CHAPTER 34
INTERCHANGE RAMP TERMINALS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 34 is the supplemental chapter for Chapter 23, Ramp Terminals and Alternative Intersections, which is found in Volume 3 of the *Highway Capacity Manual* (HCM). This chapter provides 17 example problems demonstrating the application of the Chapter 23 methodologies for evaluating the performance of distributed intersections, including restricted crossing U-turn (RCUT), median U-turn (MUT), and displaced left-turn (DLT) intersections. It also presents a procedure for interchange type selection, which can be used to evaluate the operational performance of various interchange types. Finally, this chapter provides worksheets for converting origin–destination (O-D) flows to turn movement flows, and vice versa, for various interchange types.

Methodologies for the analysis of interchanges involving freeways and surface streets (i.e., service interchanges) were developed primarily on the basis of research conducted through the National Cooperative Highway Research Program (*1*–*3*) and elsewhere (*4*). Development of HCM analysis procedures for alternative intersection and interchange designs was conducted through the Federal Highway Administration (*5*).

2. EXAMPLE PROBLEMS

INTRODUCTION

This section describes the application of each of the final design, operational analysis for interchange type selection, and roundabouts analysis methods through the use of example problems. Exhibit 34-1 describes each of the example problems included in this chapter and indicates the methodology applied.

Exhibit 34-1
Example Problem Descriptions

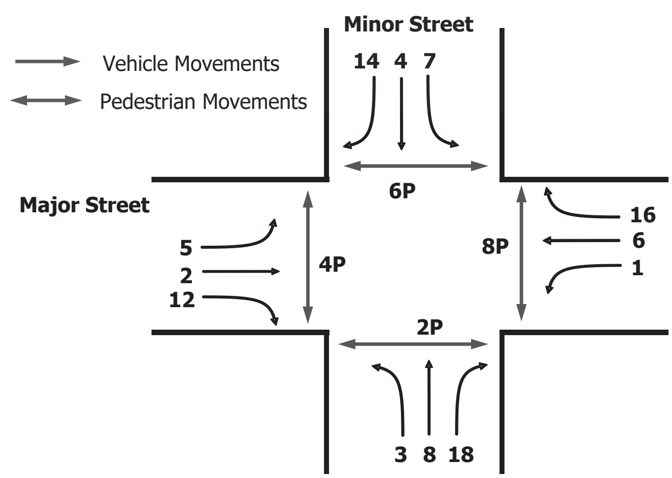
Example Problem	Description	Application
1	Diamond interchange	Operational
2	Parclo A-2Q interchange	Operational
3	Diamond interchange with four-phase signalization and queue spillback	Operational
4	Diamond interchange with demand starvation	Operational
5	Diverging diamond interchange with signalized control	Operational
6	Diverging diamond interchange with YIELD-controlled turns	Operational
7	Single-point urban interchange	Operational
8	Diamond interchange with closely spaced intersections	Operational
9	Diamond interchange with roundabouts	Operational
10	Compare eight types of signalized interchanges	Interchange type selection
11	Diamond interchange analysis using simulation	Alternative tools
12	Four-legged RCUT with merges	Operational
13	Three-legged RCUT with STOP signs	Operational
14	Four-legged RCUT with signals	Operational
15	Four-legged MUT with STOP signs	Operational
16	Partial DLT intersection	Operational
17	Full DLT intersection	Operational

Note: Parclo = partial cloverleaf, RCUT = restricted crossing U-turn, MUT = median U-turn, DLT = displaced left turn.

INTERSECTION TRAFFIC MOVEMENTS

Exhibit 34-2 illustrates typical vehicle and pedestrian traffic movements for the intersections in this chapter. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. Each movement is assigned a unique number or a number and letter combination. The letter P denotes a pedestrian movement. The number assigned to each left-turn and through movement is the same as the number assigned to each phase by National Electrical Manufacturers Association (NEMA) specification.

Exhibit 34-2
Intersection Traffic Movements and Numbering Scheme



Intersection traffic movements are assigned the right-of-way by the signal controller. Each movement is assigned to one or more signal phases. A phase is defined as the green, yellow change, and red clearance intervals in a cycle that are assigned to a specified traffic movement (or movements) (*6*). The assignment of movements to phases varies in practice with the desired phase sequence and the movements present at the intersection.

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 34-3 provides the interchange volumes and channelization, and Exhibit 34-4 provides the signalization information. The offset is referenced to the beginning of green on the EB direction of the arterial.

Exhibit 34-3
Example Problem 1: Interchange Volumes and Channelization

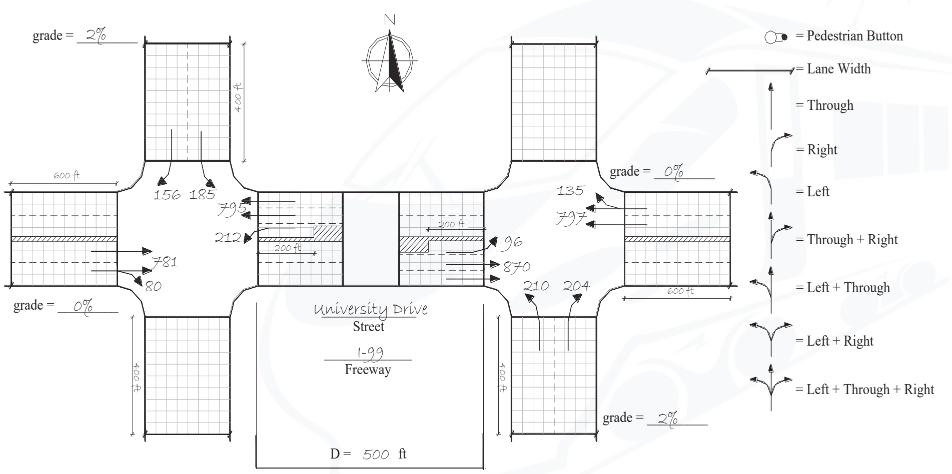


Exhibit 34-4
Example Problem 1: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (2+6)	Φ (1+6)	Φ (4+7)	Φ (2+6)	Φ (3+8)	Φ (2+5)
Green time (s)	63	43	39	63	53	29
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		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. Extra distance traveled along each freeway ramp is 100 ft.

Heavy vehicles account for 6.1% of 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. The grade is 2% on the NB and SB approaches.

Solution

Calculation of Origin–Destination Movements

O-D movements through this diamond interchange are calculated on the basis of the worksheet provided in Exhibit 34-169 in Section 4. 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 34-5.

Exhibit 34-5
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. Lane utilization factors for the external through approaches are presented in Exhibit 34-6.

Exhibit 34-6
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 in the base saturation flow rate of 1,900 pc/hg/ln by using Equation 23-14. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-6. Traffic pressure is calculated by using Equation 23-15. The left- and right-turn adjustment factors are estimated by using Equations 23-20 through 23-23. These equations use an adjustment factor for travel path radius calculated by Equation 23-19. The remaining adjustment factors are calculated as indicated in Chapter 19, Signalized Intersections. The estimated saturation flow rates for all approaches are shown in Exhibit 34-7 and Exhibit 34-8.

Exhibit 34-7
Example Problem 1: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

Value	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 and grade adjustment (f_{HVg})	0.952	0.952	1.000	0.952	0.952	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-8
Example Problem 1: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

Value	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 and grade adjustment (f_{HVg})	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

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

Exhibit 34-9 first provides the beginning and end times of the green for each phase at the two intersections on the assumption that Phase 1 of the first intersection begins at time zero. On the basis of the information provided in Exhibit 34-9, 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 34-9
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			

Notes: EXT = external, INT = internal, THRU = through, EB = eastbound, WB = westbound, SB = southbound, NB = northbound.

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 23-33 and Equation 23-34, respectively. Exhibit 34-10 presents the calculation of these downstream queues followed by the calculation of the respective lost time due to those queues.

Exhibit 34-10
Example Problem 1: Lost Time due to Downstream Queues

Value	Movement: EB EXT-TH	SB-L	WB EXT-TH	NB-L
Downstream Queue Calculations				
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 Calculations				
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

Notes: EXT = external, TH = through, L = left, EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

The lost time due to demand starvation is calculated by using Equation 23-38. The respective calculations are presented in Exhibit 34-11. As shown, in this case there is no lost time due to demand starvation (L_{DS} = 0).

Exhibit 34-11
Example Problem 1: Lost Time due to Demand Starvation

Value	Movement	
	EB-INT-TH	WB-INT-TH
$v_{Ramp\text{-}L}$ (veh/h)	206	233
$v_{Arterial}$ (veh/h)	868	886
C (s)	160	160
$N_{Ramp\text{-}L}$	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

Notes: EB-INT-TH = eastbound internal through, WB-INT-TH = westbound internal through.

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 31-154. Exhibit 34-12 and Exhibit 34-13 present the calculations of the queue storage ratio for all movements in Example 1. Those exhibits also show the volume-to-capacity (v/c) ratio for each movement. Control delay for each movement is calculated according to Equation 19-18. Exhibit 34-14 and Exhibit 34-15 provide the control delay for each movement of the interchange.

Exhibit 34-12
Example Problem 1: Queue Storage Ratio for Eastbound and Westbound Movements

Value	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_r (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

Notes: EXT = external, INT = internal, TH = through, R = right, L= left.

Exhibit 34-13
Example Problem 1: Queue Storage Ratio for Northbound and Southbound Movements

Value	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 34-14
Example Problem 1: Control Delay for Eastbound and Westbound Movements

Value	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

Notes: EXT = external, INT = internal, TH = through, R = right, L= left.

Exhibit 34-15
Example Problem 1: Control Delay for Northbound and Southbound Movements

Value	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)	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

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 23-2. 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 34-16 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 23-10. After extra distances are measured according to the Exhibit 23-8 discussion, EDTT can be obtained from Equation 23-50 [i.e., EDTT = 100 / (1.47 × 35) + 0 = 1.9 s/veh]. Intersection-wide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-16
Example Problem 1: O-D Movement LOS

O-D Movement	Control Delay (s/veh)	EDTT (s/veh)	ETT (s/veh)	$v/c > 1$?	$R_Q > 1$?	LOS
A	45.6	1.9	47.5	No	No	C
B	43.7	−1.9	41.8	No	No	C
C	54.6	−1.9	52.7	No	No	C
D	63.6	1.9	65.5	No	No	D
E	99.2	1.9	101.1	No	No	E
F	44.2	−1.9	42.3	No	No	C
G	37.5	−1.9	35.6	No	No	C
H	82.7	1.9	84.6	No	No	D
I	52.0	0.0	52.0	No	No	C
J	39.8	0.0	39.8	No	No	C

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 34-17 provides the interchange volumes and channelization, while Exhibit 34-18 provides the signalization information. The offset is referenced to the beginning of green on the EB direction of the arterial.

Exhibit 34-17
Example Problem 2:
Intersection Plan View

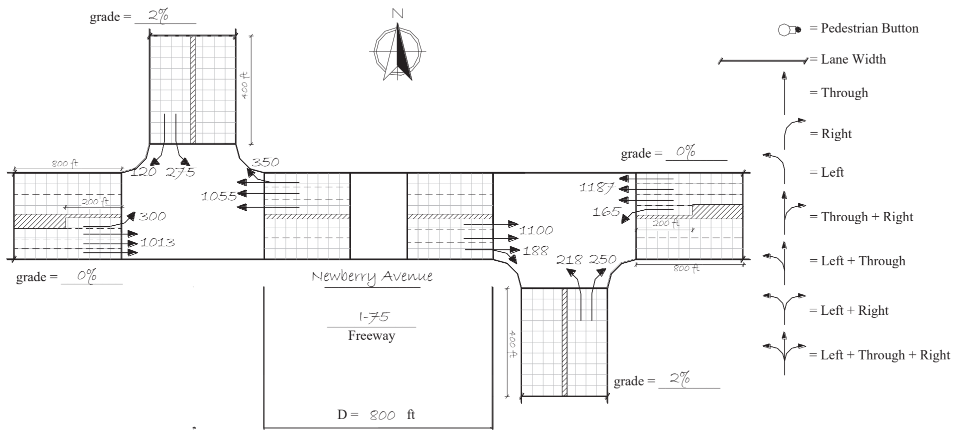


Exhibit 34-18
Example Problem 2:
Signalization Information

	Intersection I			Intersection II		
Phase	**1**	**2**	**3**	**1**	**2**	**3**
NEMA	Φ (2+5)	Φ (2+6)	Φ (4+7)	Φ (1+6)	Φ (3+8)	Φ (2+6)
Green time (s)	25	60	40	25	35	65
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		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. Extra distance traveled along each freeway loop ramp is 1,600 ft. The grade is 2% on the NB and SB approaches.

There are 11.7% 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 Origin–Destination Movements

O-Ds through this parclo interchange are calculated on the basis of the worksheet provided in Exhibit 34-163 in Section 4. 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 34-19.

Exhibit 34-19
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 23-24 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 34-20.

Exhibit 34-20
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 in the base saturation flow rate of 1,900 pc/hg/ln by using Equation 23-14. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-20. Traffic pressure is calculated by using Equation 23-15. The left- and right-turn adjustment factors are estimated by using Equations 23-20 through 23-23. These equations use an adjustment factor for travel path radius calculated by Equation 23-19. The remaining adjustment factors are calculated according to Chapter 19, Signalized Intersections. The results of these calculations for all approaches are presented in Exhibit 34-21 and Exhibit 34-22.

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

Value	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 and grade adjustment (f_{HVg})	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

Exhibit 34-22
Example Problem 2: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

Value	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 and grade adjustment (f_{HVg})	0.909	1.000	0.909	0.909	1.000	0.909
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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

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

Exhibit 34-23 provides the beginning and end times of the green for each phase followed by the beginning and end of green for the four pairs of movements at the two intersections. Phase 1 of the first intersection is assumed to begin at time zero (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 34-23
Example Problem 2: Common Green Calculations

Phase	Intersection I		Intersection II		
	Green Begin	Green End	Green Begin	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**		**Second Green Time Within Cycle**		**Common Green Time**
	Begin	**End**	**Begin**	**End**	
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			

Notes: EXT = external, INT = internal, EB = eastbound, WB = westbound, NB = northbound, SB = southbound, THRU = through.

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 23-33 and Equation 23-34, respectively. Exhibit 34-24 presents the calculation of these downstream queues followed by the calculation of the respective lost time due to those queues.

Exhibit 34-24
Example Problem 2: Lost Time due to Downstream Queues

Value	Movement			
	EB EXT-TH	**SB-L**	**WB EXT-TH**	**NB-L**
Downstream Queue Calculations				
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 Calculations				
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

Notes: EXT = external, TH = through, L = left, EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

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 31-154. Exhibit 34-25 and Exhibit 34-26 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 19-18. Exhibit 34-27 and Exhibit 34-28 provide the control delay for each movement of this interchange.

Exhibit 34-25
Example Problem 2: Queue Storage Ratio for Eastbound and Westbound Movements

Value	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-26
Example Problem 2: Queue Storage Ratio for Northbound and Southbound Movements

Value	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements 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_r (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

Exhibit 34-27
Example Problem 2: Control Delay for Eastbound and Westbound Movements

Value	Eastbound Movements EXT-TH	Eastbound Movements EXT-L	Eastbound Movements INT-TH&R	Westbound Movements EXT-TH	Westbound Movements EXT-L	Westbound Movements 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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-28
Example Problem 2: Control Delay for Northbound and Southbound Movements

Value	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements 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

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 23-2. 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 34-29 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 23-10. After extra distances are measured according to the Exhibit 23-9 discussion, EDTT can be obtained from Equation 23-50 [i.e., EDTT = 1,200 / (1.47 × 25) + 5 = 37.7 s/veh]. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-29
Example Problem 2: O-D Movement LOS

O-D Movement	Control Delay (s/veh)	EDTT (s/veh)	ETT (s/veh)	$v/c > 1$?	$R_Q > 1$?	LOS
A	78.9	20.6	99.5	No	No	E
B	55.7	−15.6	40.1	No	No	C
C	41.1	−15.6	25.5	No	No	B
D	70.0	20.6	90.6	No	No	E
E	33.8	37.7	71.5	No	No	D
F	115.7	20.6	136.3	Yes	Yes	F
G	61.6	20.6	82.2	No	No	D
H	40.0	37.7	77.7	No	No	D
I	33.8	0.0	33.8	No	No	C
J	40.0	0.0	40.0	No	No	C

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 34-30 and Exhibit 34-31. The offset is referenced to the beginning of green on the EB direction of the arterial.

Exhibit 34-30
Example Problem 3:
Intersection Plan View

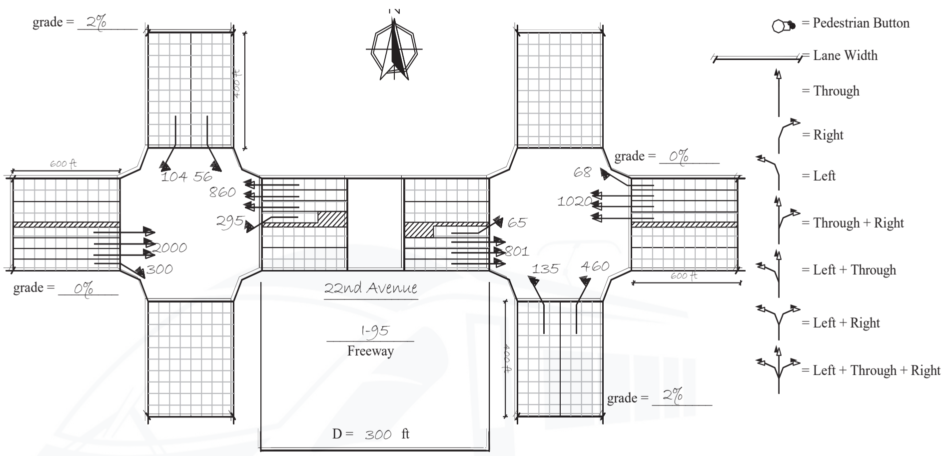


Exhibit 34-31
Example Problem 3:
Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (4+7)	Φ (2+6)	Φ (1+6)	Φ (2+5)	Φ (2+6)	Φ (3+8)
Green time (s)	27	59	19	27	39	39
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		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 movements, which have radii of 75 ft. Extra distance traveled along each freeway ramp is 60 ft. The grade is 2% on the NB and SB approaches.

There are 6.1% 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.

Solution

Calculation of Origin–Destination Movements

O-Ds through this diamond interchange are calculated on the basis of the worksheet provided in Exhibit 34-169 in Section 4. 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 34-32.

Exhibit 34-32
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 23-24. Since there is an exclusive right-turn lane for both external approaches, according to the first note of Exhibit 23-24 the lane utilization for Lane 3 should be estimated by assuming that the right-turning O-D (v_F, v_G) is zero. Exhibit 34-33 presents the calculation results and the lane utilization factor for each approach.

Exhibit 34-33
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

Notes: EB = eastbound, WB = westbound.

Saturation flow rates are calculated on the basis of reductions in the base saturation flow rate of 1,900 pc/hg/ln by using Equation 23-14. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-6. Traffic pressure is calculated by using Equation 23-15. The left- and right-turn adjustment factors are estimated by using Equations 23-20 through 23-23. These equations use an adjustment factor for travel path radius calculated by Equation 23-19. The remaining adjustment factors are calculated as indicated in Chapter 19, Signalized Intersections. The results of these calculations for all approaches are presented in Exhibit 34-34 and Exhibit 34-35.

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

Value	Eastbound EXT-TH	Eastbound EXT-R	Eastbound INT-TH	Eastbound INT-L	Westbound EXT-TH	Westbound EXT-R	Westbound INT-TH	Westbound 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 and grade adjustment (f_{HVg})	0.952	1.000	0.952	1.000	0.952	1.000	0.952	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-35
Example Problem 3: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

Value	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})	0.899	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 34-36 first provides the beginning and ending of the green time for each phase at the two intersections, on the assumption that Phase 1 of the first intersection begins at time zero. In this case, the offset for both intersections is zero; therefore, the beginning of Phase 1 for the second intersection is also zero.

Exhibit 34-36
Example Problem 3: Common Green Calculations

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			

Notes: EXT = external, INT = internal, EB = eastbound, WB = westbound, NB = northbound, SB = southbound, THRU = through.

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 23-33 and Equation 23-34, respectively. Exhibit 34-37 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 23-38. The respective calculations are presented in Exhibit 34-38. As shown, in this case there is no lost time due to demand starvation.

Exhibit 34-37
Example Problem 3: Lost Time due to Downstream Queues

Value	Movement: EB EXT-TH	SB-L	WB EXT-TH	NB-L
Downstream Queue Calculations				
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 Calculations				
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, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (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

Notes: EXT = external, TH = through, L = left, EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

Exhibit 34-38
Example Problem 3: Lost Time due to Demand Starvation Calculations

Value	Movement: EB-INT-TH	Movement: WB-INT-TH
$v_{Ramp\text{-}L}$ (veh/h)	58	139
$v_{Arterial}$ (veh/h)	2,062	1,052
C (s)	120	120
$N_{Ramp\text{-}L}$	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

Notes: EB-INT-TH = eastbound internal through, WB-INT-TH = westbound internal through.

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 31-154. Exhibit 34-39 and Exhibit 34-40 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 19-18. Exhibit 34-41 and Exhibit 34-42 provide the control delay for each movement of the interchange.

