stated limitations of the procedures presented in that chapter. The use of a typical simulation tool to address the limitations is described in this section.

The need to determine performance measures from an analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, the example presented in this chapter has applied existing versions of alternative tools and therefore does not reflect the trajectory-based measures described in Chapter 7.

Operational Characteristics

A two-way STOP-controlled (TWSC) intersection was introduced 600 ft west of the first signalized intersection of the interchange. Ramp metering signals were installed on both of the freeway entrance ramps. Right-turn storage bays were introduced on all approaches to the interchange that accommodated right turns. The demand volumes were modified to introduce conditions that varied from undersaturated to heavily oversaturated. The signal timing plan was modified to accommodate the distribution of volumes.

Exhibit 34-88 shows the interchange configuration and demand volumes. The demand volumes are referenced to the total directional arterial demand *d*, which varies from 600 to 1,800 veh/h. The turning movement volumes entering and leaving the arterial have been balanced for continuity of traffic flow. The turning movements entering and leaving the freeway were set at 25% of the total approach volumes and were adjusted proportionally to match the arterial demand volumes. The cross-street entry demand from the TWSC intersection was held constant at 100 veh/h in each direction, with 50% assigned to the left and right turns. No through vehicles were assigned from the cross street at this intersection.

Exhibit 34-88
Example Problem 8: Interchange Configuration and Demand Volumes

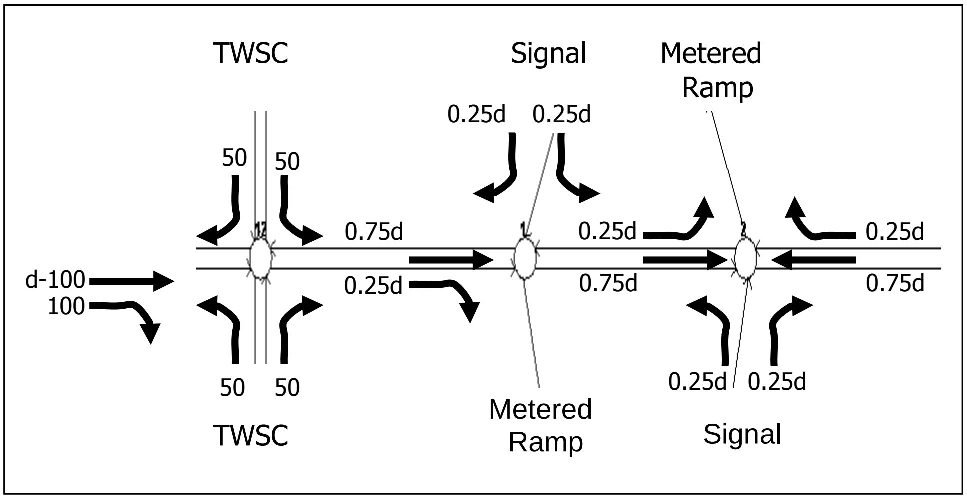


Exhibit 34-89 shows the signal timing plan for both intersections of the diamond interchange. A simple three-phase operation at each intersection is depicted in this table. No attempt has been made to optimize the phasing or timing since the main purpose of this example is to demonstrate self-aggravating phenomena that are not recognized by the Chapter 22 procedures. The ramp metering signals installed on each of the entrance ramps were set to release a single vehicle at 10-s intervals, giving a capacity of 360 veh/h for each ramp.

Exhibit 34-89
Example Problem 8: Signal Timing Plan

Movement	Green	Yellow	All Red
Entry through/left	20	4	1
Entry and exit through/right	45	4	1
Ramp	20	4	1
Cycle length		100 s	

Summary of Simulation Runs

The operation of this interchange was simulated by using demand volumes *d* ranging from 600 veh/h (very undersaturated) to 1,800 veh/h (very oversaturated). The volume increment was 200 veh/h. Thirty simulation runs were made for each condition to accommodate the stochastic variation inherent in simulation runs.