Exhibit 34-39
Example Problem 3: Queue Storage Ratio for Eastbound and Westbound Movements

	Eastbound Movements				Westbound Movements			
Value	**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_f (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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-40
Example Problem 3: Queue Storage Ratio for Northbound and Southbound Movements

Value	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements 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_r (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 34-41
Example Problem 3: Control Delay for Eastbound and Westbound Movements

Value	Eastbound Movements EXT-TH	Eastbound EXT-R	Eastbound INT-L	Eastbound INT-TH	Westbound Movements EXT-TH	Westbound EXT-R	Westbound INT-L	Westbound 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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-42
Example Problem 3: Control Delay for Northbound and Southbound Movements

Value	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements 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 23-2. 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 34-43 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 23-10. After extra distances are measured according to the Exhibit 23-8 discussion, EDTT can be obtained from Equation 23-50 [i.e., EDTT = 60 / (1.47 × 35) + 0 = 1.2 s/veh]. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-43
Example Problem 3: O-D Movement LOS

O-D Movement	Control Delay (s/veh)	EDTT (s/veh)	ETT (s/veh)	$v/c > 1$?	$R_Q > 1$?	LOS
A	32.7	1.2	33.9	No	No	C
B	53.6	−1.2	52.4	No	No	C
C	40.7	−1.2	39.5	No	No	C
D	49.3	1.2	50.5	No	No	C
E	178.6	1.2	179.8	Yes	Yes	F
F	20.3	−1.2	19.1	No	No	B
G	29.1	−1.2	27.9	No	No	B
H	157.0	1.2	158.2	Yes	Yes	F
I	146.8	0.0	146.8	Yes	Yes	F
J	44.0	0.0	44.0	No	No	C

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 34-44 and Exhibit 34-45. The offset is referenced to the beginning of green on the EB direction of the arterial.

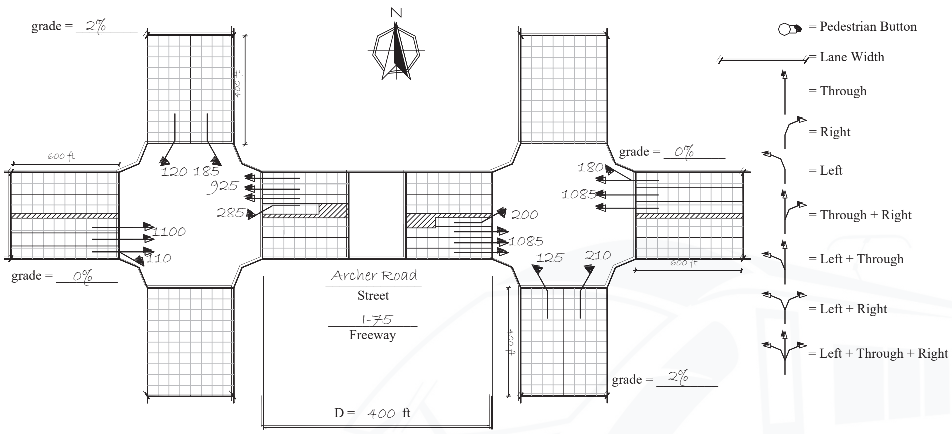


Exhibit 34-44
Example Problem 4: Intersection Plan View

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

Exhibit 34-45
Example Problem 4: Signalization Information

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. Extra distance traveled along each freeway ramp is 100 ft.

There are 6.1% 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. The grade is 2% on the NB and SB approaches.

Solution

Calculation of Origin–Destination Movements

O-Ds through this diamond interchange are calculated by using the worksheet given in Exhibit 34-169 in Section 4. 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 PHF-adjusted values are presented in Exhibit 34-46.

Exhibit 34-46
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 23-24 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 34-47.

Exhibit 34-47
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

Notes: EB = eastbound, WB = westbound.

Saturation flow rates are calculated on the basis of reductions in the base saturation flow rate of 1,900 pc/hg/ln by using Equation 23-14. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 34-6. Traffic pressure is calculated by using Equation 23-15. The left- and right-turn adjustment factors are estimated by using Equations 23-20 through 23-23. These equations use an adjustment factor for travel path radius calculated by Equation 23-19. The remaining adjustment factors are calculated as indicated in Chapter 19, Signalized Intersections. The results of the saturation flow rate calculations for all approaches are presented in Exhibit 34-48 and Exhibit 34-49.

Exhibit 34-48
Example Problem 4: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

Value	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 and grade adjustment (f_{HVg})	0.952	0.952	1.000	0.952	0.952	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-49
Example Problem 4: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

Value	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 and grade adjustment (f_{HVg})	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

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

Exhibit 34-50 presents the beginning and end times of the green for each phase at the two intersections. Phase 1 of the first intersection is assumed to begin at time zero. In this case the offset for both intersections is zero; therefore the beginning of Phase 1 for the second intersection is also zero.

Exhibit 34-50
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			

Notes: EXT = external, INT = internal, EB = eastbound, WB = westbound, NB = northbound, SB = southbound, THRU = through.

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 23-33 and Equation 23-34, respectively. Exhibit 34-51 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.

Exhibit 34-51
Example Problem 4: Lost Time due to Downstream Queues

Value	Movement: EB EXT-TH	SB-L	WB EXT-TH	NB-L
Downstream Queue Calculations				
V_R or V_A (veh/h)	191	1,134	129	1,119
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 Calculations				
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

Notes: EXT = external, EB = eastbound, WB = westbound, NB = northbound, SB = southbound, TH = through, L = left.

The lost time due to demand starvation is calculated by using Equation 23-38. The respective calculations are presented in Exhibit 34-52. As shown, both internal through movements experience lost time due to demand starvation.

Exhibit 34-52
Example Problem 4: Lost Time due to Demand Starvation Calculations

Value	Movement	
	EB-INT-TH	**WB-INT-TH**
$v_{Ramp\text{-}L}$ (veh/h)	191	129
$v_{Arterial}$ (veh/h)	1,134	1,119
C (s)	100	100
$N_{Ramp\text{-}L}$	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

Notes: EB-INT-TH = eastbound internal through, WB-INT-TH = westbound internal through.

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 31-154. Exhibit 34-53 and Exhibit 34-54 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 19-18. Exhibit 34-55 and Exhibit 34-56 provide the control delay for each movement of the interchange.

Exhibit 34-53
Example Problem 4: Queue Storage Ratio for Eastbound and Westbound Movements

Value	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-54
Example Problem 4: Queue Storage Ratio for Northbound and Southbound Movements

Value	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_r (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 34-55
Example Problem 4: Control Delay for Eastbound and Westbound Movements

Value	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-56
Example Problem 4: Control Delay for Northbound and Southbound Movements

Value	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements 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

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 23-2. 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 34-57 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 23-10 for these movements. After extra distances are measured according to the Exhibit 23-8 discussion, EDTT can be obtained from Equation 23-50 [i.e., EDTT = 80 / (1.47 × 35) + 0 = 1.6 s/veh]. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-57
Example Problem 4: O-D Movement LOS

O-D Movement	Control Delay (s/veh)	EDTT (s/veh)	ETT (s/veh)	*v/c* > 1?	R_Q > 1?	LOS
A	43.9	1.6	45.5	No	No	C
B	31.2	−1.6	29.6	No	No	B
C	27.8	−1.6	26.2	No	No	B
D	43.6	1.6	45.2	No	No	C
E	119.9	1.6	121.5	Yes	No	F
F	87.6	−1.6	86.0	Yes	No	F
G	57.9	−1.6	56.3	No	No	D
H	88.0	1.6	89.6	No	No	E
I	101.1	0.0	101.1	Yes	No	F
J	73.9	0.0	73.9	No	No	D

EXAMPLE PROBLEM 5: DIVERGING DIAMOND INTERCHANGE WITH SIGNAL CONTROL

The Interchange

The interchange of Main Street at Interstate I-40 is a diverging diamond interchange (DDI) with signalized right turns and left turns controlling movements from the freeway onto the Main Street arterial. The turning movements onto the freeway from Main Street are not signalized. The traffic, geometric, and signalization conditions of the interchange are provided in Exhibit 34-58 and Exhibit 34-59.

Exhibit 34-58
Example Problem 5: DDI Geometry, Lane, and Volume Inputs

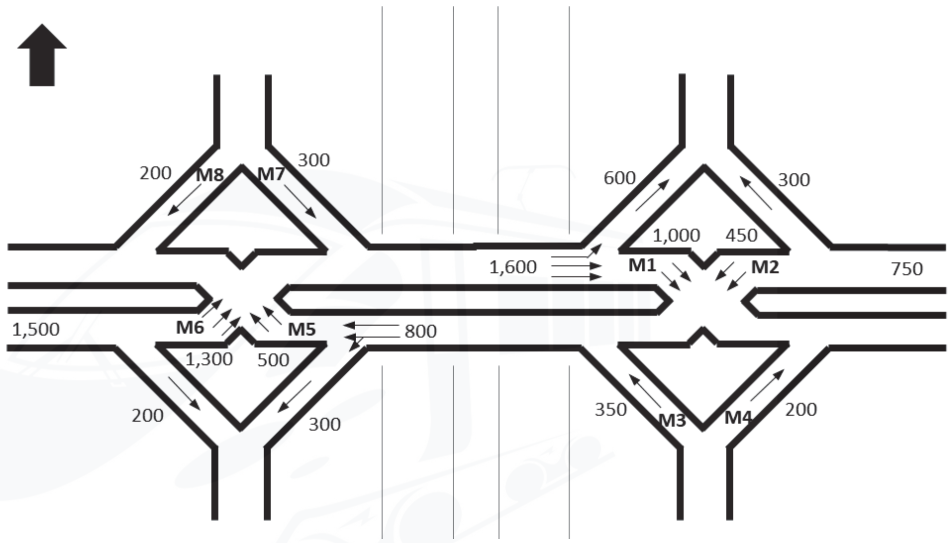


Exhibit 34-58 shows movement numbers M1 through M8, their associated volume levels (in vehicles per hour), and the number of lanes for each movement approach. Note that the eastbound movement has an exclusive left-turn lane onto the freeway between crossovers, which is carried through the external crossover at Movement M6. For the westbound movement, the left turn onto the freeway is made from a shared lane, which is expected to affect the lane utilization of Movement M2.

The Question

What are the control delays, experienced travel time, 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 right-turn movements and 150 ft for left turns.

There are 6.1% heavy vehicles for all 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.

Exhibit 34-59 provides basic signal timing information for the DDI. The cycle length is set at 70 s for this pretimed signal. The arterial street free-flow speed is 35 mi/h.

Exhibit 34-59
Example Problem 5: Signal Timing and Volume Inputs

	West Crossover				East Crossover			
Movement	**M5**	**M6**	**M7**	**M8**	**M1**	**M2**	**M3**	**M4**
Green time (s)	25	35	25	35	35	25	35	25
Yellow time (s)	4	4	4	4	4	4	4	4
All-red time (s)	1	1	1	1	1	1	1	1
Phase split (s)	30	40	30	40	40	30	40	30
Turn radius (ft)			150	75			150	75
Width of clear zone (ft)			200	100			200	100
Shortest distance, stop bar to conflict point (ft)			20	60			20	60
Volume (veh/h)	500	1,300	300	200	1,000	450	350	200

The DDI is timed with two critical phases to allow the northbound and southbound through movements to be processed through the interchange sequentially. The signalized right-turn movements from the freeway move concurrently with the inbound through movement into the interchange at each crossover, and the left turns move concurrently with the outbound through movements. Overlap phasing is used to reduce the lost time for the through movement while providing adequate clearance times for the turning traffic. In the methodology, this results in additional lost time applied to the ramp movements (Step 4 of DDI methodology in Chapter 23).

Solution

Calculation of Origin–Destination Movements

O-D movements through this diamond interchange are calculated by using the worksheet in Exhibit 34-169 in Section 4. Because 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 34-60.

Exhibit 34-60
Example Problem 5: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	350	368
B	200	211
C	200	211
D	300	316
E	600	632
F	200	211
G	300	316
H	300	316
I	700	737
J	150	158
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

Lane utilization for DDIs is calculated by using Exhibit 23-26 for the two external approaches to the DDI. The eastbound movement has an exclusive left-turn lane onto the freeway between crossovers, which is carried through the external crossover at Movement M6. For the westbound movement, the left turn onto the freeway is made from a shared lane, which is expected to affect lane utilization at Movement M2. The calculated maximum lane utilization and associated lane utilization factors are shown in Exhibit 34-61.

Exhibit 34-61
Example Problem 5: Lane Utilization Adjustment Calculations

Approach	Lane Configuration	Left-Turn Demand Ratio	Maximum Lane Utilization	Lane Utilization Factor
Eastbound external	3-lane exclusive	0.46	0.45	0.74
Westbound external	2-lane shared	0.67	0.77	0.65

Saturation flow rates are calculated on the basis of reductions in the base saturation flow rate of 1,900 pc/hg/ln by using Equation 23-14. The lane utilization of the approaches external to the interchange is obtained as shown above. Traffic pressure is calculated by using Equation 23-15. The left- and right-turn adjustment factors are estimated by using Equation 23-20 through Equation 23-23. These equations use an adjustment factor for travel path radius calculated by Equation 23-19. The DDI adjustment factor is applied to the internal and external through movements at both crossovers. The remaining adjustment factors are calculated as indicated in Chapter 19, Signalized Intersections. The estimated saturation flow rates for all approaches are shown in Exhibit 34-62.

Exhibit 34-62
Example Problem 5: Saturation Flow Rate Calculation for All Approaches

	West Crossover				East Crossover			
Value	M5	M6	M7	M8	M1	M2	M3	M4
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)	2	3	1	1	2	2	1	1
Lane width adjustment (f_w)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Heavy vehicle and grade adjustment (f_{HVg})	0.952	0.952	0.952	0.952	0.952	0.952	0.952	0.952
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	0.740	1.000	1.000	1.000	0.649	1.000	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.964	1.000	1.000	1.000	0.964	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	1.000	0.930	1.000	1.000	1.000	0.930
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	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Traffic pressure adjustment for lane group (f_v)	0.956	0.972	0.960	0.951	0.978	0.954	0.964	0.951
DDI adjustment factor (f_{DDI})	0.913	0.913	1.000	1.000	0.913	0.913	1.000	1.000
Adjusted saturation flow per lane (s, veh/hg/ln)	1,578	1,188	1,674	1,601	1,615	1,022	1,682	1,601
Adjusted approach saturation flow (s, veh/hg)	3,156	3,563	1,674	1,601	3,229	2,045	1,682	1,601

Effective Green and Lost Time Calculations

Next, effective green time adjustments for the DDI movements are calculated according to Step 4 of the DDI methodology, as shown in Exhibit 34-63. The lost time adjustment due to internal queues was illustrated in previous examples and is assumed to be 4 s/veh for this example. Lost time due to demand starvation does not apply to DDIs and is set at zero. Lost time due to overlap phasing for the DDI ramp movements is calculated from Equation 23-37.

Exhibit 34-63
Example Problem 5:
Lost Time and Effective Green Calculations

	West Crossover				East Crossover			
Value	**M5**	**M6**	**M7**	**M8**	**M1**	**M2**	**M3**	**M4**
Lost time due to internal queues (s)	0	4	4	0	0	4	4	0
Lost time due to demand starvation (s)	0	0	0	0	0	0	0	0
Lost time on DDI ramps from overlap phasing (s)	0	0	6.5	4.9	0	0	6.5	4.9
Start-up lost time (s)	2	2	2	2	2	2	2	2
Extension of effective green (s)	2	2	2	2	2	2	2	2
Adjusted lost time, external (s)		8	15	9		9	15	9
Adjusted lost time, internal (s)	4				4			
Effective green time (s)	25	31	14	30	35	20	24	20

Results

With the effective green time and saturation flow adjustments complete, the volume-to-capacity ratios for each lane group are calculated from Equation 23-48. Because this is an isolated DDI, no adjustments due to closely spaced intersections apply. Because all turning movements from the freeway are signalized, Step 6 for estimating performance of YIELD-controlled turns also does not apply. The results are shown in Exhibit 34-64.

Control delay and its various components (uniform delay, incremental delay, and initial queue delay) are calculated by using the procedures in Chapter 19, and the results are shown in Exhibit 34-64.

Exhibit 34-64
Example Problem 5:
Performance Results

	West Crossover				East Crossover			
Value	**M5**	**M6**	**M7**	**M8**	**M1**	**M2**	**M3**	**M4**
Demand flow rate, lane group (veh/h)	500	1,300	300	200	1,000	450	350	200
Saturation flow rate, lane group (veh/h)	3,156	3,563	1,674	1,601	3,229	2,045	1,682	1,601
Effective green time (s)	25	31	14	30	35	20	24	20
Cycle length (s)	70	70	70	70	70	70	70	70
g/C ratio	0.36	0.44	0.21	0.43	0.50	0.29	0.35	0.29
v/c ratio for lane group	0.44	0.82	0.87	0.29	0.62	0.77	0.60	0.44
Uniform delay (s/veh)	16.0	17.6	26.8	13.2	21.7	25.4	19.1	22.3
Incremental delay (s/veh)	1.2	5.2	25.7	0.1	0.2	23	1.9	0.6
Initial queue delay (s/veh)	0	0	0	0	0	0	0	0
Control delay (s/veh)	17.2	22.8	52.5	13.3	21.9	48.4	21.0	22.9

From these results, the performance measures are aggregated for each O-D movement. The naming convention for converting turning movements to O-Ds is followed. Furthermore, for each O-D movement, the EDTT is calculated with Equation 23-50. The LOS for each lane group can then be determined from Exhibit 23-10. The results of all steps are shown in Exhibit 34-65.

In the exhibit, the extra distance traveled is 100 ft for the left turn from the freeway (Movements A and D), reflecting some out-of-direction travel distance at the interchange. Similarly, 40 ft of added travel distance is applied to the arterial through movements (I and J) to account for the two crossover shifts. For an actual site, these distances should be measured from design drawings or aerial images. The EDTT is then calculated on the assumption of a travel speed of 35 mi/h for that added distance. Note that the methodology does not consider delays for the free-flow right-turn bypass movements onto the freeway, which are therefore assumed to be zero. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-65
Example Problem 5: ETT and LOS Results

O-D	PHF-Adjusted Demand (veh/h)	Movement	Control Delay Components	Total Control Delay (s/veh)	Extra Dis-tance (ft)	EDTT (s/veh)	ETT (s/veh)	LOS
A	368	NB L	M3 + M5	38.2	100	1.9	40.1	C
B	211	NB R	M4	22.9	−100	−1.9	21.0	B
C	211	SB R	M8	13.3	−100	−1.9	11.4	A
D	316	SB L	M7 + M1	74.4	100	1.9	76.3	D
E	632	EB L	M6	22.8	100	1.9	24.7	B
F	211	EB R	N/A	0.0	0	0.0	0.0	A
G	316	WB R	N/A	0.0	0	0.0	0.0	A
H	316	WB L	M2	48.4	100	1.9	50.3	C
I	737	EB T	M6 + M1	44.7	40	0.8	45.5	C
J	158	WB T	M2 + M5	65.6	40	0.8	66.4	D

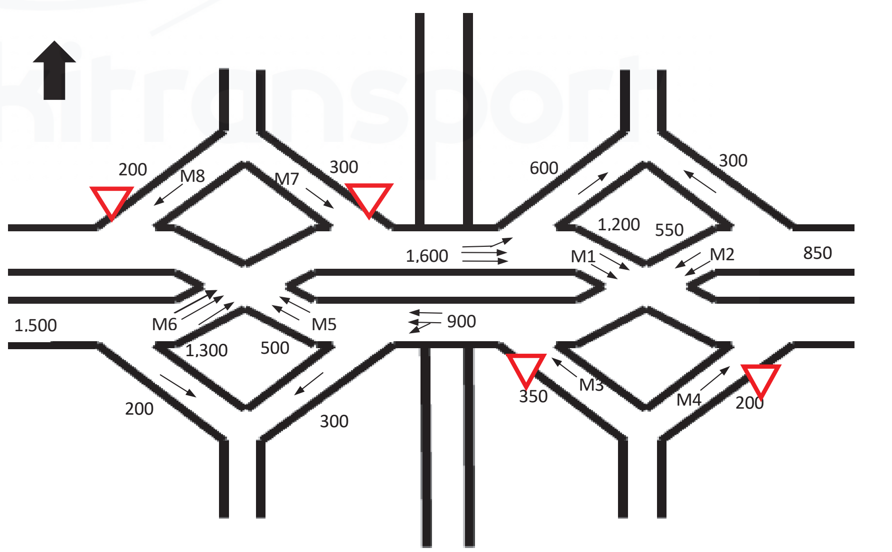
Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, L = left, R = right, T = through, N/A = not applicable.

EXAMPLE PROBLEM 6: DIVERGING DIAMOND INTERCHANGE WITH YIELD CONTROL

The Interchange

In this example, the same DDI is used that was introduced in Example Problem 5. The only difference is that the left turns (M3 and M7) and right turns (M4 and M8) from the freeway off-ramps are now YIELD-controlled movements. The estimation of control delays for Movements M1, M2, M5, and M6 is unchanged from the previous example. The geometry is shown in Exhibit 34-66.

Exhibit 34-66
Example Problem 6: Geometry, Lane, and Volume Inputs



The Question

What are the control delays, experienced travel time, and LOS for the turning movements off the freeway for this interchange if they are controlled by YIELD signs?

The Facts

The basic assumptions for this freeway are the same as for Example Problem 5. Similarly, Steps 1 through 5 are unchanged for the signalized movements.

Solution

Capacity of YIELD-Controlled Movement

Step 6 of the interchange methodology evaluates the capacity of the YIELD-controlled movement in three regimes: (*a*) Regime 1—blocked by conflicting platoon when the conflicting signal has just turned green, with zero capacity for turning movement; (*b*) Regime 2—gap acceptance in conflicting traffic after the initial platoon has cleared, with gap acceptance controlled by the critical gap, follow-up time, and conflicting flow rate; and (*c*) Regime 3—no conflicting flow when the conflicting signal is red, with full capacity, controlled by the follow-up time of the YIELD-controlled approach.

For each regime, the methodology computes the proportion of time the regime is active, as well as the capacity that applies over that period of time. The evaluation is performed for the two right-turn movements (M4 and M8) and the two left-turn movements (M3 and M7).