Two configurations were examined for each of the demand levels:

1. A single intersection at the west end of the diamond interchange, and
2. The full diamond interchange with ramp metering.

Both of these configurations are illustrated in Exhibit 34-90.

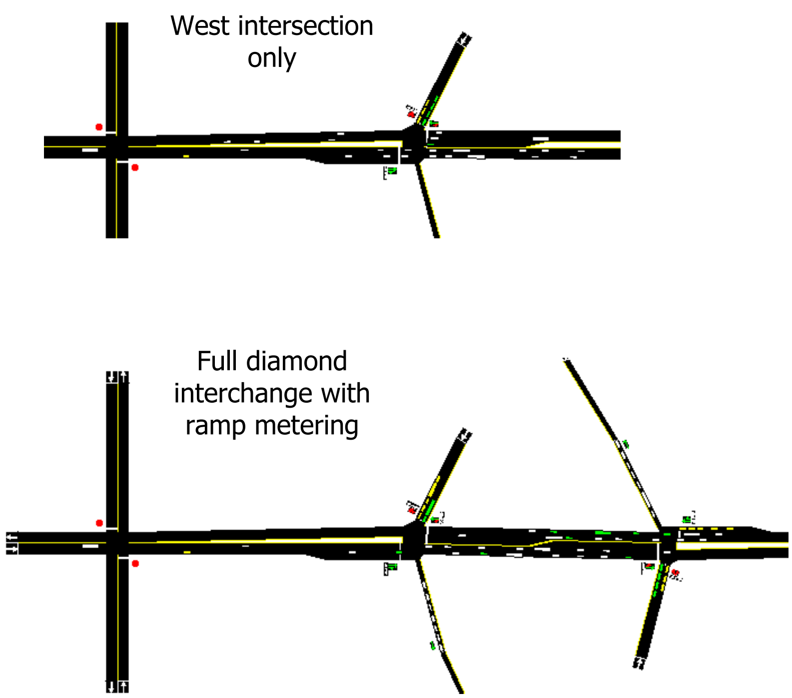


Exhibit 34-90
Example Problem 8: Physical Configurations Examined

The west intersection was examined separately to demonstrate the differences between a signalized intersection operating independently and one that operates as part of a diamond interchange with mutual interactions between the intersections at each end.

Diamond Interchange Operation

Exhibit 34-91 illustrates the self-aggravating effects from interactions among the two signals that make up the interchange and the ramp metering. Backup and congestion are observed at high demands on all approaches. The left-turn bays on the internal interchange segments spill over to block through traffic. Backup from the ramp metering signals causes additional impediment to traffic trying to leave the interchange.

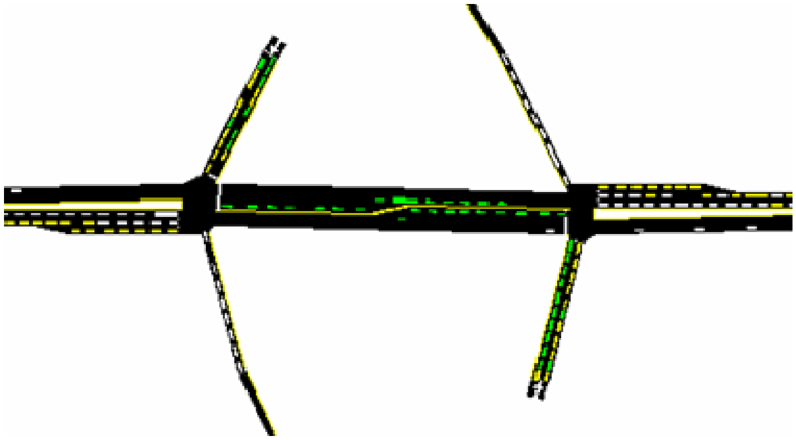


Exhibit 34-91
Example Problem 8: Congested Approaches to Diamond Interchange

Excessive delays will be associated with the oversaturated operation. However, for purposes of this example, the reduction in capacity is of more interest because capacity reductions due to self-aggravating phenomena are not fully recognized by the Chapter 22 methodology. Proper assessment of delay with heavy oversaturation would require a more complex procedure involving multiperiod analysis with possible consideration of route diversion due to the excessive congestion. Therefore, this example will be limited to examining the capacity reduction that results from interaction between the elements within this system. The extent of the capacity reduction will be estimated by the relationship between demand (input) and discharge (output) on the various segments.