In Regime 1, the capacity is equal to zero, since no YIELD-controlled movements can enter the interchange while the opposing queue clears. The duration of the blocked period is estimated from Equation 23-51. For an isolated interchange, Equation 23-52 and Equation 23-54 are used to estimate the time to clear the opposing queue and the time for the last queued vehicle to clear the conflict point, respectively. The calculation results are shown in Exhibit 34-67.

Exhibit 34-67
Example Problem 6: Capacity of Blocked Regime

Value	M7	M8	M3	M4
Green time for opposing movement (s)	31	25	20	35
Red time for opposing movement (s)	39	45	50	35
Volume of opposing movement per lane (veh/h/ln)	433	250	225	500
Saturation flow rate for opposing movement (veh/h)	1,188	1,578	1,022	1,615
Time to clear queue, t_{CQ} (s)	22.4	8.5	14.1	15.7
Distance to clear, x_{clear} (ft)	200.0	100.0	200.0	100.0
Speed of opposing movement (mi/h)	25.0	25.0	25.0	25.0
Time to clear last vehicle, t_{clear} (s)	5.5	2.7	5.5	2.7
Proportion of time blocked, p_b	0.40	0.16	0.28	0.26
Capacity of blocked period, c_b (veh/h)	0	0	0	0

In Regime 2, the capacity of the YIELD-controlled movement when gaps are accepted in opposing traffic is estimated by using Equation 23-42. The proportion of time for that gap acceptance regime is estimated from Equation 23-43. The computation results are shown in Exhibit 34-68. Note that in the exhibit, the p_{GA} time calculated for M3 was originally negative and therefore was set to zero.

Exhibit 34-68
Example Problem 6: Capacity of Gap Acceptance Regime

Value	M7	M8	M3	M4
Critical gap, t_c (s)	3.9	1.8	3.9	1.8
Follow-up time, t_f (s)	2.6	2.4	2.6	2.4
Conflicting flow rate, q_c (veh/h)	1,300	500	450	1,200
Capacity of gap acceptance regime, c_{GA} (veh/h)	541	1,380	1,000	1,228
Proportion of time of gap acceptance, p_{GA}	0.04	0.20	0.00[a]	0.24

Note: [a] Set to zero to avoid negative numbers.

In Regime 3, conflicting flow is stopped at the crossover signal, and the capacity is estimated from Equation 23-44. The proportion of time for this regime is estimated from Equation 23-45. The results are shown in Exhibit 34-69.

Exhibit 34-69
Example Problem 6: Capacity of No-Opposing-Flow Regime

Value	M7	M8	M3	M4
Capacity of no-opposing-flow regime, c_{NOF} (veh/h)	1,385	1,500	1,385	1,500
Proportion of time with no opposing flow, p_{NOF}	0.56	0.64	0.71	0.50

Results

The combined capacity of the YIELD-controlled movement is estimated from Equation 23-46 or Equation 23-47. With that capacity and the movement demand, a volume-to-capacity ratio can be estimated. The control delay for the movement is then estimated by using the control delay procedure for roundabouts given in Equation 22-17. The computations of other terms contributing to the experienced travel time service measure are consistent with Example Problem 5. The results are shown in Exhibit 34-70.

Exhibit 34-70
Example Problem 6: Performance Results

Value	M7	M8	M3	M4
Demand flow rate for lane group (veh/h)	300	200	350	200
v/c ratio for lane group (decimal)	0.38	0.16	0.35	0.19
Control delay (s/veh)	34.7	13.4	31.0	16.3

The results suggest that under these assumptions, YIELD-controlled left-turn Movements M7 and M4 perform better than the signalized alternatives evaluated in Example Problem 5, while unsignalized right-turn Movements M8 and M3 show slightly higher delay than with the signal.

From these results, the performance measures are aggregated for each O-D movement. The naming convention for converting turning movements to O-Ds is followed. Furthermore, for each O-D movement, the EDTT is calculated with Equation 23-50. From the O-D ETT, the LOS for each lane group is estimated from Exhibit 23-10. The results of all steps are shown in Exhibit 34-71.

In the exhibit, the extra distance traveled is 100 ft for the left turn from the freeway (Movements A and D), reflecting some out-of-direction travel distance at the interchange. For right turns from the freeway (Movements B and C), an equivalent negative extra travel distance is applied. Similarly, 40 ft of added travel distance is applied to the arterial through movements (I and J) to account for the two crossover shifts. For an actual site, these distances should be measured from design drawings or aerial images. The EDTT is then calculated on the assumption of a travel speed of 35 mi/h for that added distance. Note that the methodology does not consider delays for the free-flow right-turn bypass movements onto the freeway, which are therefore assumed to be zero.

Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-71
Example Problem 6: ETT and LOS Results

O-D	PHF-Adjusted Demand (veh/h)	Movement	Control Delay Components	Total Control Delay (s/veh)	Extra Dis-tance (ft)	EDTT (s/veh)	ETT (s/veh)	LOS
A	368	NB L	M3 + M5	48.2	100	1.9	50.2	D
B	211	NB R	M4	16.3	−100	−1.9	14.3	A
C	211	SB R	M8	13.4	−100	−1.9	11.5	A
D	316	SB L	M7 + M1	56.6	100	1.9	58.5	D
E	632	EB L	M6	22.8	100	1.9	24.7	B
F	211	EB R	N/A	0.0	0	0.0	0.0	A
G	316	WB R	N/A	0.0	0	0.0	0.0	A
H	316	WB L	M2	48.4	100	1.9	50.3	C
I	737	EB T	M6 + M1	44.7	40	0.8	45.5	C
J	158	WB T	M2 + M5	65.6	40	0.8	66.4	D

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, L = left, R = right, T = through, N/A = not applicable.

EXAMPLE PROBLEM 7: 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 34-72 and Exhibit 34-73.

Exhibit 34-72
Example Problem 7: Intersection Plan View

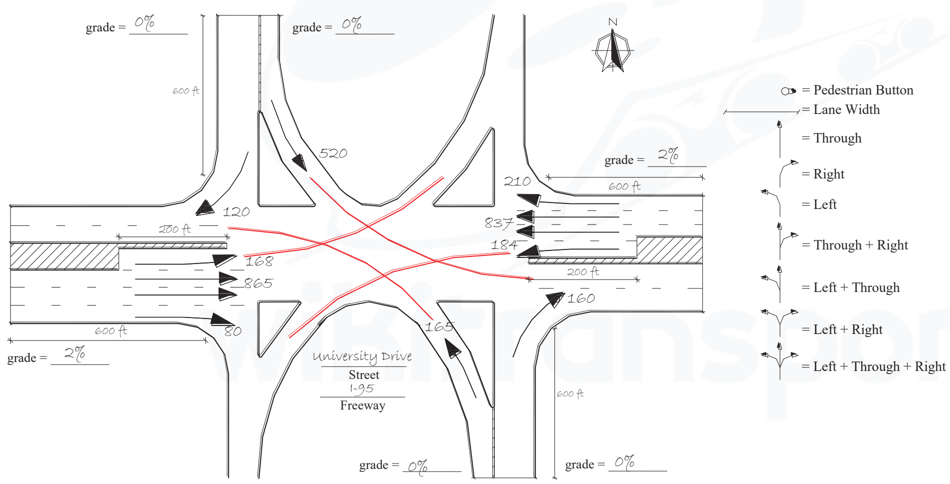


Exhibit 34-73
Example Problem 7: Signalization Information

Phase	SPUI Interchange 1	2	3
NEMA	Φ (1+5+4R+8R)	Φ (2+6)	Φ (3+8+2R+6R)
Green time (s)	16	32	38
Yellow + all red (s)	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 87 ft and 50 ft for all left-turn and right-turn movements, respectively. Lane widths are 10.3 ft for all lanes. There is no extra distance traveled along the freeway ramps. The grade is 2% on the eastbound and westbound approaches.

There are 3.4% heavy vehicles on all eastbound and westbound movements. There are 5% heavy vehicles on all northbound and southbound movements. 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 Origin–Destination Movements

O-Ds through this SPUI are calculated on the basis of the worksheet provided in Exhibit 34-170. 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 34-74.

Exhibit 34-74
Example Problem 7: 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 in the base saturation flow rate of 1,900 pc/hg/ln by using Equation 23-14. Traffic pressure is calculated by using Equation 23-15. The left- and right-turn adjustment factors are estimated by using Equation 23-20 through Equation 23-23. These equations use an adjustment factor for travel path radius calculated by Equation 23-19. The remaining adjustment factors are calculated as indicated in Chapter 19, Signalized Intersections. The results of the saturation flow rate calculations for all approaches are presented in Exhibit 34-75 and Exhibit 34-76.

Exhibit 34-75
Example Problem 7: Saturation Flow Rate Calculation for Eastbound and Westbound Approaches

Value	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 and grade adjustment (f_{HVg})	0.961	0.961	0.961	0.961	0.961	0.961	0.961	0.961
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 34-76
Example Problem 7: Saturation Flow Rate Calculation for Northbound and Southbound Approaches

Value	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 and grade adjustment (f_{HVg})	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 19, Signalized Intersections, and are shown in Exhibit 34-77.

Exhibit 34-77
Example Problem 7: Uniform Delay Calculations for Left Turns Featuring Both Permissive and Protected Phasing

Value	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 31-154. Exhibit 34-78 and Exhibit 34-79 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 19-18. Exhibit 34-80 and Exhibit 34-81 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 34-78
Example Problem 7: Queue Storage Ratio for Eastbound and Westbound Movements

Value	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_r (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 34-79
Example Problem 7: Queue Storage Ratio for Northbound and Southbound Movements

Value	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_r (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 34-80
Example Problem 7: Control Delay for Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
Value	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

Exhibit 34-81
Example Problem 7: Control Delay for Northbound and Southbound Movements

	Northbound Movements			Southbound Movements		
Value	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

Results

Delay for each O-D is estimated as the movement delay for the corresponding movement, as shown in Exhibit 34-82. 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. As shown, no movements have a *v/c* ratio or R_Q exceeding 1, and therefore the LOS result is based on the second column of Exhibit 23-10. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-82
Example Problem 7: O-D Movement LOS

O-D Movement	ETT (s/veh)	*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

EXAMPLE PROBLEM 8: 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 34-83 and Exhibit 34-84. The offset is referenced to the beginning of the green for the respective EB arterial approach.

Exhibit 34-83
Example Problem 8: Intersection Plan View

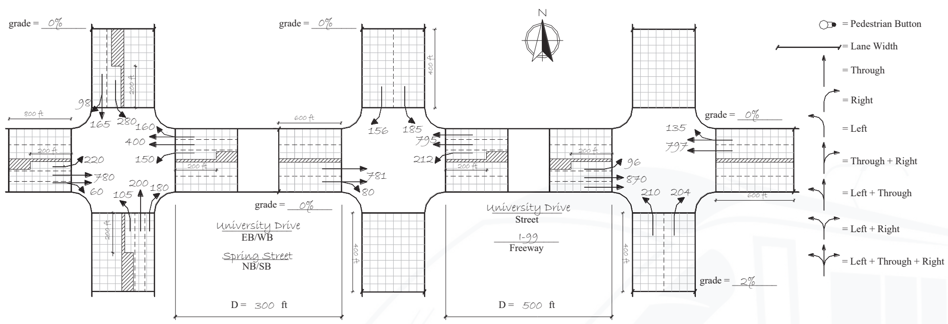


Exhibit 34-84
Example Problem 8: Signalization Information

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

Phase	Adjacent Intersection 1	2	3	4
NEMA	Φ (1+5)	Φ (2+6)	Φ (4+7)	Φ (3+8)
Green time (s)	33	59	24	24
Yellow + all red (s)	5	5	5	5
Offset (s)		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. Extra distance traveled along each freeway ramp is 100 ft.

There are 6.1% 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. The grade is 2% on the northbound approach.

Solution

Calculation of Origin–Destination Movements

The O-Ds for the interchange are obtained as explained in Example Problem 1 and were presented in Exhibit 34-5.

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 at the interchange) and the 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 19-15 and subtracting 0.05. The resulting lane utilization factors are shown in Exhibit 34-85.

Exhibit 34-85
Example Problem 8: Lane Utilization Adjustment Calculations

Lane Group	Lane Utilization Factor
2-lane group eastbound (inbound)	0.902
2-lane group westbound (outbound)	0.902

Saturation flow rates are calculated on the basis of reductions in the base saturation flow rate of 1,900 pc/h/ln by using Equation 23-14. The saturation flows for each lane group of the adjacent intersection are estimated according to Chapter 19, Signalized Intersections. The results of the saturation flow rate calculations for all movements of the adjacent intersection and the interchange are presented in Exhibit 34-86 through Exhibit 34-88. Note that turn radius and traffic pressure adjustments are not considered in the adjacent intersection.

Exhibit 34-86
Example Problem 8: Saturation Flow Rate Calculation for Interchange Eastbound and Westbound Approaches

	Eastbound			Westbound		
Value	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 and grade adjustment (f_{HVg})	0.952	0.952	1.000	0.952	0.952	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

Notes: EXT = external, INT = internal, TH = through, R = right, L = left.

Exhibit 34-87
Example Problem 8: Saturation Flow Rate Calculation for Interchange Northbound and Southbound Approaches

Value	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 and grade adjustment (f_{HVg})	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 34-88
Example Problem 8: Saturation Flow Rate Calculation for Adjacent Intersection

Value	Eastbound TH&R	Eastbound L	Westbound TH&R	Westbound L	Northbound TH	Northbound R	Northbound L	Southbound TH&R	Southbound 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	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 and grade adjustment (f_{HVg})	0.952	0.952	0.952	0.952	0.952	0.952	0.952	0.952	0.952
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

Notes: TH = through, R = right, L = left.

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 34-89 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 34-89
Example Problem 8: 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			

Notes: ADJ = adjacent, EXT = external, INT = internal, THRU = through, EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

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 determines 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 Equation 23-24 and Equation 23-25. 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 34-90 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 34-90
Example Problem 8: Lost Time due to Downstream Queues

Movement	Interchange EB EXT-TH	SB-L	WB EXT-TH	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	SB-L	NB-R	Interchange WB INT-TH	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	SB-L	NB-R	Interchange WB INT-TH	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

Notes: EXT = external, INT = internal, TH = through, L = left, R = right, EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

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 31-154. Exhibit 34-91 and Exhibit 34-92 present the calculations of the queue storage ratio for all approaches of the interchange, while Exhibit 34-93 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 19-18. Exhibit 34-94 through Exhibit 34-96 summarize the control delay estimates for all approaches of the interchange and adjacent signalized intersection.

Exhibit 34-91
Example Problem 8: Queue Storage Ratio for Interchange Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
Value	**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

Notes: EXT = external, INT = internal, TH = through, L = left, R = right.

Exhibit 34-92
Example Problem 8: Queue Storage Ratio for Interchange Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
Value	**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 34-93
Example Problem 8: Queue Storage Ratio for Adjacent Intersection Movements

Value	Eastbound Through & Right	Eastbound Left	Westbound Through & Right	Westbound Left	Northbound Through	Northbound Right	Northbound Left	Southbound Through & Right	Southbound 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 34-94
Example Problem 8: Control Delay for Interchange Eastbound and Westbound Movements

Value	Eastbound Movements EXT-TH&R	Eastbound Movements INT-L	Eastbound Movements INT-TH	Westbound Movements EXT-TH&R	Westbound Movements INT-L	Westbound Movements 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

Notes: EXT = external, INT = internal, TH = through, L = left, R = right.

Exhibit 34-95
Example Problem 8: Control Delay for Interchange Northbound and Southbound Movements

Value	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 34-96
Example Problem 8: Control Delay for Adjacent Intersection Movements

Value	Eastbound Through & Right	Eastbound Left	Westbound Through & Right	Westbound Left	Northbound Through	Northbound Right	Northbound Left	Southbound Through & Right	Southbound 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 23-2. 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 34-97 and Exhibit 34-98. 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 the second column of Exhibit 23-10. The LOS for each lane group at the adjacent intersection is assigned on the basis of Chapter 19, Signalized Intersections. After extra distances are measured according to the Exhibit 23-8 discussion, EDTT can be obtained from Equation 23-50 [i.e., EDTT = 100 / (1.47 × 35) + 0 = 1.9 s]. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-97
Example Problem 8: Interchange O-D Movement LOS

O-D Movement	Control Delay (s/veh)	EDTT (s/veh)	ETT (s/veh)	*v/c* > 1?	R_Q > 1?	LOS
A	45.3	1.9	47.2	No	No	C
B	43.0	−1.9	41.1	No	No	C
C	53.8	−1.9	51.9	No	No	C
D	72.6	1.9	74.5	No	No	D
E	98.1	1.9	100.0	No	No	E
F	39.1	−1.9	37.2	No	No	C
G	36.0	−1.9	34.1	No	No	C
H	87.0	1.9	88.9	No	No	E
I	56.2	0.0	56.2	No	No	D
J	38.2	0.0	38.2	No	No	C

Exhibit 34-98
Example Problem 8: 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

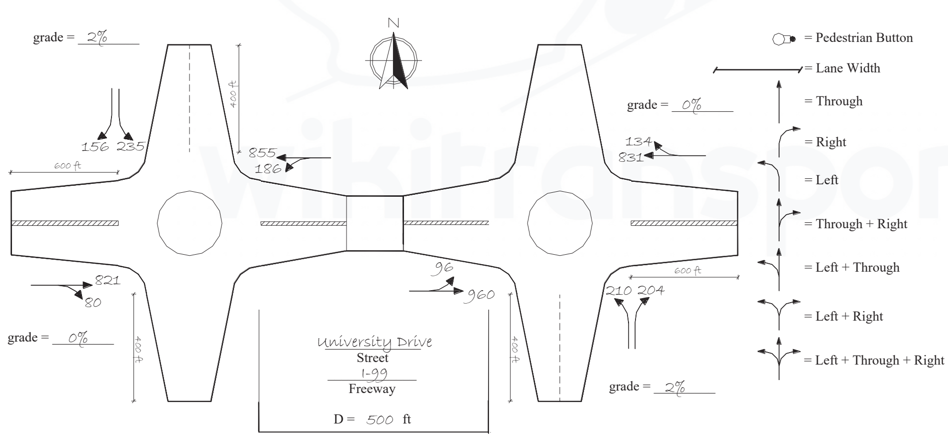
Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

EXAMPLE PROBLEM 9: 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 34-99.

Exhibit 34-99
Example Problem 9: 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 a PHF of 0.97. During the analysis period, there is no parking, and no buses, bicycles, or pedestrians utilize the interchange.

Extra distance traveled along each freeway ramp is 100 ft. The grade is 2% on the NB and SB approaches.

Solution

Calculation of Origin–Destination Movements

O-Ds through this diamond interchange are calculated by using the worksheet provided in Exhibit 34-169 in Section 4. The results of the O-D calculations and the resulting PHF-adjusted values are presented in Exhibit 34-100.

Exhibit 34-100
Example Problem 9: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)	Heavy Vehicle–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 Section 4 are used to estimate the entering and conflicting flow rates and the resulting capacity of each approach. Exhibit 34-160 and Exhibit 34-161 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 34-160) consists of O-D Movements A, B, K, and M at a diamond interchange (Exhibit 34-161). The conflicting flow (Number 12) consists of O-D Movements D, E, I, and N. Exhibit 34-101 shows the entering and conflicting flow for each approach, along with the corresponding capacity and delay.

Exhibit 34-101
Example Problem 9: 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

Notes: EXT = external, INT = internal, EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

O-D Movement Control Delay and LOS

Delay for each O-D is estimated as the sum of approach delays for each approach utilized by the O-D. For example, O-D Movement A will utilize the northbound ramp approach and westbound internal through approach. Control delays for these approaches are then summed to estimate control delay for O-D Movement A. LOS for each O-D is assigned on the basis of Exhibit 23-14. The resulting control delay and LOS for all movements are shown in Exhibit 34-102. After extra distances are measured according to the Exhibit 23-8 discussion, EDTT can be obtained from Equation 23-50 [i.e., EDTT = 100 / (1.47 × 35) + 0 = 1.9 s]. Intersectionwide performance measures are not calculated for interchange ramp terminals.

Exhibit 34-102
Example Problem 9: Control Delay and LOS for Each O-D Movement

O-D	Control Delay (s/veh)	EDTT (s/veh)	ETT (s/veh)	LOS
A	44.2	1.9	46.1	D
B	30.9	−1.9	29.0	C
C	31.1	−1.9	29.2	C
D	44.5	1.9	46.4	D
E	47.9	1.9	49.8	D
F	34.5	−1.9	32.6	C
G	33.8	−1.9	31.9	C
H	47.1	1.9	49.0	D
I	47.9	0.0	47.9	D
J	47.1	0.0	47.1	D

EXAMPLE PROBLEM 10: OPERATIONAL ANALYSIS FOR TYPE SELECTION

The Interchange

An interchange is to be built at the junction of I-83 (NB/SB) and Archer Road (EB/WB) in an urban area. The interchange type selection methodology described in Section 3 is used.

The Question

Which interchange type is likely to operate better under the given demands?

The Facts

This interchange will have two-lane approaches with single left-turn lanes on the arterial approaches. Freeway ramps will consist of two-lane approaches with channelized right turns in addition to the main ramp lanes. Default saturation flow rates for use in the type selection analysis are given in Exhibit 34-151. The O-D movements of traffic through this interchange are shown in Exhibit 34-103.

Exhibit 34-103
Example Problem 10: O-D Demand Information for the Interchange

O-D Movement	Volume (veh/h)
A	400
B	350
C	400
D	550
E	150
F	200
G	225
H	185
I	600
J	800
K	2,500
L	3,200
M	0
N	10

Outline of Solution

Mapping O-D Flows into Interchange Movements

The primary objective of this example is to compare up to eight interchange types against a given set of design volumes. The first step is to convert these O-D flows into movement flows through the signalized interchange. The interchange type methodology uses the standard NEMA numbering sequence for interchange phasing, and Exhibit 34-152 in Section 3 demonstrates which O-Ds make up each NEMA phase at the eight interchange types. Exhibit 34-104 shows the corresponding volumes for this example on the basis of the O-Ds from Exhibit 34-103. Since this interchange has channelized right turns, Exhibit 34-105 shows only the NEMA phasing volumes utilizing the signals.

Exhibit 34-104
Example Problem 10: NEMA Flows (veh/h) for the Interchange

Interchange Type	NEMA Phase Movement Number							
	1	2	3	4	5	6	7	8
SPUI	185	800	400	400	150	1,025	560	350
TUDI /CUDI	185	950	--	960	160	1,210	--	750
CDI (I)	185	950	--	960	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	960	--	1,385	--	--
Parclo A-4Q (II)	--	1,310	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	960	200	1,385	--	--
Parclo A-2Q (II)	225	1,310	--	--	--	985	--	750
Parclo B-4Q (I)	185	950	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	950	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Notes: SPUI = single-point urban interchange, TUDI = tight urban diamond interchange, CUDI = compressed urban diamond interchange, CDI = conventional diamond interchange, Parclo = partial cloverleaf.
(I) and (II) indicate the intersections within the interchange type.
-- indicates that the movement does not exist in this interchange type.

Exhibit 34-105
Example Problem 10: NEMA Flows for the Interchange Without Channelized Right Turns

Interchange Type	NEMA Phase Movement Number 1	2	3	4	5	6	7	8
SPUI	185	600	400	0	150	1,025	560	350
TUDI /CUDI	185	750	--	560	160	1,210	--	750
CDI (I)	185	750	--	560	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	560	--	1,385	--	--
Parclo A-4Q (II)	--	1,150	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	560	200	1,385	--	--
Parclo A-2Q (II)	225	1,150	--	--	--	985	--	750
Parclo B-4Q (I)	185	750	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	750	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Notes: (I) and (II) indicate the intersections within the interchange type.
-- indicates that the movement does not exist in this interchange type.

Computation of Critical Flow Ratios

Comparison between the eight intersection types begins with computation of the critical flow ratio at each interchange type. The first intersection type to be calculated is the SPUI by using Equation 34-1. On the basis of the default saturation flow rate for a SPUI and the values for the NEMA phases, Exhibit 34-106 shows the output from these calculations for a SPUI. The TUDI critical flow ratios are calculated by using Equation 34-4. Exhibit 34-107 shows these calculations for a 300-ft distance between the two TUDI intersections.

Exhibit 34-106
Example Problem 10: SPUI Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements, *A*	0.368	0.306
Critical flow ratio for the ramp movements, *R*	0.350	0.156
Sum of critical flow ratios, Y_c	0.718	0.462

Exhibit 34-107
Example Problem 10: TUDI Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Effective flow ratio for concurrent phase when dictated by travel time, y_t	0.070	0.070
Effective flow ratio for concurrent Phase 3, y_3	0.070	0.070
Effective flow ratio for concurrent Phase 7, y_7	0.070	0.070
Critical flow ratio for the arterial movements, *A*	0.461	0.294
Critical flow ratio for the ramp movements, *R*	0.474	0.315
Sum of critical flow ratios, Y_c	0.935	0.609

Exhibit 34-108
Example Problem 10: CUDI Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Flow ratio for Phase 2 with consideration of pre-positioning, y_2	0.264	0.208
Flow ratio for Phase 6 with consideration of pre-positioning, y_6	0.208	0.208
Critical flow ratio for the arterial movements, *A*	0.373	0.332
Critical flow ratio for the ramp movements, *R*	0.267	0.156
Sum of critical flow ratios, Y_c	0.640	0.488

Exhibit 34-109
Example Problem 10: CDI Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.655	0.498
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.118
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.486
Maximum sum of critical flow ratios, Y_c	0.655	0.498

The CUDI critical flow ratios are calculated by using Equation 34-9. Exhibit 34-108 shows these calculations for a CUDI with the given O-D flows.

The CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q all use separate controllers. For these interchanges the critical flow ratios are calculated for each intersection, and then the maximum is taken for the overall interchange critical flow ratio. These numbers are all calculated by using Equation 34-14 and the default saturation flows. Exhibit 34-109 through Exhibit 34-113 show the calculations for these interchanges utilizing two controllers.

Exhibit 34-110
Example Problem 10: Parclo A-4Q Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.385	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.282
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.667	0.615
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.364	0.333
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.208	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.572	0.444
Maximum sum of critical flow ratios, Y_c	0.667	0.615

Exhibit 34-111
Example Problem 10: Parclo A-2Q Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.502	0.451
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.784	0.616
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.452
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.563
Maximum sum of critical flow ratios, Y_c	0.784	0.616

Exhibit 34-112
Example Problem 10: Parclo B-4Q Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.000	0.000
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.373	0.333
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.000	0.000
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.430	0.368
Maximum sum of critical flow ratios, Y_c	0.430	0.368

Exhibit 34-113
Example Problem 10: Parclo B-2Q Critical Flow Ratio Calculations

Value	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.111	0.111
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.484	0.444
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.103	0.103
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.533	0.471
Maximum sum of critical flow ratios, Y_c	0.533	0.471

Estimation of Interchange Delay

Estimation of interchange delay is the final step when interchange types are compared. On the basis of the critical flow ratios calculated previously, Exhibit 34-159 in Section 3 can be used to calculate the delay at the eight interchange types. Exhibit 34-114 shows the solutions to these calculations.

Exhibit 34-114
Example Problem 10: Interchange Delay for the Eight Interchange Types

Intersection Type	Interchange Delay d_I (s/veh) Right Turns Signalized	Interchange Delay d_I (s/veh) Right Turns Free or YIELD-Controlled
SPUI	62.9	22.0
TUDI	217.7	33.3
CUDI	35.9	27.4
CDI	26.6	21.7
Parclo A-4Q	26.2	21.6
Parclo A-2Q	47.4	29.0
Parclo B-4Q	11.9	11.3
Parclo B-2Q	30.7	29.0

Results

As demonstrated by Exhibit 34-114, a Parclo B-4Q would be the best interchange type to select in terms of operational performance for the given O-D flows at this interchange. For the final interchange type selection, however, additional criteria should be considered, including those related to economic, environmental, and land use concerns.

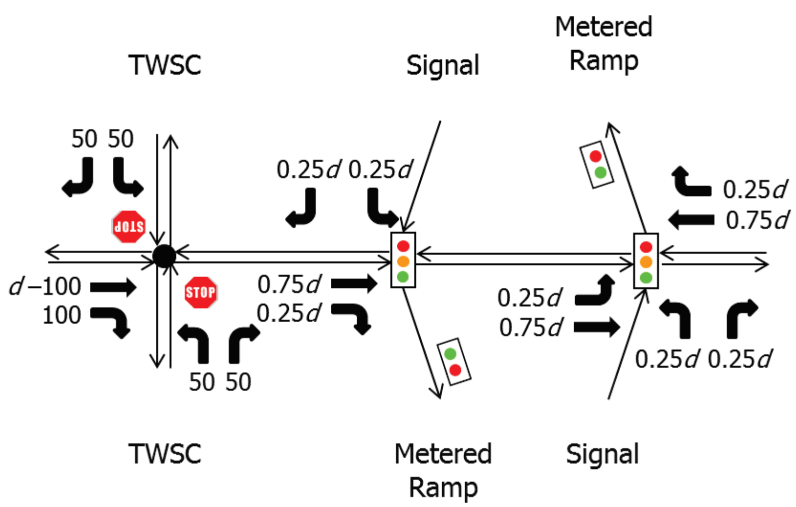
EXAMPLE PROBLEM 11: ALTERNATIVE ANALYSIS TOOL

This example presents a simulation analysis of the diamond interchange configuration originally described in Example Problem 1. A few changes have been made to introduce elements that are beyond the stated limitations of the interchange ramp terminal procedures. The use of a typical simulation tool to address the limitations is described in this section. The need to determine performance measures from 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 development of alternative tools. Pending further development, the example presented in this chapter applied existing versions of alternative tools and thus 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 34-115 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 34-115
Example Problem 11: Interchange Configuration and Demand Volumes



Note: TWSC = two-way STOP control.

Exhibit 34-116 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 23 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 34-116
Example Problem 11: Signal Timing Plan

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

Summary of Simulation Runs

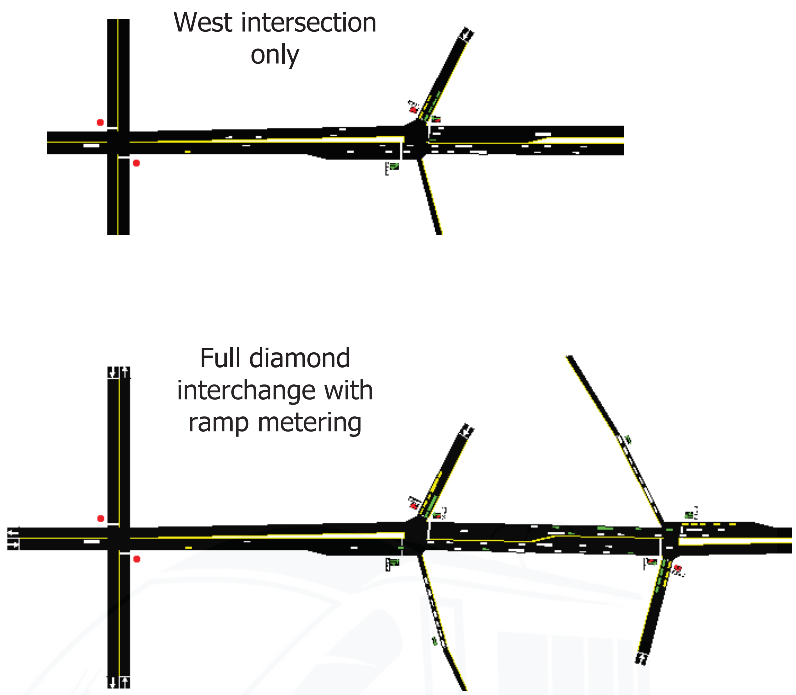
Operation of this interchange was simulated by using demand volumes *d* ranging from 600 veh/h (very undersaturated) to 1,800 veh/h (very oversaturated). The volume increment was 200 veh/h. Thirty simulations were run for each condition to capture stochastic variations inherent to simulation.

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 34-117. The west intersection was examined separately to show the difference between a signalized intersection operating independently and one operating as part of a diamond interchange with mutual interactions between intersections at each end.

Exhibit 34-117
Example Problem 11: Physical Configurations Examined



Diamond Interchange Operation

Exhibit 34-118 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 34-118
Example Problem 11: 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 23 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 34-119 shows the westbound arterial discharge from the diamond interchange (through plus left turns) as a function of arterial demand *d*. Note that the discharge tracks the demand at low volumes, which indicates that all 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 nears 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 23 methodology.

Exhibit 34-120 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.

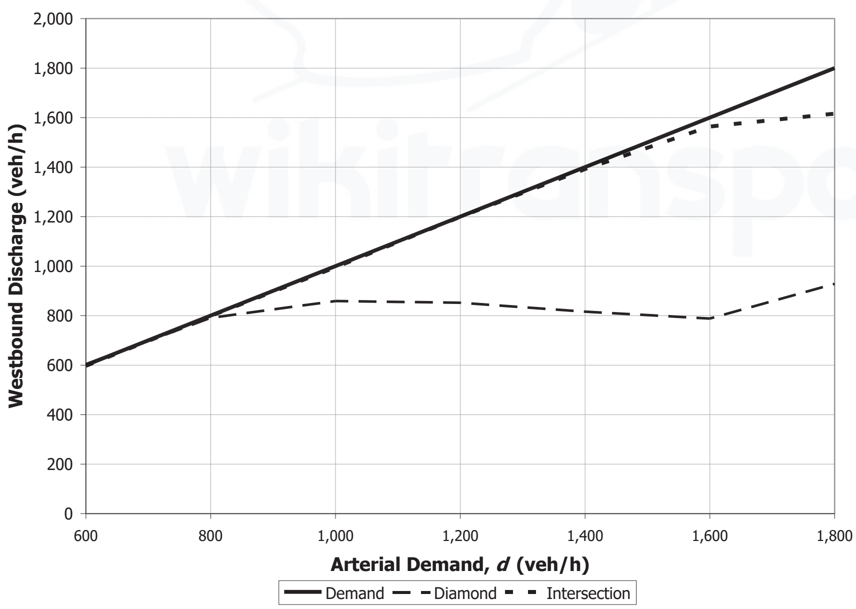
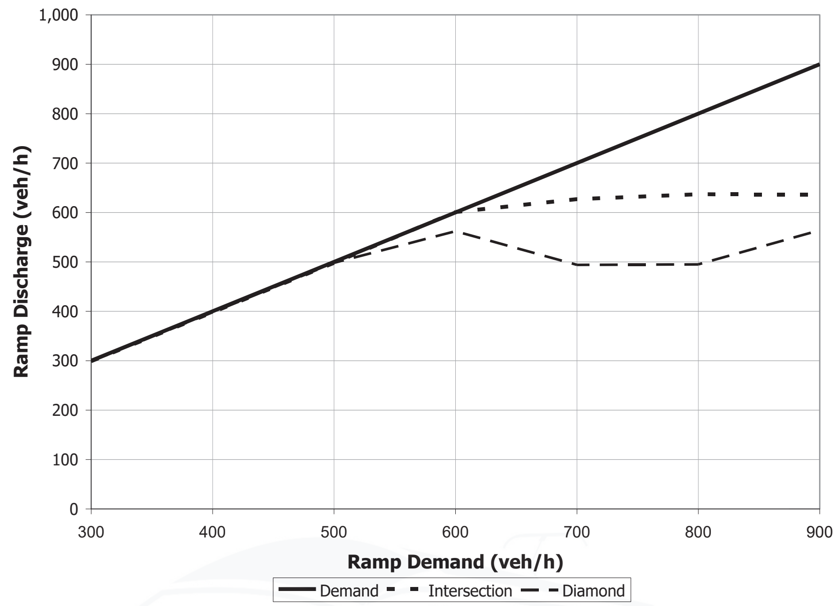


Exhibit 34-119
Example Problem 11: Discharge from the Diamond Interchange Under the Full Range of Arterial Demand

Exhibit 34-120
Example Problem 11: Discharge from the Southbound Exit Ramp Under the Full Range of Ramp Demand



TWSC Intersection Operation

The TWSC analysis procedures prescribed in Chapter 20 recognize the effects of adjacent signalized intersections to some extent, but they do not address cases in which 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 34-121, 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 34-121
Example Problem 11: Congested Approaches to the TWSC Intersection

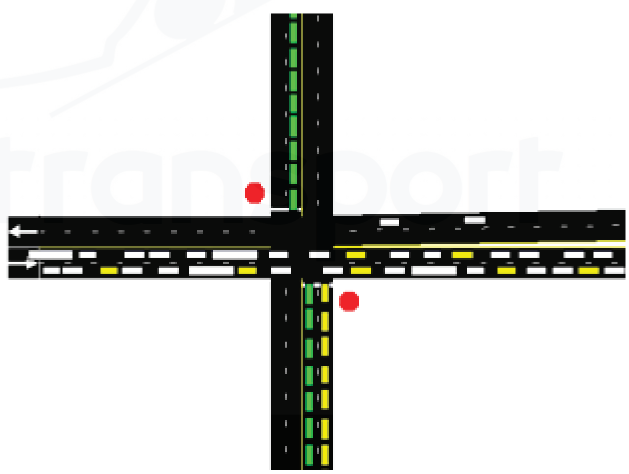
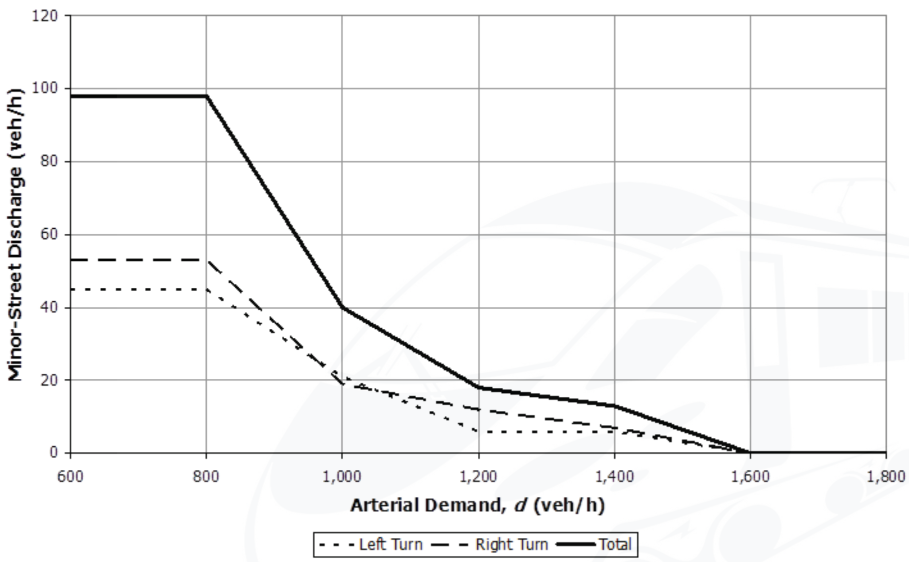
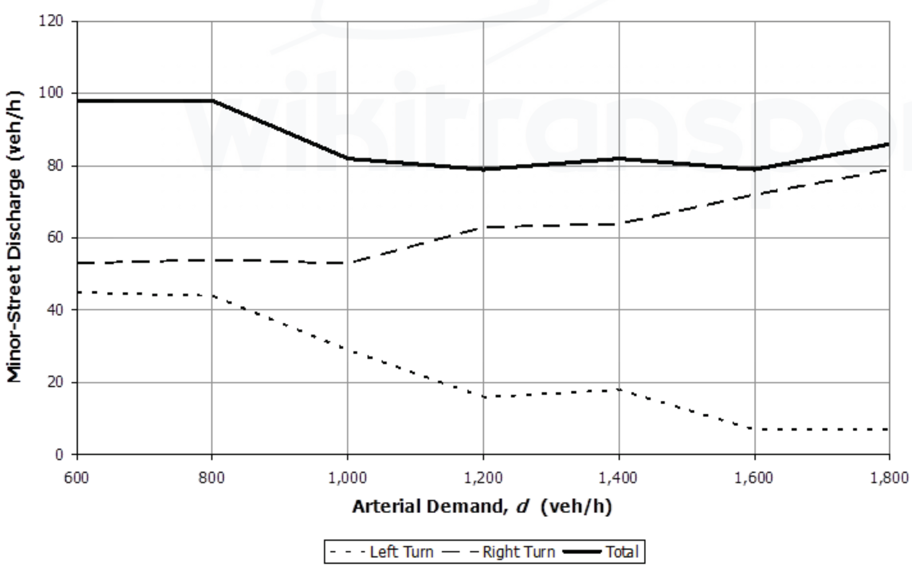


Exhibit 34-122 shows the minor-street entry as a function of the arterial demand. Unlike the other movements in this example, the minor-street demand was kept constant throughout the entire range of arterial demand. According to a well-established principle of TWSC analysis, the entry capacity for minor-street movements diminishes with increasing major-street volumes. That phenomenon is depicted clearly for northbound traffic in Exhibit 34-121. It is evident here that capacity begins to drop below demand at about 800 veh/h in each arterial direction. The southbound situation, on the other hand, presents some surprising results. The southbound left turn is impeded by a 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, producing entry volumes that exceed the original demand. Animated graphics indicate 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 34-122
Example Problem 11: Effect of Arterial Demand on Minor-Street Discharge at the TWSC Intersection



(a) Northbound



(b) Southbound

EXAMPLE PROBLEM 12: FOUR-LEGGED RESTRICTED CROSSING U-TURN INTERSECTION WITH MERGES

The Intersection

An RCUT with merges in a rural area has four approaches.

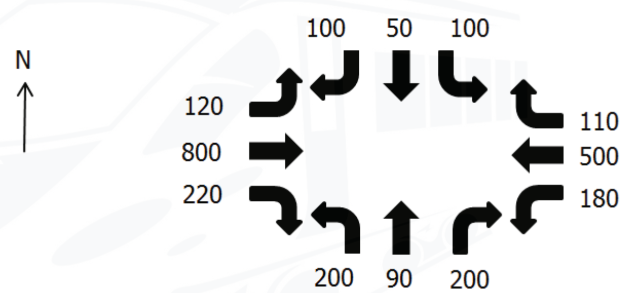
The Question

What is the LOS for each of the 12 movements at the intersection?

The Facts

The geometry is as pictured in Exhibit 23-42, with the main street running east–west. The distance from the main intersection to each U-turn crossover is 2,000 ft. The storage bay length for each left-turn crossover is 300 ft. The PHF is 0.92. Free-flow speed on the major street is 60 mi/h. The truck percentages are zero, and there are no significant grades on any approach. Exhibit 34-123 shows the vehicular demands (veh/h).

Exhibit 34-123
Example Problem 12: Turning Movement Demands



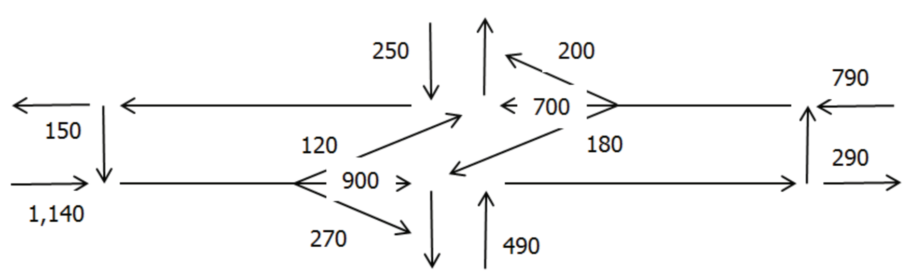
Solution

The solution follows the 10-step procedure outlined in Chapter 23. Once the *v*/*c* ratio, 95% queue-to-storage ratio, and experienced travel time have been determined for a movement, its LOS will be found by using Exhibit 23-13.