Exhibit 34-92 shows the westbound arterial discharge from the diamond interchange (through plus left turns) as a function of the arterial demand d. Note that the discharge tracks the demand at low volumes, indicating that all of the arrivals were accommodated. As the demand increases, the discharge levels off at a point that indicates the capacity of the approach. When the approach is a part of an isolated intersection, the capacity approaches about 1,600 veh/h. A much lower capacity (about 850 veh/h) is attainable in the case of the diamond interchange with ramp metering. A number of self-aggravating phenomena reduce the capacity. Some westbound vehicles are unable to enter the east intersection because of backup from internal westbound left-turn bay spillover. Other westbound vehicles are unable to exit the interchange because of backup from the ramp metering signal and because of blockage of the intersection by left-turning exit ramp vehicles. The net result is a substantial reduction in capacity that would not be evident from application of the Chapter 22 methodology.

Exhibit 34-92
Example Problem 8:
Discharge from the Diamond Interchange Under the Full Range of Arterial Demand

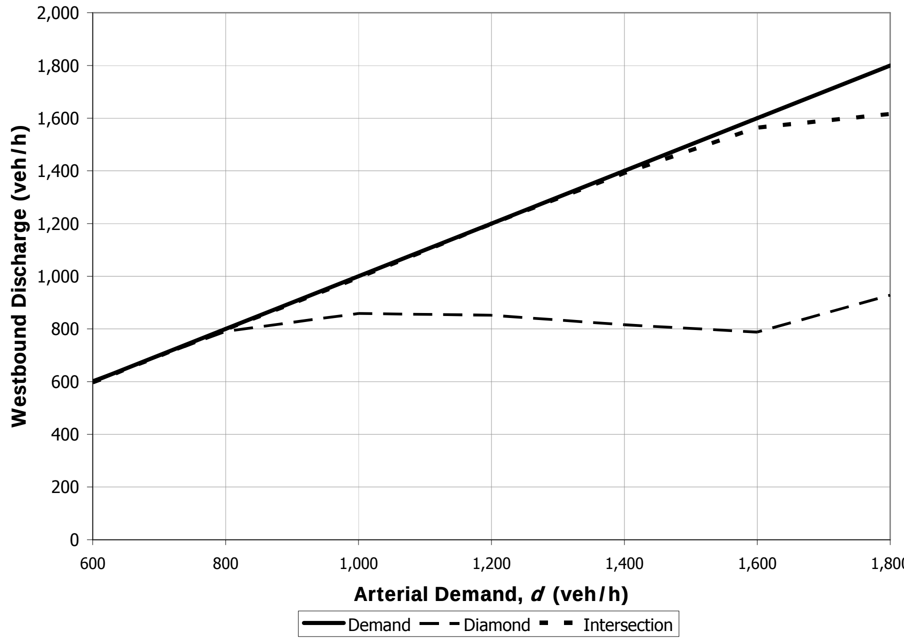
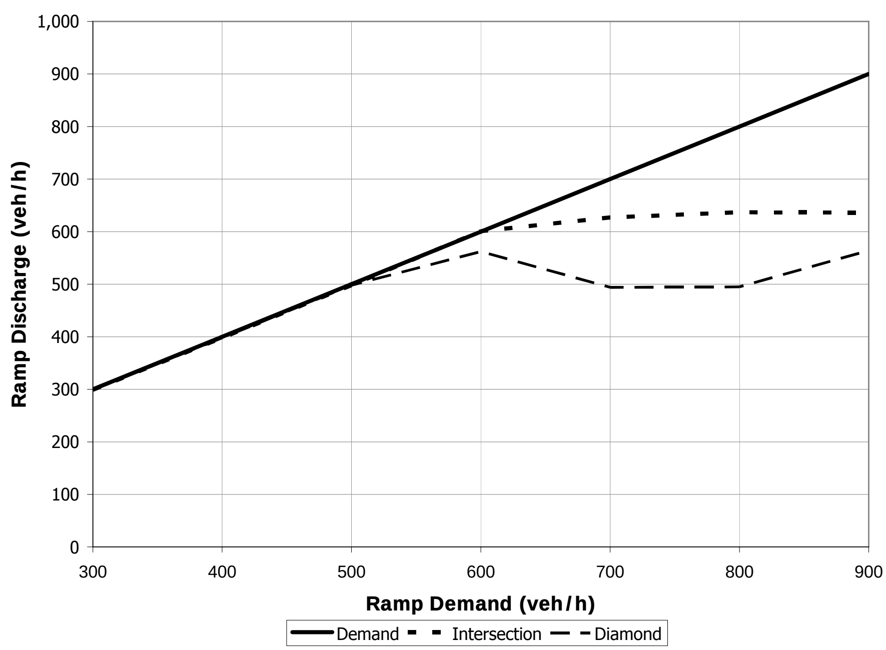