Determination of O-D Demands and Movement Demands

Exhibit 34-124 shows the demands (veh/h) redistributed to the different junctions of the RCUT.

Exhibit 34-124
Example Problem 12: Demands Converted to the RCUT Geometry



Determination of Lane Groups

RCUTs with merges do not have signals, so there is no need to determine lane or movement groups at each approach. Exhibit 34-125 shows the redistributed demands converted to flow rates (veh/h) by using the PHF and Equation 23-55.

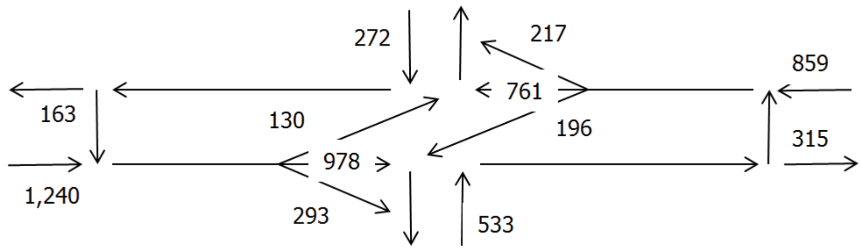


Exhibit 34-125
Example Problem 12: Flow Rates in the RCUT Geometry

Determination of Lane Utilization

This step is not needed for an RCUT with merges.

Calculation of Signal Progression Adjustments

This step is not needed for an RCUT with merges.

Calculation of Additional Control-Based Adjustments

For an RCUT with merges, the analyst may use judgment to determine whether significant weaving delay exists. When significant weaving delay exists, the analyst must develop an estimate of this delay from field measurements or an alternative tool and add it to the EDTT estimate calculated later.

Calculation of Junction-Specific Performance Measures

At an RCUT with merges that passes the weaving area tests in Step 5, control delay is only experienced by the major-street left turns. Use of the methods of Chapter 20 with the inputs listed above, and with default values for all other factors provided, produces the following results:

- For the eastbound left turn (at the north main intersection), v/c = 0.18, 95% queue length = 0.66 veh or 16.5 ft at 25 ft/veh, and control delay = 11.2 s/veh; and
- For the westbound left turn (at the south main intersection), v/c = 0.35, 95% queue length = 1.58 veh or 39.5 ft at 25 ft/veh, and control delay = 15.0 s/veh.

Calculation of Extra Distance Travel Time

The bottom portion of Exhibit 23-48 shows that at a four-legged RCUT with merges, extra travel distance is experienced by the left turns from the minor street and by the through movements on the minor street. Both minor left turns will experience the same extra distance travel time (EDTT) since the distances from the main intersection to both U-turn crossovers are the same. Use of Equation 23-56 results in the following EDTT:

$$EDTT = \frac{D_t + D_f}{1.47 \times FFS} + a$$

$$EDTT = \frac{2{,}000 + 2{,}000}{1.47 \times 60} + 10 = 55.4 \text{ s/veh}$$

Both minor-street through movements will experience the same EDTT, since the distances from the main intersection to both U-turn crossovers are the same. Use of Equation 23-56 results in the following EDTT:

$$EDTT = \frac{2{,}000 + 2{,}000}{1.47 \times 60} + 15 = 60.4 \text{ s/veh}$$

Calculation of Additional Weaving Delay

In this example problem, it is assumed that no significant weaving delay exists, in the analyst's judgment. Therefore, there are no adjustments to make in this step.

Calculation of Experienced Travel Time

Experienced travel time (ETT) is computed with Equation 23-58:

$$ETT = \sum d_i + \sum EDTT$$

The bottom portion of Exhibit 23-48 gives the following:

- For the EB left from the major street, ETT = 11.2 + 0 = 11.2 s/veh.
- For the WB left from the major street, ETT = 15.0 + 0 = 15.0 s/veh.
- For the major-street through movements, ETT = 0 + 0 = 0 s/veh.
- For the major-street right-turn movements, ETT = 0 + 0 = 0 s/veh.
- For the left turns from the minor street, ETT = (0 + 0) + 55.4 = 55.4 s/veh.
- For the through movements from the minor street, ETT = (0 + 0) + 60.4 = 60.4 s/veh.
- For the right turns from the minor street, ETT = 0 + 0 = 0 s/veh.

Determination of Level of Service

The LOS for each movement is obtained with Exhibit 23-13 (it has been established that the v/c ratio was less than 1.0 at all junctions and that the queue-to-storage ratios were well below 1.0 for the 300-ft bay lengths provided):

- For the eastbound left from the major street, LOS = B.
- For the westbound left from the major street, LOS = B.
- For the major-street through movements, LOS = A.
- For the major-street right-turn movements, LOS = A.
- For the minor-street left turns, LOS = E.
- For the minor-street through movements, LOS = E.
- For minor-street right turns, LOS = A.

Discussion

The minor-street left-turn and through movements experience LOS E because of the distances from the main intersection to the U-turn crossovers and the major-street free-flow speed. Chapter 23 explores the sensitivity of EDTT and LOS to these factors. It shows that, over typical ranges, there is some change in EDTT and LOS as a result of these factors but that achievement of a LOS better than D or E for minor-street left-turn and through movements with this design will be difficult.

EXAMPLE PROBLEM 13: THREE-LEGGED RESTRICTED CROSSING U-TURN INTERSECTION WITH STOP SIGNS

The Intersection

An RCUT with STOP signs in a rural area has three approaches.

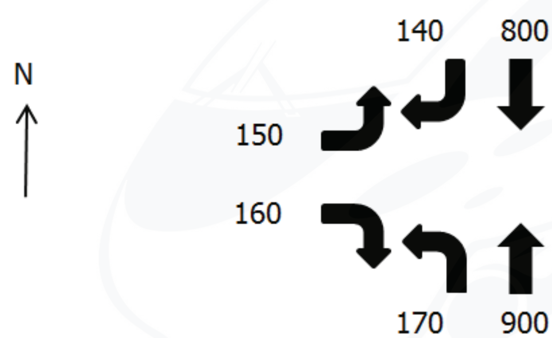
The Question

What is the LOS for each of the six movements at the intersection?

The Facts

The main street runs north–south. The distance from the main intersections to the U-turn crossover is 700 ft. The storage bay lengths for the left-turn and U-turn crossovers are 400 ft. The PHF is 0.90. The free-flow speed on the major street is 60 mi/h. The truck percentage is 5.9% on the EB approach and 6.1% on all other approaches. The grade on the EB approach is 2%, there are no pedestrians, and there are no nearby traffic signals. Exhibit 34-126 shows the vehicular demands (veh/h).

Exhibit 34-126
Example Problem 13: Turning Movement Demands



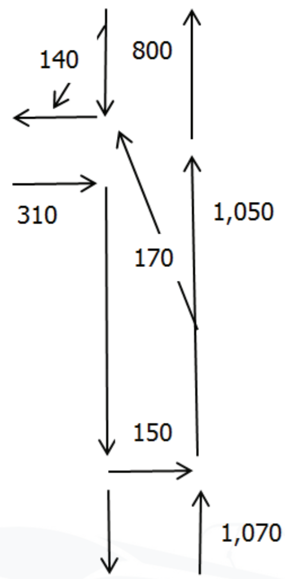
Solution

The solution follows the 10-step procedure outlined in Chapter 23. Once the *v*/*c* ratio, queue-to-storage ratio, and experienced travel time have been determined for a movement, its LOS will be found with Exhibit 23-13.

Determination of O-D Demands and Movement Demands

Exhibit 34-127 shows the demands (veh/h) redistributed to the various junctions of the RCUT.

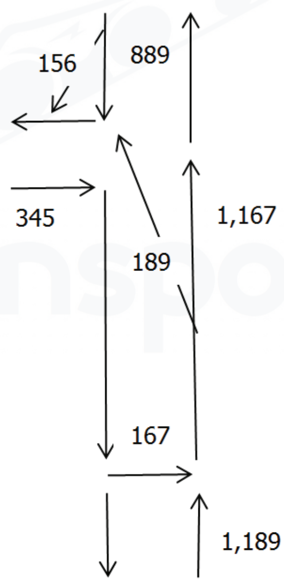
Exhibit 34-127
Example Problem 13: Demands Converted to the RCUT Geometry



Determination of Lane Groups

RCUTs with STOP signs do not have traffic signals, so there is no need to determine lane or movement groups at each approach. Exhibit 34-128 shows the redistributed demands converted to flow rates (veh/h) on the basis of the PHF and Equation 23-55.

Exhibit 34-128
Example Problem 13: Flow Rates in the RCUT Geometry



Determination of Lane Utilization

This step is not needed for an RCUT with STOP signs.

Calculation of Signal Progression Adjustments

This step is not needed for an RCUT with STOP signs.

Calculation of Additional Control-Based Adjustments

For this RCUT with STOP signs, no field data on the base critical headway and base follow-up time are available, so the solution will use the default values suggested in Chapter 23.

Calculation of Junction-Specific Performance Measures

The bottom of Exhibit 23-49 shows that, for a three-legged RCUT with STOP signs, control delay is experienced by the major-street left-turn and minor-street left-turn and right-turn vehicles at the main junction and by the minor-street left-turn vehicles at the U-turn crossover. The methods of Chapter 20, with the inputs listed above and default values for all other factors, provide the following results:

- For the eastbound minor-street left-turn and through vehicles at the main junction, v/c = 0.59, 95% queue length = 3.8 veh or 95 ft at 25 ft/veh, and control delay = 19.4 s/veh.
- For the northbound major-street left turn at the main junction, v/c = 0.29, 95% queue length = 1.2 veh or 30 ft at 25 ft/veh, and control delay = 12.9 s/veh.
- For the eastbound minor-street left turn at the U-turn crossover, v/c = 0.19, 95% queue length = 0.69 veh or 17 ft at 25 ft/veh, and control delay = 10.0 s/veh.

Calculation of Extra Distance Travel Time

The bottom portion of Exhibit 23-49 shows that at a three-legged RCUT with STOP signs, extra travel distance is experienced by the left turns from the minor street. Use of Equation 23-57 gives the extra distance travel time (EDTT):

$$EDTT = \frac{D_t + D_f}{1.47 \times FFS}$$

$$EDTT = \frac{700 + 700}{1.47 \times 60} = 15.9 \text{ s/veh}$$

Calculation of Additional Weaving Delay

For an RCUT with STOP signs there are no adjustments to make in this step.

Calculation of Experienced Travel Time

Experienced travel time (ETT) is computed with Equation 23-58:

$$ETT = \sum d_i + \sum EDTT$$

Use of the bottom portion of Exhibit 23-49 gives the following:

- For the northbound left from the major street, ETT = 12.9 + 0 = 12.9 s/veh.
- For the major-street through movements, ETT = 0 + 0 = 0 s/veh.
- For the major-street right-turn movement, ETT = 0 + 0 = 0 s/veh.
- For the left turn from the minor street, ETT = (19.4 + 10.0) + 15.9 = 45.3 s/veh.
- For the right turn from the minor street, ETT = 19.4 + 0 = 19.4 s/veh.

Determination of Level of Service

LOS for each movement is obtained with Exhibit 23-13 (it has been established that the v/c ratio was less than 1.0 at all junctions and that the queue-to-storage ratios were well below 1.0 for the 400-ft bay lengths provided):

- For the eastbound left from the major street, LOS = B.
- For the major-street through movements, LOS = A.
- For the major-street right-turn movement, LOS = A.
- For the left turn from the minor street, LOS = D.
- For the right turn from the minor street, LOS = B.

Discussion

Interesting factors to examine in this problem are the base critical headway and base follow-up time at the U-turn crossover and the minor-street left-turn demand. Recalculation of the example by using the default values for base critical headway and base follow-up time for minor-street left turns (7.1 s and 3.5 s, respectively) results in control delay at the U-turn crossover rising from 10.0 to 18.6 s/veh. In turn, this changes the EDTT value for the minor-street left-turn movement to 52.7 s/veh, which is still LOS D. It is apparent that the base critical headway and base follow-up time values used in the U-turn crossover analysis could affect LOS by one level.

In general, the RCUT design requires extra travel time for the minor-street left-turn and through movements while minimizing delays for the major-street movements. Chapter 23 shows, for the conditions in this example, how far the minor street can be pushed before it reaches LOS F. In this case, a demand of more than 250 veh/h minor-street left turns in conjunction with 250 veh/h minor-street right turns results in LOS F. If these are peak-period flows and typical *K*- and *D*-factors apply, these demand levels translate to annual average daily traffic values of 8,000 to 10,000 veh/day. Of course, better levels of service can be achieved on the minor-street approach with an additional lane. Chapter 23 also illustrates that minor-street left-turn LOS at an RCUT with STOP signs will rarely achieve better than LOS D. It is apparent that the LOS constraint at an RCUT will typically be the minor-street approach, which serves more movements than the major-street left-turn crossover or the U-turn crossover.

EXAMPLE PROBLEM 14: FOUR-LEGGED RESTRICTED CROSSING U-TURN INTERSECTION WITH SIGNALS

The Intersection

An RCUT with signals in a suburban area has four approaches.

The Question

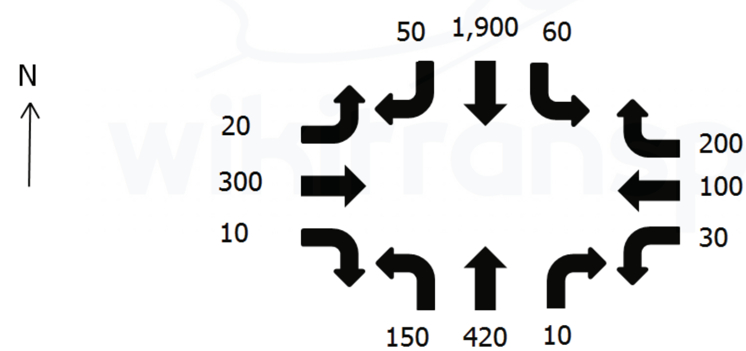
What is the LOS for each of the 12 movements at the intersection and for the facility as a whole?

The Facts

The main street runs north–south. The distance from the main intersections to the U-turn crossovers is 800 ft. The storage bay lengths for the left-turn and U-turn crossovers are 400 ft. The median is 40 ft wide. All crossovers have a single lane. The major street has two through lanes and exclusive right-turn lanes at the main junction in each direction. The minor street has two lanes on each of the approaches to the main junctions. The PHF is 0.93. Free-flow speed on the major street is 50 mi/h. The truck percentages are 3.7%. Grades are flat on all approaches. There are no pedestrians, and there are no significant volumes turning on a red signal. Exhibit 34-129 shows the vehicular demands (veh/h).

The signals are pretimed as part of a longer RCUT corridor. The arrival type is 6 on the major street at the U-turn crossover signals in both directions and 3 for the minor street. At both southbound signals, the cycle length is 90 s, with 60 s of major-street green, 20 s of minor-street or crossover green, 4 s of yellow, and 1 s of all-red. At both northbound signals, the cycle length is 60 s, with 25 s of major-street green, 25 s of minor-street or crossover green, 4 s of yellow, and 1 s of all-red.

Exhibit 34-129
Example Problem 14: Turning Movement Demands



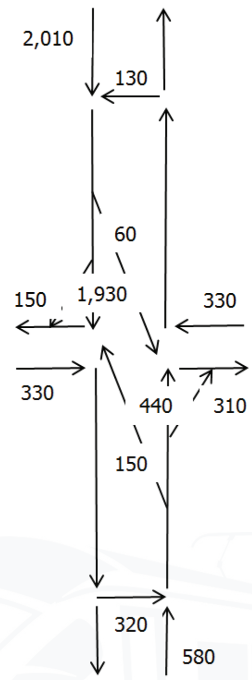
Solution

The solution follows the 10-step procedure outlined in Chapter 23. Once the *v*/*c* ratio, queue-to-storage ratio, and experienced travel time have been determined for a movement, its LOS will be found with Exhibit 23-13.

Determination of O-D Demands and Movement Demands

Exhibit 34-130 shows the demands (veh/h) redistributed to the various junctions of the RCUT.

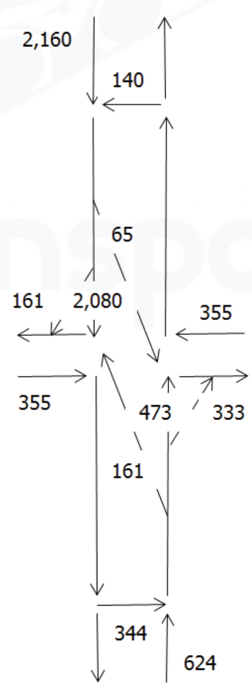
Exhibit 34-130
Example Problem 14: Demands Converted to the RCUT Geometry



Determination of Lane Groups

Lane and movement groups at each approach are determined with the methods of Chapter 19. Exhibit 34-131 shows the redistributed demands converted to flow rates (veh/h) obtained by using the PHF and Equation 23-55.

Exhibit 34-131
Example Problem 14: Flow Rates in the RCUT Geometry



Determination of Lane Utilization

With no field data on hand, the default lane distribution is applied to all approaches to signals.

Calculation of Signal Progression Adjustments

The top portion of Exhibit 23-51 is used to find arrival types for each approach to each signal after the first signal encountered.

Calculation of Additional Control-Based Adjustments

For this RCUT with signals, no field data are available on the saturation flow rate for traffic in the U-turn crossover, so the solution will use the default value of 0.85 suggested in Exhibit 23-52 for a 40-ft median width.

Calculation of Junction-Specific Performance Measures

The top portion of Exhibit 23-48 shows that, for a four-legged RCUT with signals, one to three increments of control delay are experienced by each movement. The methods of Chapter 19 are applied to calculate these delays, on the basis of the inputs listed above and defaults for all other values. The results are shown in Exhibit 34-132.

Exhibit 34-132
Example Problem 14: Control Delay for Each Junction

Junction	Movement	*v/c*	95% Queue Length (veh)	Control Delay (s/veh)
North crossover	SB through	0.92	4.4	7.6
	WB crossover	0.40	5.0	33.3
West main intersection	SB through	0.89	3.2	5.4
	SB right turn	0.16	0.2	0.3
	EB right turn	0.58	6.4	35.1
	NB left turn	0.41	5.7	33.2
South crossover	NB through	0.43	1.4	4.1
	EB crossover	0.53	5.9	16.1
East main intersection	NB through	0.32	1.7	6.4
	NB right turn	0.51	3.1	9.1
	WB right turn	0.31	2.4	12.4
	SB left turn	0.09	0.8	10.8

Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

Calculation of Extra Distance Travel Time

The top portion of Exhibit 23-48 shows that at a four-legged RCUT with signals, extra travel distance is experienced by the left turns and through movements from the minor street. Use of Equation 23-57 gives the following extra distance travel time (EDTT):

$$EDTT = \frac{D_t + D_f}{1.47 \times S_f}$$

$$EDTT = \frac{800 + 800}{1.47 \times 50} = 21.8 \text{ s/veh}$$

Calculation of Additional Weaving Delay

For an RCUT with signals, there are no adjustments to make in this step.

Calculation of Experienced Travel Time

Experienced travel time (ETT) is computed with Equation 23-58:

$$ETT = \sum d_i + \sum EDTT$$

Use of the top portion of Exhibit 23-48 gives the results in Exhibit 34-133.

Exhibit 34-133
Example Problem 14: ETT and LOS Results

Movement	Control Delay (s/veh) by Traffic Control Device: First	Second	Third	EDTT (s/veh)	ETT (s/veh)	LOS
NB left	4.1	33.2	None	0	37.3	D
SB left	7.6	10.8	None	0	18.4	B
NB through	4.1	6.4	None	0	10.5	B
SB through	7.6	5.4	None	0	13.0	B
NB right	4.1	9.1	None	0	13.2	B
SB right	7.6	0.3	None	0	7.9	A
EB left	35.1	16.1	6.4	21.8	79.4	E
WB left	12.4	33.3	5.4	21.8	72.9	E
EB through	35.1	16.1	9.1	21.8	82.1	F
WB through	12.4	33.3	0.3	21.8	67.8	E
EB right	35.1	None	None	0	35.1	D
WB right	12.4	None	None	0	12.4	B

Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

Determination of Level of Service

Levels of service for each movement are shown above in Exhibit 34-133. The results were obtained with Exhibit 23-13, after establishing that the v/c ratio was less than 1.0 at all junctions and that the queue-to-storage ratios were well below 1.0 for the 400-ft bay lengths provided.

The ETT for the entire intersection is obtained from Equation 23-60:

$$ETT_I = \frac{\sum(ETT_j \times v_j)}{\sum v_j}$$

ETT_I is 79,900 / 3,500 = 22.8 s/veh, which corresponds to LOS C.

Discussion

One of the concerns at an RCUT is the possibility of uneven lane distribution on a multilane minor-street approach or a multilane U-turn crossover. The results above were produced by assuming a relatively even lane distribution on the two-lane minor-street approaches. On the westbound minor-street approach, there was a demand of 200 veh/h to turn right and 130 veh/h to turn left or make a through movement. Placing all of the right-turn vehicles in the right lane and all of the other vehicles in the left lane would add just 0.3 s/veh of control delay to those movements, which indicates that for situations like the one in this example, lane distribution may not matter too much.

The effect of the saturation flow adjustment factor for U-turns can also be examined. The default suggested in Exhibit 23-52 for this case, with a 40-ft-wide median, is 0.85. If field data showed that the factor should be 0.8, control delay for each movement using a crossover would increase by 0.7 to 0.9 s/veh from the results in Exhibit 34-133. On the other hand, if field data showed that the factor should be 0.9, the control delay for each movement using a crossover would decrease by 0.6 to 0.7 s/veh, compared with the results in Exhibit 34-133. Overall, the U-turn saturation flow adjustment factor only makes a small difference in this problem.