Exhibit 34-93 shows the effect of the demand volume on the southbound exit ramp discharge at the west signal of the diamond interchange. With an isolated signal, the discharge levels off at the approach capacity. As shown, the capacity is reduced slightly when the signal is part of a diamond interchange. The reduction was not as apparent as it was for the arterial movements because the blockage effects are not as significant. Some left turns were unable to enter the intersection because of backup from the east signal. The right turns from the ramp were not subject to any blockage effects.

Exhibit 34-93
Example Problem 8:
Discharge from the Southbound Exit Ramp Under the Full Range of Ramp Demand



TWSC Intersection Operation

The TWSC analysis procedures prescribed in Chapter 19 recognize the effects of adjacent signalized intersections to some extent, but they do not address cases when an approach is blocked throughout part of a cycle by stationary queues that prevent vehicles from entering on the minor street. This situation is depicted in Exhibit 34-94, in which a stationary queue of eastbound vehicles backed up from the west intersection of the diamond interchange has blocked the entry to the intersection for three of the four minor street movements.

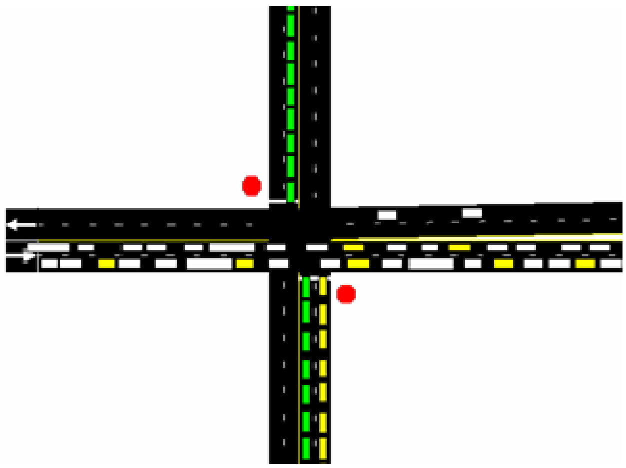
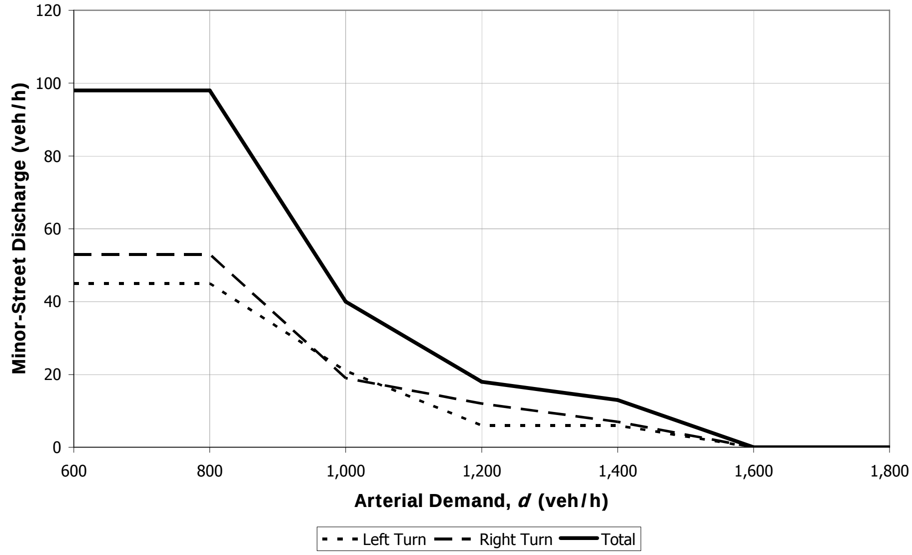


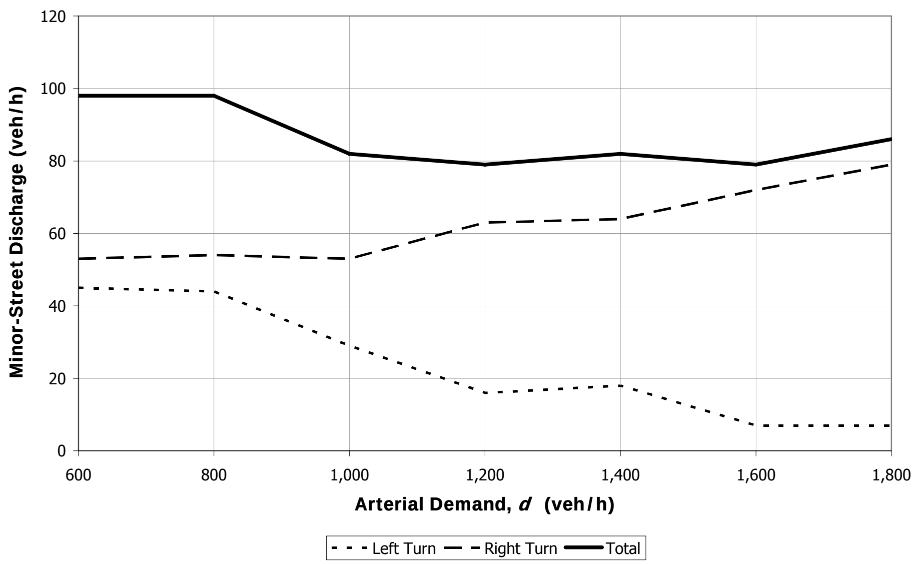
Exhibit 34-94
Example Problem 8: Congested Approaches to the TWSC Intersection

Exhibit 34-95 shows the minor-street entry as a function of the arterial demand. Unlike the other movements in this example, the minor-street demand was kept constant throughout the entire range of arterial demand. It is a well-established principle of TWSC analysis that the entry capacity for the minor-street movements diminishes with increasing major-street volumes. That phenomenon is depicted clearly for northbound traffic in Exhibit 34-94. It is evident here that the capacity begins to drop below the demand at about 800 veh/h in each direction on the arterial. The southbound situation, on the other hand, presents some surprising results. The southbound left turn is impeded by the queue of westbound vehicles backed up from the interchange, as expected. The southbound right turn, assisted by gaps created by the interchange signal, experiences an increase in capacity that produces entry volumes that exceed the original demand. The animated graphics display indicates that some of the southbound left-turn vehicles were unable to maneuver into the proper lane. The driver behavior model of the simulation tool reassigned these vehicles to right turns because of excessive waiting times. This effect provides a clear example of the difference between simulation modeling and the analytical approach presented throughout the HCM.

Exhibit 34-95
Example Problem 8: Effect of Arterial Demand on Minor-Street Discharge at the TWSC Intersection



(a) Northbound



(b) Southbound