EXAMPLE PROBLEM 15: FOUR-LEGGED MEDIAN U-TURN INTERSECTION WITH STOP SIGNS

The Intersection

An MUT with STOP signs at the U-turn crossovers in a suburban area has four approaches.

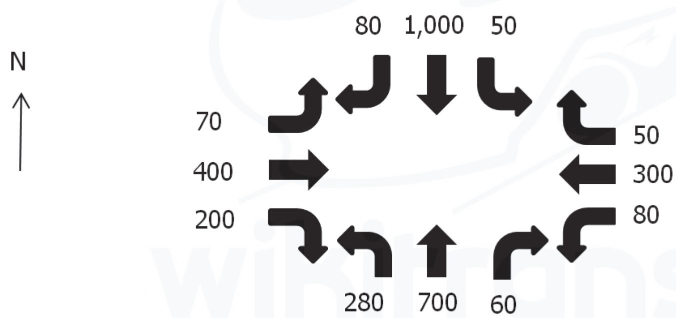
The Question

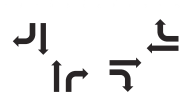
What is the LOS for each of the 12 movements at the intersection?

The Facts

The main street runs north–south. The distance from the main intersections to the U-turn crossovers is 600 ft. The storage bay lengths for the left-turn and U-turn crossovers are 500 ft. Both U-turn crossovers have a single lane. The major street has two through lanes at the main junction, with shared right-turn lanes. The minor street has one through lane and one exclusive right-turn lane on each approach to the main junction. The PHF is 0.95. Free-flow speed on the major street is 40 mi/h. The truck percentages are 2.6%. Grades are flat on all approaches. There are 100 pedestrians per hour on each crosswalk at the main junction, and there are no turns on red at the signal due to the pedestrians. Exhibit 34-134 shows the vehicular demands (veh/h). The signal is actuated and not coordinated. The yellow time is 4 s and the all-red is 1 s. Maximum green times are 30 s for east–west phases and 50 s for north–south phases.

Exhibit 34-134
Example Problem 15: Turning Movement Demands and Average Interval Durations





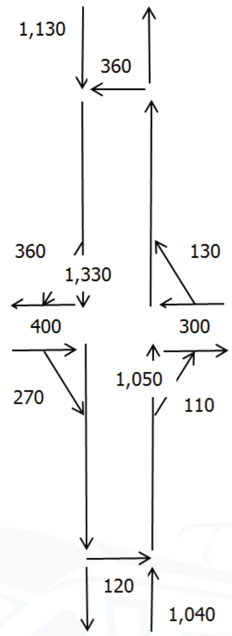
Green (s)	39.8	18.9
Yellow (s)	4.0	4.0
Red (s)	1.0	1.0

Solution

Determination of O-D Demands and Movement Demands

Exhibit 34-135 shows the demands (veh/h) redistributed to the various junctions of the MUT.

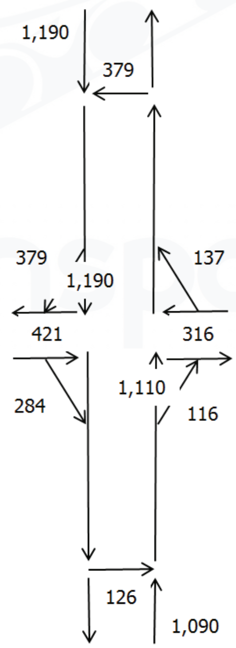
Exhibit 34-135
Example Problem 15: Demands Converted to the MUT Geometry



Determination of Lane Groups

Lane and movement groups at each approach are determined with the methods of Chapter 19. Exhibit 34-136 shows the redistributed demands converted to flow rates (veh/h) obtained by using the PHF and Equation 23-55.

Exhibit 34-136
Example Problem 15: Flow Rates in the MUT Geometry



Determination of Lane Utilization

With no field data on hand, the default lane distribution is applied to the major-street approaches to the signal.

Calculation of Signal Progression Adjustments

Because the signal is not coordinated, arrival types of 3 will be used on all approaches to the signal.

Calculation of Additional Control-Based Adjustments

For this MUT with STOP signs at the U-turn crossovers, no field data on the base critical headway and no base follow-up time are available, so the solution uses the default values suggested in Chapter 23.

Calculation of Junction-Specific Performance Measures

The middle portion of Exhibit 23-50 shows that, for a four-legged MUT with STOP signs at the U-turn crossovers, one to three increments of control delay are experienced by each movement. The methods of Chapters 19 and 20 are applied, by using the inputs listed above and defaults for all other values. The results are shown in Exhibit 34-137.

Exhibit 34-137
Example Problem 15: Control Delay for Each Junction

Junction	Movement	*v/c*	95% Queue Length (veh)	Control Delay (s/veh)
North crossover	WB crossover	0.78	7.1	34.6
Main intersection	EB through	0.82	10.2	25.1
	EB right turn	0.74	7.1	23.7
	WB through	0.62	7.5	22.2
	WB right turn	0.35	3.0	20.2
	NB through	0.58	8.3	9.3
	NB right turn	0.58	8.0	9.4
	SB through	0.76	12.2	12.3
	SB right turn	0.80	12.0	13.7
South crossover	EB crossover	0.24	0.9	14.0

Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

Calculation of Extra Distance Travel Time

The middle portion of Exhibit 23-50 shows that at a four-legged MUT with STOP signs at the U-turn crossovers, extra travel distance is experienced by the left turns from the major and minor streets. Use of Equation 23-57 gives the extra distance travel time (EDTT) as follows:

$$EDTT = \frac{D_t + D_f}{1.47 \times S_f}$$

$$EDTT = \frac{800 + 800}{1.47 \times 50} = 21.8 \text{ s/veh}$$

Calculation of Additional Weaving Delay

For an MUT, there are no adjustments to make in this step.

Calculation of Experienced Travel Time

Experienced travel time (ETT) is computed with Equation 23-58:

$$ETT = \sum d_i + \sum EDTT$$

Use of the middle portion of Exhibit 23-50 gives the results in Exhibit 34-138.

Exhibit 34-138
Example Problem 15: ETT and LOS Results

	Control Delay (s/veh) by Traffic Control Device					
Movement	First	Second	Third	EDTT (s/veh)	ETT (s/veh)	LOS
NB left	9.3	34.6	13.7	20.4	78.0	E
SB left	12.3	14.0	9.4	20.4	56.1	E
NB through	9.3	None	None	0	9.3	A
SB through	12.3	None	None	0	12.3	B
NB right	9.4	None	None	0	9.4	A
SB right	13.7	None	None	0	13.7	B
EB left	23.7	14.0	9.3	20.4	67.4	E
WB left	20.2	34.6	12.3	20.4	87.5	F
EB through	25.1	None	None	0	25.1	C
WB through	22.2	None	None	0	22.2	C
EB right	23.7	None	None	0	23.7	C
WB right	20.2	None	None	0	20.2	C

Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound.

Determination of Level of Service

LOS for each movement is shown above in Exhibit 34-138. The results were obtained by using Exhibit 23-13, having established that the v/c ratio was less than 1.0 at all junctions and that the queue-to-storage ratios were well below 1.0 for the 500-ft bay lengths provided.

Discussion

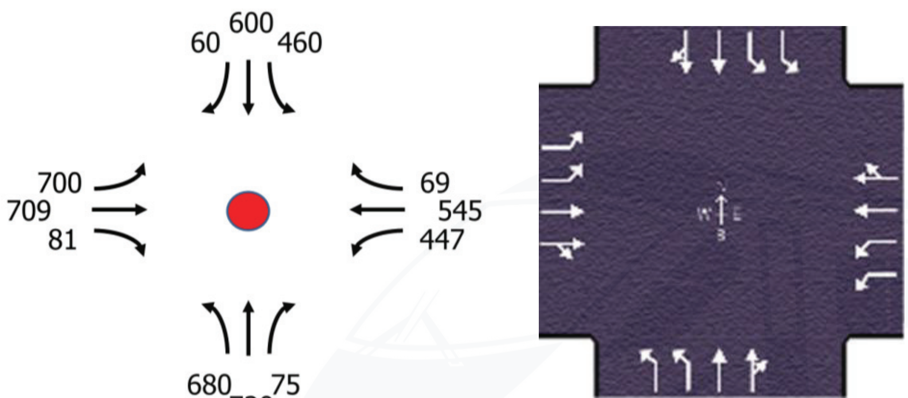
MUT and RCUT intersections are particularly aided by right turns and U-turns on red because the demands for those movements are relatively higher than at conventional intersections. If right turns on red were allowed from the minor-street approaches in this case, where there are exclusive right-turn lanes, the Chapter 23 example results in Part C show the effects on ETT. If 40% of the right-turning volume (which includes the traffic that will eventually turn left) is able to turn on red, with an estimated zero control delay, ETT will be reduced by more than 11 s/veh for some of the minor-street movements, which will change LOS by one level in some cases.

EXAMPLE PROBLEM 16: PARTIAL DISPLACED LEFT-TURN INTERSECTION

The Intersection

The intersection of Speedway Boulevard (east–west) and Campbell Avenue (north–south) has multiple failing movements and heavy left-turn demands. Many of the nonfailing movements are close to failing, and future traffic growth is a concern. Exhibit 34-139 provides the intersection volumes and channelization, and Exhibit 34-140 provides the signalization information. Volumes (hourly flow rates) listed in Exhibit 34-139 are only valid during the peak 15-min period.

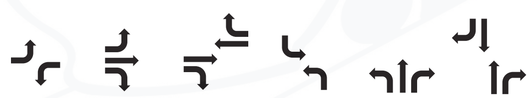
Exhibit 34-139
Example Problem 16: Intersection Volumes and Channelization



(a) Peak 15-min Volumes (veh/h)

(b) Lane Channelization

Exhibit 34-140
Example Problem 16: Intersection Signalization



Green (s)	20.9	5.9	23.0	21.6	4.4	26.0
Yellow (s)	4.0	4.0	4.0	4.0	4.0	4.0
Red (s)	1.0	1.0	1.0	1.0	1.0	1.0

The Question

Will displacing the left turns on the major street significantly improve performance of this intersection?

The Facts

No other signalized intersections exist within 1 mi. The intersection is controlled by a fully actuated signal, with no right turns on red allowed. There are no heavy vehicles, and the PHF is estimated to be 0.92. The start-up lost time and the 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 intersection.

Solution

The analyst wishes to evaluate potential improvements when the east–west left turns are displaced 350 ft upstream of the main intersection. These upstream locations are now classified as the supplemental intersections. In the HCM context, a DLT intersection analysis can be considered an extension of the urban streets procedure. Thus, definitions of volume, geometric, and signalization data for an urban street having three intersections are necessary at this stage.

Determination of Movement Demands

Exhibit 34-141 illustrates the demand volumes at each intersection in the partial DLT configuration. The displaced eastbound and westbound left-turn volumes are assumed to be zero at the main intersection, according to Step 1 of the DLT computational procedure. At the western supplemental intersection, eastbound through (709 veh/h) and right-turn (81 veh/h) demands at the main intersection are combined into a single through (790 veh/h) demand. Similarly, three feeding demands (northbound left, westbound through, and southbound right) at the main intersection are combined into a westbound through (1,285 veh/h) demand. Similar flow aggregations are made at the eastern supplemental intersection. Exhibit 34-142 illustrates lane geometries in the DLT configuration.

Exhibit 34-141
Example Problem 16: Flow Rates at the Supplemental and Main Intersections

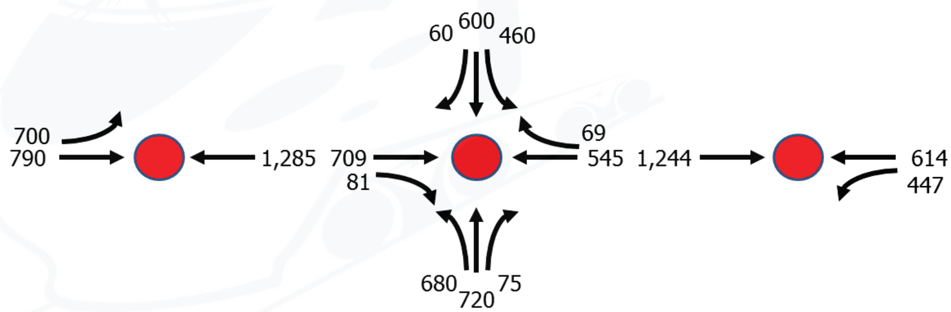
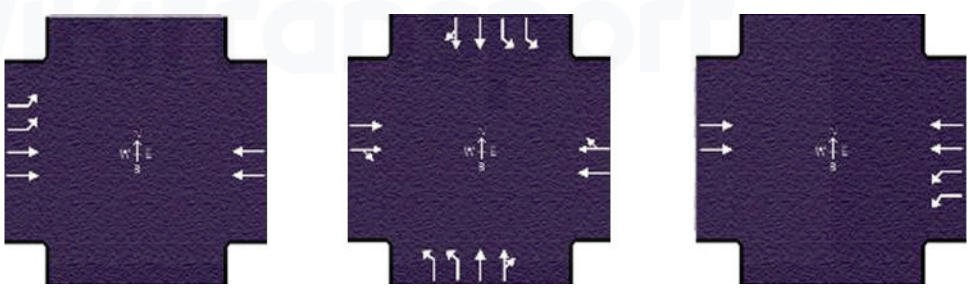


Exhibit 34-142
Example Problem 16: Lane Geometries at the Supplemental and Main Intersections



Determination of Lane Groups, Lane Utilization, and Signal Progression Adjustments

Steps 2 through 4 of the DLT procedure involve lane group determination, lane utilization, and arrival type adjustments, respectively. Lane group determination and lane utilization are performed by the Chapter 19, Signalized Intersections, procedures. Arrival type adjustments are handled by the flow profile analysis from Chapter 18, Urban Street Segments.

Calculation of Additional Control-Based Adjustments

In Step 5 of the DLT procedure, a right-turn saturation flow rate adjustment factor is applied to the left-turn movements at the supplemental intersections. In addition, signalization offsets must be set such that displaced left-turn vehicles always arrive during the guaranteed green window at the main intersection. The signalization information provided in Exhibit 34-140 should no longer be used in a potential DLT configuration, because the major-street left-turn phases will no longer exist at the main intersection. To ensure proper coordination, the supplemental intersections must have the same cycle length as the main intersection, and major-street through phases must now be treated as non-actuated phases. Exhibit 34-143 provides the new timing plans at each intersection. The new timing plans were generated by an alternative tool for signal optimization.

Exhibit 34-143
Example Problem 16: Signalization at the DLT Intersections

Supplemental Intersection Timing Plans

Green (s)	18.7	36.3	17.0	11.8	4.8	16.4	12.7	42.3
Yellow (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Red (s)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Main Intersection Timing Plan

After the overall new timing plans are determined, the signalization offsets can be recalculated according to Step 5. The following steps represent the offset computation process for DLT intersections in Chapter 23:

1. Determine the travel distance for (i.e., segment length of) the displaced left-turn roadway TD_{DLT}, in feet. The displaced left-turn roadway is the roadway used by displaced left-turning vehicles as they travel from the upstream crossover at the supplemental intersection to the stop bar at the main intersection. In this case, the distance is 350 ft.
2. Compute the left-turn travel time TT_{DLT} with Equation 23-61:

$$TT_{DLT} = \frac{TD_{DLT}}{S_{f,DLT} \times 1.47}$$

$$TT_{DLT} = \frac{350}{35 \times 1.47} = 6.8 \text{ s}$$

3. For the upstream supplemental intersection, obtain the duration between the reference point and the start of the displaced left-turn phase LAG_{DLT}, in seconds. For the downstream main intersection, obtain the duration between the reference point and the start of the major-street through phase LAG_{TH}, in seconds. These durations should be based on input phase splits instead of output phase durations.

   In this example, the reference point at all intersections is assumed to be the end of the major-street through phase. From Exhibit 34-143, the supplemental intersection's displaced left-turn phases always begin exactly when the major-street through phases end, so that LAG_{DLT} is equal to zero.

Exhibit 34-143 indicates that at the main intersection, after the major-street through phase ends, the signal must cycle through all minor-street phases before reaching a point where the major-street through phase begins. However, Exhibit 34-143 illustrates average phase durations. To determine the window of green time that is guaranteed to occur on the major street, it is necessary to observe what the timing plan would be if actuated phases were driven to their maximum durations. Exhibit 34-144 illustrates this timing plan.

Exhibit 34-144
Example Problem 16: Maximum Phase Times at the Main Intersection

Green (s)	8.0	21.0	1.0	15.0
Yellow (s)	4.0	4.0	4.0	4.0
Red (s)	1.0	1.0	1.0	1.0

Thus LAG_{TH} is equal to 21 + 4 + 1 + 1 + 4 + 1 + 15 + 4 + 1 = 52 s. This means that the major-street through phase begins 52 s after the reference point.

4. Obtain the offsets at the upstream supplemental intersection O_{SUPP} and the downstream main intersection O_{MAIN}, both in seconds.

   In this example, the initial offsets at all intersections are assumed equal to 0 s. When an existing DLT intersection having nonzero offsets is evaluated, the existing offsets would be assigned here.

5. Compute the system start time of the displaced left-turn phase ST_{DLT}, in seconds, for the upstream crossover at the supplemental intersection, by using Equation 23-62:

$$ST_{DLT} = LAG_{DLT} + O_{SUPP}$$
$$ST_{DLT} = 0 + 0 = 0 \text{ s}$$

6. Compute the system start time of the major-street through phase ST_{TH} at the main intersection by using Equation 23-63:

$$ST_{TH} = LAG_{TH} + O_{MAIN}$$
$$ST_{TH} = 52 + 0 = 52 \text{ s}$$

7. Change O_{SUPP} so that ST_{TH} is equal to $ST_{DLT} + TT_{DLT}$ by using Equation 23-64:

$$O_{SUPP} = O_{SUPP} - ST_{DLT} + ST_{TH} - TT_{DLT}$$
$$O_{SUPP} = 0 - 0 + 52 - 7 = 45 \text{ s}$$

8. If the offset value is greater than the background cycle length value, decrement the offset value by the cycle length C to obtain an equivalent offset within the valid range.

   In this example, the new offset value of 45 s is not greater than the cycle length value of 65 s.

9. If any offset value is lower than zero, increment the offset value by the cycle length to obtain an equivalent offset within the valid range.

   In this example, the new offset value of 45 s is not lower than zero. Thus, when the offset is set to 45 s at the supplemental intersections, displaced left-turn vehicles are expected to pass through the main intersection without stopping.

Calculation of Junction-Specific Performance Measures

After the offset calculation in Step 5, Step 6 of the alternative intersection procedure estimates the *v/c* ratio and control delay at each intersection. Steps 7 through 9 are not applicable to DLT intersections, and Step 10 is the LOS determination.

For the conventional intersection design from Exhibit 34-139, intersection-wide control delay is calculated as 64.1 s/veh by using Chapter 19 methods. For the DLT intersection design from Exhibit 34-141, after Steps 1 through 5 of the alternative intersection procedure are used to adjust the input data, *v/c* and control delay for each isolated turn movement can be calculated by using methods from Chapters 18 and 19. However at the overall DLT facility, turn movement–specific control delays are encountered sequentially at each intersection, as shown in Exhibit 34-145.

Exhibit 34-145
Example Problem 16: Weighted Average Control Delays

Move-	Flows				Delays			Products		
ment	**Orig.**	**Int 1**	**Int 2**	**Int 3**	**Int 1**	**Int 2**	**Int 3**	**Int 1**	**Int 2**	**Int 3**
EB L	761	761			22.5			17,123	0	0
EB TH	437	859	437	1,352	0.4	41.9	2.5	344	18,310	3,380
EB R	422		422			42.5		0	17,935	0
WB L	486			486			25.7	0	0	12,490
WB TH	340	1,397	340	667	4.0	29.3	0.4	5,588	9,962	267
WB R	328		328			29.7		0	9,742	0
NB L	739		739			23.7		0	17,514	0
NB TH	439		439			19.8		0	8,692	0
NB R	425		425			19.8		0	8,415	0
SB L	500		500			26.2		0	13,100	0
SB TH	364		364			23.4		0	8,518	0
SB R	353		353			23.5		0	8,296	0
Total	5,594								159,675	
Avg.									28.5	

Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound, TH = through, L = left, R = right, Orig. = original (non-DLT) intersection, Int = intersection, Avg. = average.

Determination of Level of Service

Comparison of the conventional intersection delay of 64.1 s/veh with the alternative intersection delay of 28.5 s/veh indicates that the alternative design is expected to offer a 55% average delay reduction while processing the same number (5,594) of vehicle trips. For DLT intersections, experienced travel time (ETT) can be assumed as equal to control delay. According to the LOS thresholds given in Chapter 19, Signalized Intersections, the overall DLT intersection would operate at LOS C, in contrast to the conventional intersection operating at LOS E. This raises the question of what might happen if left turns could be displaced on all four intersection approaches. This is the subject of Example Problem 17.

Validity Checks

Chapter 23 cites a number of conditions that would invalidate the DLT analysis method. If any of these conditions are met, the analysis results are unreliable, and alternative tool analysis is recommended:

- Displaced left-turn vehicles are significantly delayed at the main intersection,
- Displaced left-turn approach's through and left-turning movements are not served by exactly the same signal phasing and timing,
- Green times at the main intersection are not long enough to serve displaced left-turning vehicle demands fully, or
- Side street green durations do not exceed the sum of (*a*) main street travel time between supplemental and main intersections and (*b*) displaced left-turn queue clearance time.

EXAMPLE PROBLEM 17: FULL DISPLACED LEFT-TURN INTERSECTION

The Intersection

The conventional intersection conditions in Example Problem 17 are identical to those given in Example Problem 16, before DLT conversion.

The Question

Will displacement of left-turn movements on all four approaches significantly improve performance of this intersection?

The Facts

The facts of the example problem are the same as in Example Problem 16.

Solution

The analyst wishes to evaluate potential improvements when left turns on all four approaches are displaced 350 ft upstream of the main intersection. In this case, two partial DLT analyses must be performed: one for the major street and one for the minor street.

Determination of Movement Demands (East–West Partial DLT Analysis)

Exhibit 34-146 illustrates the major-street flow rates. Displaced left-turn volumes are again assumed to be zero at the main intersection, according to Step 1 of the DLT computational procedure. Unlike partial DLT intersections, pseudo right-turn modeling adjustments are needed at full DLT intersections. Minor-street left-turn lanes have been converted to pseudo right-turn lanes on the opposite side of the intersection. Similarly, minor-street left-turn volumes have been combined with right-turn volumes on the opposite side of the intersection. Exhibit 34-147 further illustrates the lane geometries at all three intersections in the DLT configuration.

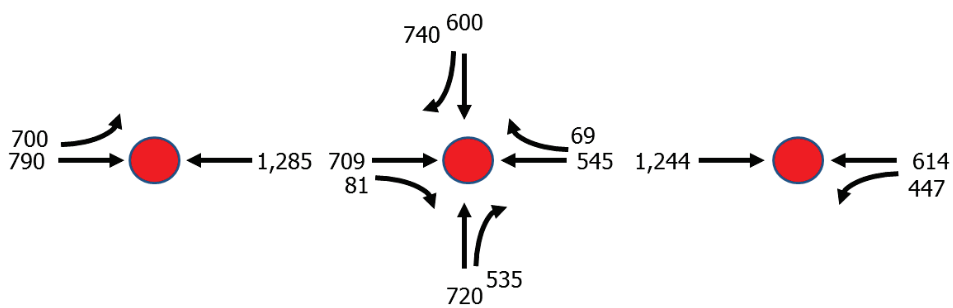


Exhibit 34-146
Example Problem 17: Flow Rates at the Supplemental and Main Intersections

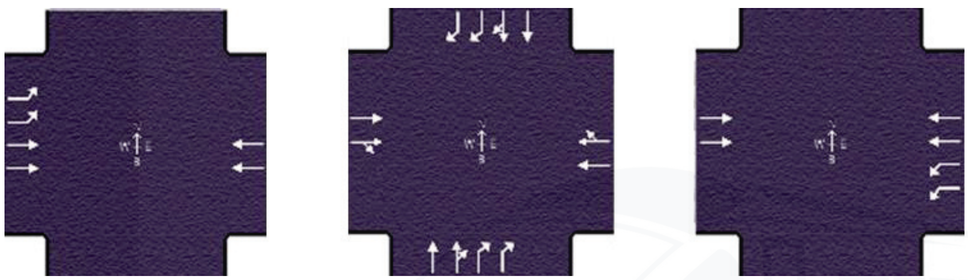


Exhibit 34-147
Example Problem 17: Lane Geometries at the Supplemental and Main Intersections

Determination of Lane Groups, Lane Utilization, and Signal Progression Adjustments (East–West Partial DLT Analysis)

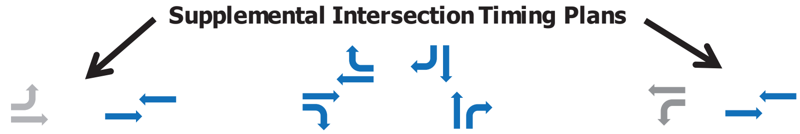
Steps 2 through 4 of the DLT procedure involve lane group determination, lane utilization, and arrival type adjustments, respectively. Lane group determination and lane utilization are performed by the Chapter 19, Signalized Intersections, procedures. Arrival type adjustments should be handled by the flow profile analysis from Chapter 18, Urban Street Segments.

Determination of Additional Control-Based Adjustments (East–West Partial DLT Analysis)

In Step 5 of the DLT procedure, a right-turn saturation flow rate adjustment factor is applied to the left-turn movements at the supplemental intersections. A left-turn saturation flow rate adjustment factor is applied to both pseudo right-turn movements at the main intersection. A start-up lost time of 0 s is assumed for both pseudo right-turn movements at the main intersection.

Signalization offsets must then be set to allow displaced left-turn vehicles to arrive during the guaranteed green window at the main intersection. The signalization information provided in Exhibit 34-140 should no longer be used in a potential DLT configuration, because the major-street left-turn phases will no longer exist at the main intersection. To ensure proper coordination, the supplemental intersections must have the same cycle length as the main intersection. Because of the full DLT configuration, all phases at the main intersection are nonactuated phases. Exhibit 34-148 illustrates new timing plans (in units of seconds) at each intersection. The new timing plans were generated by an alternative tool for signal optimization.

Exhibit 34-148
Example Problem 17: East–West Signalization at the DLT Intersections



Green (s)	13.4	21.6	17.0	18.0	9.2	25.8
Yellow (s)	4.0	4.0	4.0	4.0	4.0	4.0
Red (s)	1.0	1.0	1.0	1.0	1.0	1.0

Main Intersection Timing Plan

After the overall new timing plans are determined, signalization offsets can be recalculated according to Step 5. The following steps represent the offset computation process for DLT intersections in Chapter 23:

1. Determine the travel distance for (i.e., segment length of) the displaced left-turn roadway TD_{DLT}, in feet. The displaced left-turn roadway is the roadway used by displaced left-turning vehicles as they travel from the upstream crossover at the supplemental intersection to the stop bar at the main intersection. In this case, the distance is 350 ft.
2. Compute the left-turn travel time TT_{DLT} by using Equation 23-61:

$$TT_{DLT} = \frac{TD_{DLT}}{FFS_{DLT} \times 1.47}$$

$$TT_{DLT} = \frac{350}{35 \times 1.47} = 6.8 \text{ s}$$

3. For the upstream supplemental intersection, obtain the duration between the reference point and the start of the displaced left-turn phase LAG_{DLT}, in seconds. For the downstream main intersection, obtain the duration between the reference point and the start of the major-street through phase LAG_{TH}, in seconds. These durations should be based on input phase splits instead of output phase durations.

   In this example, the reference point at all intersections is assumed to be the end of the major-street through phase. From Exhibit 34-148, the supplemental intersection's displaced left-turn phases always begin exactly when the major-street through phases end, so that LAG_{DLT} is equal to zero.

   From Exhibit 34-148 at the main intersection, after the major-street through phase ends, the signal must cycle through the minor-street phase before reaching a point where the major-street through phase begins. For partial DLTs, it is necessary to observe what the timing plan would be if actuated phases were driven to their maximum durations, but for full DLTs, no phases are allowed to be actuated at the main intersection. Thus LAG_{TH} is equal to 18 + 4 + 1 = 23 s. This means that the major-street through phase begins 23 s after the reference point.

4. Obtain the offsets at the upstream supplemental intersection O_{SUPP} and the downstream main intersection O_{MAIN}, both in seconds.

   For this example, the initial offsets at all intersections are assumed equal to 0 s. When an existing DLT intersection having nonzero offsets is evaluated, the existing offsets would be assigned here.

5. Compute the system start time of the displaced left-turn phase ST_{DLT}, in seconds, for the upstream crossover at the supplemental intersection by using Equation 23-62:

$$ST_{DLT} = LAG_{DLT} + O_{SUPP}$$
$$ST_{DLT} = 0 + 0 = 0 \text{ s}$$

6. Compute the system start time of the major-street through phase ST_{TH} at the main intersection by using Equation 23-63:

$$ST_{TH} = LAG_{TH} + O_{MAIN}$$
$$ST_{TH} = 23 + 0 = 23 \text{ s}$$

7. Change O_{SUPP} so that ST_{TH} is equal to $ST_{DLT} + TT_{DLT}$ by using Equation 23-64:

$$O_{SUPP} = O_{SUPP} - ST_{DLT} + ST_{TH} - TT_{DLT}$$
$$O_{SUPP} = 0 - 0 + 23 - 7 = 16 \text{ s}$$

8. If the offset value is greater than the background cycle length value, decrement the offset value by the cycle length C to obtain an equivalent offset within the valid range.

   In this example, the new offset value of 16 s is not greater than the cycle length value of 45 s.

9. If any offset value is lower than zero, increment the offset value by the cycle length to obtain an equivalent offset within the valid range.

   In this example, the new offset value of 16 is not lower than zero. Thus, with offset values of 16 s at the east–west supplemental intersections, displaced left-turn vehicles are expected to pass through the main intersection without stopping. This completes the input data adjustments for a partial DLT analysis in the east–west direction.

North–South Partial DLT Analysis

Input data adjustments must now be performed for a second partial DLT analysis in the north–south direction. The cycle length of 45 s from the east–west partial DLT analysis must now be applied to the north–south partial DLT analysis. The main intersection timing plan from Exhibit 34-148 must not be changed in the north–south partial DLT analysis.

Step 1 of the north–south partial DLT analysis is similar to what was illustrated in Exhibit 34-146 and Exhibit 34-147. Steps 2 through 4 are again handled by the Chapter 19, Signalized Intersections, and Chapter 18, Urban Street Segments, procedures. In Step 5, a right-turn saturation flow rate adjustment factor is again applied to the supplemental intersection left-turn movements. A left-turn saturation flow rate adjustment factor is applied to both pseudo right-turn movements at the main intersection. A start-up lost time of 0 s is assumed for both pseudo right-turn movements at the main intersection.

Signalization offsets must now be set to allow displaced left-turn vehicles to arrive during the guaranteed green window at the main intersection. Before the offsets are calculated, green splits must be optimized in the north–south direction, while constrained to the cycle length of 45 s. Exhibit 34-149 illustrates new timing plans (in units of seconds) at each intersection. The new timing plans were generated by an alternative tool for signal optimization.

After the overall new timing plans are determined, signalization offsets can be recalculated according to Step 5. The north–south and east–west offset calculations are mostly identical. However, LAG_{TH} is now equal to 17 + 4 + 1 = 22 s, ultimately leading to 15-s offsets at the north–south supplemental intersections. With offset values of 15 s at the north–south supplemental intersections, displaced left-turn vehicles are expected to pass through the main intersection without stopping.

Exhibit 34-149
Example Problem 17: North–South Signalization at the DLT Intersections

Supplemental Intersection Timing Plans

Green (s)	13.3	21.7	17.0	18.0	9.4	25.6
Yellow (s)	4.0	4.0	4.0	4.0	4.0	4.0
Red (s)	1.0	1.0	1.0	1.0	1.0	1.0

Main Intersection Timing Plan

Calculation of Junction-Specific Performance Measures

After the offset calculation in Step 5, Step 6 of the alternative intersection procedure estimates the v/c ratio and control delay at each intersection. Steps 7 through 9 are not applicable to DLT intersections, and Step 10 is the LOS determination.

For the conventional intersection design from Exhibit 34-139, intersectionwide control delay is calculated as 64.1 s/veh by using Chapter 19's methods.

For the DLT intersection design, after Steps 1 through 5 of the alternative intersection procedure are used to adjust the input data, v/c ratio and control delay for each isolated turn movement can be calculated with methods from Chapter 19, Signalized Intersections, and Chapter 18, Urban Street Segments. However, for the overall DLT facility, turn movement–specific control delays are encountered sequentially at each intersection, as shown in Exhibit 34-150. To avoid double counting, minor-street performance measures are not tabulated in either of the two partial DLT analyses.

The full DLT delay computed here (29.0 s/veh) is similar to the partial DLT delay (28.5 s/veh) from Example Problem 16. For DLT intersections, experienced travel time can be assumed equal to control delay. According to Chapter 19's LOS thresholds, the overall DLT intersection would operate at LOS C, in contrast to the conventional intersection operating at LOS E.

Since the major-street and minor-street demands were all relatively heavy in Example Problems 16 and 17, the failure of the full DLT configuration to outperform the partial DLT configuration was surprising. However, when the same exercise was performed with 800-ft spacings between supplemental and main intersections, the full DLT (25.3 s/veh) outperformed the partial DLT (28.4 s/veh) by more than 10%. This shows that the DLT results are sensitive to intersection spacings and that intersection spacings should be taken into consideration in designing a new DLT facility.

Exhibit 34-150
Example Problem 17: Weighted Average Control Delays

Movement	Flows						Delays				
	Orig.	Int 1	Int 2	Int 3	Int 4	Int 5	Int 1	Int 2	Int 3	Int 4	Int 5
EB L	761	761					15.8				
EB TH	437	859	437	1,352			0.6	14.5	10.4		
EB R	422		422					14.6			
WB L	486			486					17.5		
WB TH	340	1,397	340	667			17.9	12.8	0.5		
WB R	328		328					12.9			
NB L	739				739					15.2	
NB TH	439		439		864	1,618		13.1		0.6	14.2
NB R	425		425					13.2			
SB L	500					500					17.4
SB TH	364		364		1,226	717		12.2		13.8	0.5
SB R	353		353					12.3			
Total	5,594										

Movement	Products				
	Int 1	Int 2	Int 3	Int 4	Int 5
EB L	12,024	0	0	0	0
EB TH	515	6,337	14,061	0	0
EB R	0	6,161	0	0	0
WB L	0	0	8,505	0	0
WB TH	25,006	4,352	334	0	0
WB R	0	4,231	0	0	0
NB L	0	0	0	11,233	0
NB TH	0	5,751	0	518	22,976
NB R	0	5,610	0	0	0
SB L	0	0	0	0	8,700
SB TH	0	4,441	0	16,919	359
SB R	0	4,342	0	0	0
Total				162,373	
Average				29.0	

Notes: EB = eastbound, WB = westbound, NB = northbound, SB = southbound, TH = through, L = left, R = right, Orig. = original (non-DLT) intersection, Int = intersection.

Validity Checks

Chapter 23 cites a number of conditions that would invalidate the DLT analysis method. If any of these conditions are met, the analysis results are unreliable, and alternative tool analysis is recommended:

- Displaced left-turn vehicles are significantly delayed at the main intersection,
- The displaced left-turn approach's through and left-turning movements are not served by exactly the same signal phasing and timing,

- Green times at the main intersection are not large enough to serve displaced left-turning vehicle demands fully, or
- Side street green durations do not exceed the sum of (*a*) main street travel time between supplemental and main intersections and (*b*) displaced left-turn queue clearance time.

3. OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

INTRODUCTION

The operational analysis for interchange type selection can be used to evaluate the operational performance of various interchange types. It allows the user to compare eight fundamental types of interchanges for a given set of demand flows. The eight signalized interchange types covered by the interchange type selection analysis methodology are as follows:

1. SPUI,
2. Tight urban diamond interchange (TUDI),
3. Compressed urban diamond interchange (CUDI),
4. Conventional diamond interchange (CDI),
5. Parclo A—four quadrants (Parclo A-4Q),
6. Parclo A—two quadrants (Parclo A-2Q),
7. Parclo B—four quadrants (Parclo B-4Q), and
8. Parclo B—two quadrants (Parclo B-2Q).

Other types of signalized interchanges cannot be investigated with this interchange type selection analysis methodology. Also, the operational analysis methodology does not distinguish between the TUDI, CUDI, and CDI types. In general, the interchange type selection analysis methodology categorizes diamond interchanges by the distance between the centerlines of the ramp roadways that form the signalized intersections. This distance is generally between 200 and 400 ft for the TUDI, between 600 and 800 ft for the CUDI, and between 1,000 and 1,200 ft for the CDI.

The method is based on research (*4*). The research also provides a methodology for selecting unsignalized interchanges. Since unsignalized interchanges are not covered by Chapter 23, users should consult the original source for this information.

The methodology is based on the estimation of the sums of critical flow ratios through the interchange and their use to estimate interchange delay. A combination of simulation and field data was used to develop critical relationships for the methodology.

The sum of critical flow ratios is based on an identification of all flows served during a particular signal phase and the determination of maximum flow ratios among the movements served by that phase. The models are similar to those used in Chapter 19 for signalized intersections; they are modified to take into account the fact that each signal phase involves two signalized intersections. Interchange delay is defined as the total of all control delays experienced by all interchange movements involved in signalized ramp terminal movements divided by the sum of all external movement flows. Additional information is available in the source report (*4*).

Because signalization is not specified for an interchange type selection analysis, the following interchange types are assumed to be operated by a single signal controller: SPUI, TUDI, and CUDI. All other types are assumed to be operated by separate controllers at each signalized ramp terminal. In all cases, optimal signal timing and phasing are assumed.

INPUTS AND APPLICATIONS

This interchange type selection analysis methodology can be used in several ways:

1. For a given set of O-D interchange movements, eight basic types of signalized interchanges may be compared on the basis of interchange delay.
2. For a given type of interchange, the impact of intersection spacing on interchange delay can be examined (within the range of applicability for each interchange type).
3. For a given type of interchange, the impact of the number of lanes on ramp and surface arterial approaches and the movements assigned to these lanes can be examined, again by using interchange delay as the measure of effectiveness.

For any of these applications, all interchange O-D movements must be specified, generally by using full peak-hour volumes. The interchange type selection methodology is not detailed enough to use flow rates or to consider such factors as the presence of heavy vehicles.

In addition, for any given computation, the number of lanes assigned to each phase movement and the distance between the centerlines of the two ramps, measured along the surface arterial, must be specified.

SATURATION FLOW RATES

Implementation of the interchange type selection methodology requires the adoption of default values for saturation flow rate. Research (*3*) suggests the use of 1,900 veh/hg/ln for some basic cases. However, this is based on a suggested base saturation flow rate of 2,000 pc/hg/ln, which is higher than the default values suggested in Chapter 19, Signalized Intersections. For consistency with the base saturation flow rate of 1,900 pc/hg/ln specified in Chapter 19 and to recognize the impact of various movements on saturation flow rate, the default values shown in Exhibit 34-151 are recommended for use in conjunction with the interchange type selection methodology. Alternatively, if relevant information is available, the default values provided in Chapter 19 (Exhibit 19-11 and Exhibit 19-12) may be used. Where turning movements are in shared lanes, the "through" saturation flow rates should be used for analysis.

Interchange Type	Default Saturation Flow Rate (veh/hg/ln) Left Turns	Through	Right Turns
SPUI	1,800	1,800	1,800
TUDI	1,700	1,800	1,800
CUDI	1,700	1,800	1,800
CDI	1,700	1,800	1,800
Parclo A-4Q	1,700	1,800	1,800
Parclo A-2Q	1,700	1,800	1,800
Parclo B-4Q	1,700	1,800	1,800
Parclo B-2Q	1,700	1,800	1,800

Exhibit 34-151
Default Values of Saturation Flow Rate for Use with the Operational Analysis for Interchange Type Selection

COMPUTATIONAL STEPS

Step 1: Mapping O-D Flows into Interchange Movements

Since the primary objective of an interchange type selection analysis is to compare up to eight interchange types against a given set of design volumes, conversion of a given set of design origin and destination volumes to movement flows through the signalized interchange is necessary first. The methodology identifies volumes by signal phase by using the standard NEMA numbering sequence for interchange phasing. Thus, movements are numbered 1 through 8 on the basis of the signal phase that accommodates the movement. Not all configurations and signalizations include all eight NEMA phases, and for some interchange forms some movements are not signalized and do not, therefore, contribute to interchange delay.

As for the operational analysis methodology, to simplify the mapping process, the freeway is assumed to be oriented north–south and the surface arterial east–west. If the freeway is oriented in the east–west direction, rotate the interchange drawing or diagram clockwise until the freeway is in the north–south direction. In rotating clockwise, the westbound freeway direction becomes northbound and the eastbound freeway direction becomes southbound; the northbound arterial direction becomes eastbound and the southbound arterial direction becomes westbound. The methodology allows for separate consideration of freeway U-turn movements through the interchange. Thus, 14 basic movements must be mapped for each interchange type.

For interchange types using two controllers, phase movements through the left (Intersection I) and right (Intersection II) intersections of the interchange are separately mapped and used in the procedure.

Exhibit 34-152 indicates the appropriate mapping of O-D demand volumes into phase movement volumes for the eight covered interchange types. The designation of the O-D demands is shown in Exhibit 34-162. The mapped phase movement volumes are then used in Step 2 to compute critical flow ratios.

Exhibit 34-152
Mapping of Interchange Origins and Destinations into Phase Movements for Operational Interchange Type Selection Analysis

Interchange Type	NEMA Phase Movement Number 1	2	3	4	5	6	7	8
SPUI	H	I+**F**	A+M	**C**	E	J+**G**	D+N	**B**
TUDI /CUDI	H+M	E+I+**F**	--	D+**C**+N	E+N	H+J+**G**	--	A+M+**B**
CDI (I)	H+M	E+I+**F**	--	D+**C**+N	--	J+A	--	--
CDI (II)	--	I+D	--	--	E+N	H+J+**G**	--	A+M+**B**
Parclo A-4Q (I)	--	E+I	--	D+N+**C**	--	J+A+**M**+**H**	--	--
Parclo A-4Q (II)	--	I+D+**N**+**E**	--	--	--	J+H	--	A+M+**B**
Parclo A-2Q (I)	--	E+I	--	D+N+**C**	F	J+A+**H**+**M**	--	--
Parclo A-2Q (II)	G	I+D+**E**+**N**	--	--	--	H+J	--	A+M+**B**
Parclo B-4Q (I)	H+M	I+E+**F**	--	--	--	J+A	--	--
Parclo B-4Q (II)	--	I+D	--	--	E+N	H+J+**G**	--	--
Parclo B-2Q (I)	H+M	E+I+**F**	--	--	--	J+A	--	C
Parclo B-2Q (II)	--	I+D	--	B	E+N	H+J+**G**	--	--

Notes: -- indicates that phase movement does not exist for this interchange configuration.
Bold indicates movements not included when they operate from a separate lane with YIELD or STOP control.

Step 2: Computation of Critical Flow Ratios

The subsections that follow detail the computation of the critical flow ratio Y_c for the interchange for the eight basic configurations covered by this methodology.

Single-Point Urban Interchange

The phase movements in a SPUI are illustrated in Exhibit 34-153. The sum of critical flow ratios is estimated as follows:

Equation 34-1

$$Y_c = A + R$$

with

Equation 34-2

$$A = \max\left[\left(\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2}\right), \left(\frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6}\right)\right]$$

Equation 34-3

$$R = \max\left[\left(\frac{v_3}{s_3 n_3} + \frac{v_4}{s_4 n_4}\right), \left(\frac{v_7}{s_7 n_7} + \frac{v_8}{s_8 n_8}\right)\right]$$

where

Y_c = sum of the critical flow ratios,

v_i = phase movement volume for phase i (veh/h),

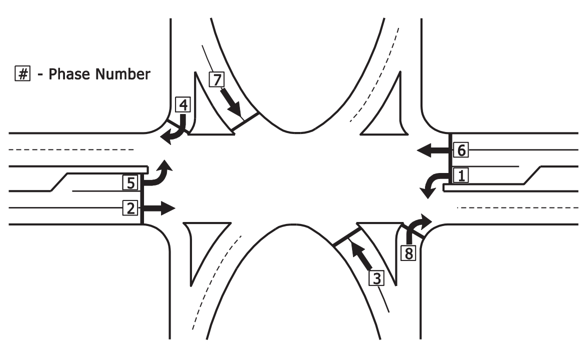
n_i = number of lanes serving phase movement i,

s_i = saturation flow rate for phase movement i (veh/hg/ln),

A = critical flow ratio for the arterial movements, and

R = critical flow ratio for the exit ramp movements.

Exhibit 34-153
Phase Movements in a SPUI

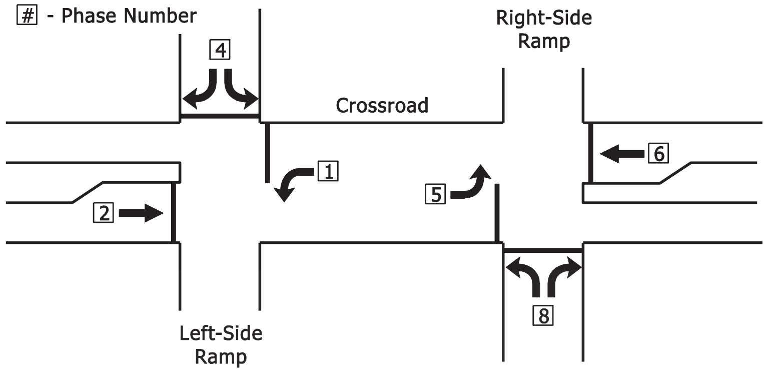


Source: Bonneson et al. (*4*).

Tight Urban Diamond Interchange

Phase movements in a TUDI are illustrated in Exhibit 34-154.

Exhibit 34-154
Phase Movements in a Tight Urban or Compressed Urban Diamond Interchange



Source: Bonneson et al. (*4*).

The sum of critical flow ratios is computed as follows:

Equation 34-4

$$Y_c = A + R$$

with

Equation 34-5

$$A = \max\left[\left(\frac{v_2}{s_2 n_2} + \frac{v_4}{s_4 n_4}\right) - y_3, \left(\frac{v_5}{s_5 n_5} + y_7\right)\right]$$

Equation 34-6

$$R = \max\left[\left(\frac{v_1}{s_1 n_1} + y_3\right), \left(\frac{v_6}{s_6 n_6} + \frac{v_8}{s_8 n_8} - y_7\right)\right]$$

Equation 34-7

$$y_3 = \min\left(\frac{v_4}{s_4 n_4}, y_t\right)$$

Equation 34-8

$$y_7 = \min\left(\frac{v_8}{s_8 n_8}, y_t\right)$$

where y_3 and y_7 are the effective flow ratios for concurrent (or transition) Phases 3 and 7, respectively; and y_t is the effective flow ratio for the concurrent phase when dictated by travel time.

For preliminary design applications, the default values of Exhibit 34-155 are recommended for y_t. The distance between the two intersections is measured from the centerline of the left ramp roadway to the centerline of the right ramp roadway.

Exhibit 34-155
Default Values for y_t

Distance Between Intersections *D′* (ft)	Default Value for y_t
200	0.050
300	0.070
400	0.085

For Phase Movements 2 and 6, the number of assigned lanes (n_2 and n_6) is related to the arterial left-turn bay design. If the left-turn bay extends back to the external approach to the interchange, the number of lanes on these external approaches is the total number of approaching lanes, including the left-turn bay. If the left-turn bay is provided only on the internal arterial link, n_2 or n_6, or both, would not include this lane.

Compressed Urban Diamond Interchange

Exhibit 34-154 illustrates the phase movement volumes for a CUDI. They are the same as for a TUDI. The sum of critical flow ratios is computed as follows:

Equation 34-9

$$Y_c = A + R$$

with

Equation 34-10

$$A = \max\left[\left(\frac{v_1}{s_1 n_1} + y_2\right), \left(\frac{v_5}{s_5 n_5} + y_6\right)\right]$$

Equation 34-11

$$R = \max\left(\frac{v_4}{s_4 n_4}, \frac{v_8}{s_8 n_8}\right)$$

Equation 34-12

$$y_2 = \max\left(\frac{v_2}{s_2 n_2}, \frac{v_5}{s_2}\right)$$

Equation 34-13

$$y_6 = \max\left(\frac{v_8}{s_8 n_8}, \frac{v_1}{s_6}\right)$$

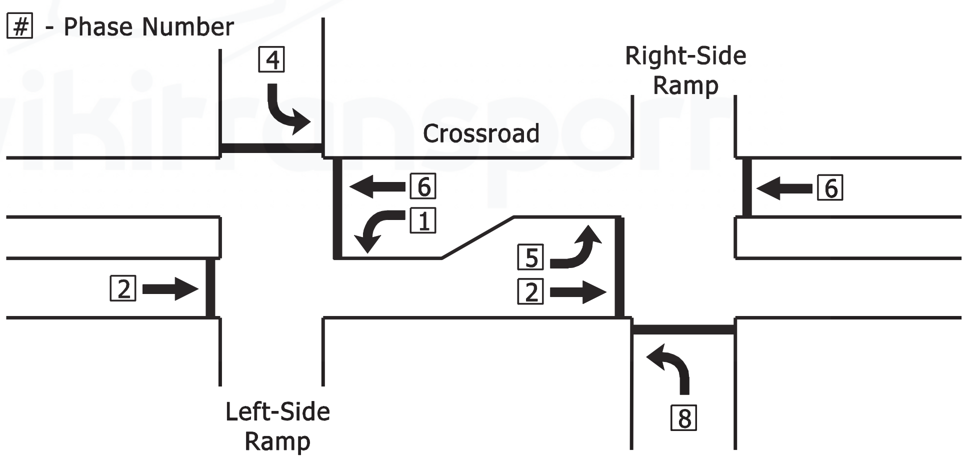
where y_2 and y_6 are the flow ratios for Phases 2 and 6, respectively, with consideration of pre-positioning.

All Interchanges with Two Signalized Intersections and Separate Controllers

These interchange types include CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q. The computation of the maximum sum of critical volumes is the same for each. Each has two signalized intersections, and each is generally operated with two controllers.

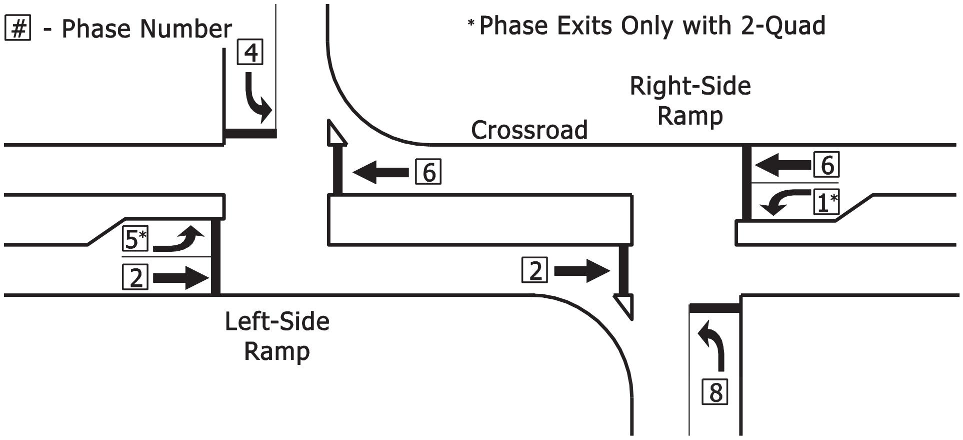
While the equations for estimating the maximum sum of critical volumes are the same, the phase movement volumes differ for each type of interchange, as was indicated in Exhibit 34-152. Exhibit 34-156 through Exhibit 34-158 illustrate the phase movements for each of these interchange types.

Exhibit 34-156
Phase Movements in a CDI



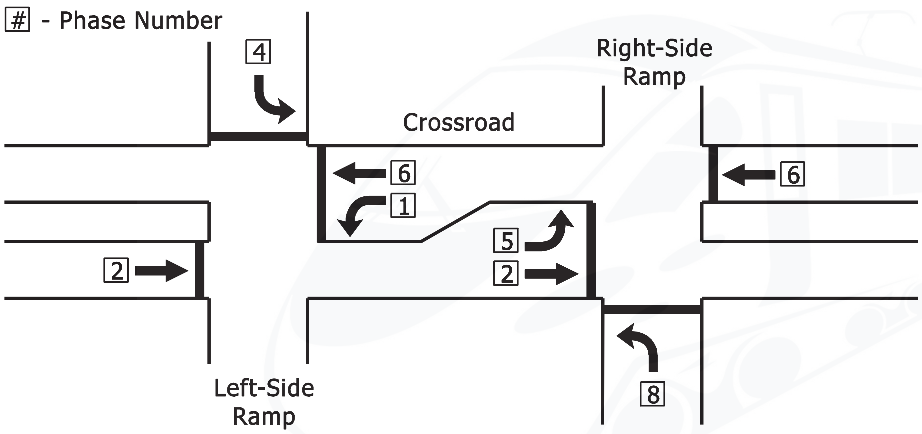
Source: Bonneson et al. (*4*).

Exhibit 34-157
Phase Movements in Parclo A-2Q and A-4Q Interchanges



Source: Messer and Bonneson (*3*).

Exhibit 34-158
Phase Movements in Parclo B-2Q and B-4Q Interchanges



Source: Messer and Bonneson (*3*).

For all conventional diamond, Parclo A, and Parclo B interchanges, the sum of critical flow ratios is computed as follows:

$$Y_{c,max} = \max\left(Y_{c,\mathrm{I}}, Y_{c,\mathrm{II}}\right) \quad \textbf{Equation 34-14}$$

with

$$Y_{c,\mathrm{I}} = A_\mathrm{I} + R_\mathrm{I} \quad \textbf{Equation 34-15}$$

$$Y_{c,\mathrm{II}} = A_\mathrm{II} + R_\mathrm{II} \quad \textbf{Equation 34-16}$$

$$A_{\mathrm{I,II}} = \max\left[\left(\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2}\right), \left(\frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6}\right)\right] \quad \textbf{Equation 34-17}$$

$$R_{\mathrm{I,II}} = \max\left(\frac{v_4}{s_4 n_4}, \frac{v_8}{s_8 n_8}\right) \quad \textbf{Equation 34-18}$$

where

$Y_{c,\mathrm{I}}$ = sum of the critical flow ratios for Intersection I,

$Y_{c,\mathrm{II}}$ = sum of the critical flow ratios for Intersection II,

$Y_{c,\max}$ = sum of the critical flow ratios for the interchange,

A_I = critical flow ratio for the arterial movements for Intersection I,

A_II = critical flow ratio for the arterial movements for Intersection II,

$A_{I,II}$ = critical flow ratio for the arterial movements for the interchange,

R_I = critical flow ratio for the exit-ramp movements for Intersection I,

R_{II} = critical flow ratio for the exit-ramp movements for Intersection II, and

$R_{I,II}$ = critical flow ratio for the exit-ramp movements for the interchange.

Note that when values of A_I, A_{II}, R_I, and R_{II} are computed, the movement volumes vary for Intersections I and II, even though the phase movement designations are the same (Exhibit 34-152).

Some of the phase movement volumes do not exist in either Intersection I or II. A value of 0 is used for the volume in each case where this occurs.

Step 3: Estimation of Interchange Delay

Interchange delay for each interchange type or design is estimated by using regression models that were developed primarily from simulation output but validated with a limited amount of field data (*4*). In each case, two delay estimators are provided on the basis of the control of the off-ramp right-turn movements:

- Case A, used where the right-turn movements from freeway off-ramps are controlled by the signal.
- Case B, used where the right-turn movements from freeway off-ramps have a separate lane or lanes that are either free (uncontrolled) or controlled by a YIELD sign.

For SPUIs, a third condition is added. Where the right turns from the freeway ramps are controlled by a signal and right turn on red is allowed, both cases are used, and the results are weighted by the proportions of right turns made during the red and green indications. Since the signal timing is unknown for an interchange type selection application, the assumption of a 50%/50% split is recommended.

This modification, applied only to SPUIs, is necessary due to difficulties experienced in simulating right turn on red at these interchanges.

Exhibit 34-159 gives the delay equations used to estimate interchange delay for the eight interchange types covered by the interchange type selection procedure. In each case, the variables used are defined as follows:

d = interchange delay (s/veh);

Y_c = critical or controlling flow ratio from Step 1; and

D' = distance between the two intersections, measured between the centerlines of the two ramp roadways along the surface arterial (ft).

Exhibit 34-159 also shows the ranges of D' over which these equations are valid. They generally represent the normal design range for these interchange types. These equations should be used with great caution beyond these ranges.

Exhibit 34-159
Estimation of Interchange Delay d_I for Eight Basic Interchange Types

Interchange Type	Valid Range of D' (ft)	Case A: Right Turns Signalized	Case B: Right Turns Free or YIELD-Controlled
SPUI	150–400	$15.1 + (16.0 + 0.01D)\left(\frac{Y_c}{1-Y_c}\right)$	$15.1 + (5.9 + 0.008D)\left(\frac{Y_c}{1-Y_c}\right)$
TUDI	200–400	$13.4 + 14.2\left(\frac{Y_c}{1-Y_c}\right)$	$13.4 + 12.8\left(\frac{Y_c}{1-Y_c}\right)$
CUDI	600–800	$19.2 + [9.4 - 0.011(D - 700)]\left(\frac{Y_c}{1-Y_c}\right)$	$19.2 + [8.6 - 0.009(D - 700)]\left(\frac{Y_c}{1-Y_c}\right)$
CDI	900–1,300	$17.1 + [5.0 - 0.011(D - 1{,}100)]\left(\frac{Y_c}{1-Y_c}\right)$	$17.1 + [4.6 - 0.009(D - 1{,}100)]\left(\frac{Y_c}{1-Y_c}\right)$
Parclo A-4Q	700–1,000	$11.7 + [7.8 - 0.011(D - 800)]\left(\frac{Y_c}{1-Y_c}\right)$	$11.7 + [6.6 - 0.009(D - 800)]\left(\frac{Y_c}{1-Y_c}\right)$
Parclo A-2Q	700–1,000	$19.1 + [8.3 - 0.011(D - 800)]\left(\frac{Y_c}{1-Y_c}\right)$	$19.1 + [8.3 - 0.009(D - 800)]\left(\frac{Y_c}{1-Y_c}\right)$
Parclo B-4Q	1,000–1,400	$9.3 + [3.5 - 0.011(D - 1{,}200)]\left(\frac{Y_c}{1-Y_c}\right)$	$9.3 + [3.4 - 0.009(D - 1{,}200)]\left(\frac{Y_c}{1-Y_c}\right)$
Parclo B-2Q	1,000–1,400	$26.2 + [3.9 - 0.011(D - 1{,}200)]\left(\frac{Y_c}{1-Y_c}\right)$	$26.2 + [3.2 - 0.009(D - 1{,}200)]\left(\frac{Y_c}{1-Y_c}\right)$

Delay estimates can be related to LOS. For consistency, the same criteria as used for the operational analysis methodology (*4*) are applied. Because LOS F is based on a *v*/*c* ratio greater than 1.00 or a queue storage ratio greater than 1.00, this interchange type selection methodology will never predict LOS F, because it does not predict these ratios. Users should be exceedingly cautious of results when interchange delay exceeds 85 to 90 s/veh.

In evaluating alternative interchange types, the exact distance, D', may not be known for each of the alternatives. It is recommended that all lengths be selected at the midpoint of the range shown in Exhibit 34-159 for this level of analysis.

Interpretation of Results

The output of the interchange type selection procedure for signalized interchanges is a set of delay predictions for (*a*) various interchange types, (*b*) various distances D' between the two intersections, or (*c*) various numbers and assignments of lanes on ramps and the surface arterials.

Although a lower interchange delay is generally better, a final choice must consider a number of other criteria that are not part of this methodology, including the following:

- Availability of right-of-way,
- Environmental impacts,
- Social impacts,
- Construction cost, and
- Benefit–cost analysis.

This methodology provides valuable information that can be used, in conjunction with other analyses, in making an appropriate choice of an interchange type and some of the primary design parameters. However, the final design will be based on many other criteria in addition to the output of this methodology.

Users are also cautioned that while the definition of interchange delay is similar for the interchange type selection methodology and the operational analysis methodology, different modeling approaches to delay prediction were taken, and there is no guarantee that the results of the two methodologies will be consistent.

4. O-D AND TURNING MOVEMENTS

O-D AND TURNING MOVEMENTS FOR INTERCHANGES WITH ROUNDABOUTS

Roundabouts are generally analyzed with the procedures of Chapter 22 of the HCM. This chapter provides guidance for translating O-D demands into movement demands at a roundabout to apply the procedures of Chapter 22.

Exhibit 34-160 defines the movements traveling through an interchange with two roundabouts, while Exhibit 34-161 lists the O-D demands contributing to each of these movements. For example, for diamond interchanges, O-D Movements G, H, and J constitute Movement 15 in Exhibit 34-160.

In analyzing interchanges with roundabouts, Exhibit 34-160 and Exhibit 34-161 should be used to establish the roundabout movements. The procedures of Chapter 22 should then be applied to estimate the capacity and delay for each roundabout approach. Finally, Exhibit 23-14 should be used to determine the LOS for each O-D demand through the interchange.

Exhibit 34-160
Illustration and Notation of O-D Demands at an Interchange with Roundabouts

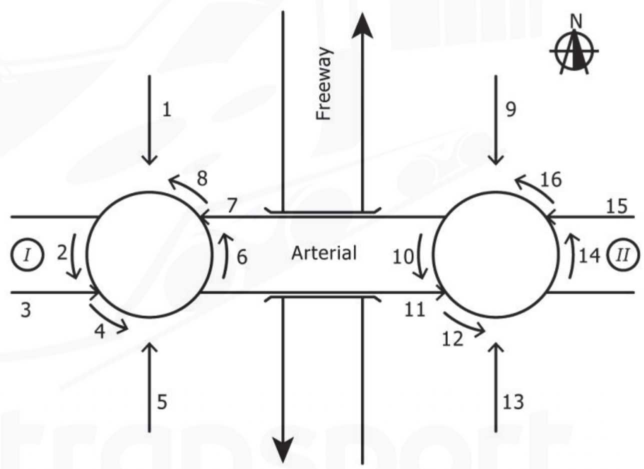


Exhibit 34-161
Notation of O-D Demands at Interchanges with Roundabouts

Movement	Diamond	Parclo A-2Q	Parclo B-2Q	Parclo B-4Q
1	C, D, L, N	C, D, N	--	C
2	D, H, L, M, N	D, N	H, M, N	H, M
3	E, F, I	E, F	E, F, I	E, F, I
4	D, E, F, H, I, L, M, N	D, E, F, I, N	E, F, H, I, M	E, F, H, I, M
5	--	--	C	D, N
6	--	F	C	--
7	A, H, J, M	A, H, J, M	A, H, J, M	A, H, J, M
8	J, M	A, F, H, J, M	A, C, H, J, M	A, H, J, M
9	--	--	A, B, M	A, M
10	--	G	B	-
11	D, E, I, N	D, E, I, N	D, E, I, N	D, E, I, N
12	D, E, I, N	D, E, G, I, N	B, D, E	D, E, I, N
13	A, B, K, M	A, B, M	--	B
14	A, E, K, M, N	A, M	E, N	E, N
15	G, H, J	G, H, J	G, H, J	G, H, J
16	A, E, G, H, J, K, M, N	A, G, H, J, M	E, G, H, J, N	E, G, H, J, N
Movement	**SPUI**	**Parclo AB-4Q**	**Parclo A-4Q**	**Parclo AB-2Q**
1	C, D, L, N	C	C, D, N	--
2	D, H, L, M, N	H, M	D, N	H, M
3	E, F, I	E, F, I	E, F, I	E, F, I
4	D, E, I, N	E, F, H, I, M	D, E, F, I, N	E, F, H, I, M
5	A, B, K, M	D, N	--	C, D, N
6	A, E, K, M, N	--	--	C
7	G, H, J	A, H, J, M	A, H, J, M	A, H, J, M
8	A, H, J, M	A, H, J, M	A, H, J, M	A, C, H, J, M
9	--	--	--	--
10	--	--	--	G
11	--	D, E, I, N	D, E, I, N	D, E, I, N
12	--	D, E, I, N	D, E, I, N	D, E, G, I, N
13	--	A, B, M	A, B, M	A, B, M
14	--	A, M	A, M	A, M
15	--	G, H, J	G, H, J	G, H, J
16	--	A, G, H, J, M	A, G, H, J, M	A, G, H, J, M

Note: -- indicates movements that do not exist for a given interchange form.

O-D AND TURNING MOVEMENTS FOR CONVENTIONAL INTERCHANGES

Exhibit 34-162 illustrates how O-D movements can be obtained from turning movements for each type of interchange considered in this methodology. Exhibit 34-163 through Exhibit 34-177 provide the corresponding calculations for obtaining turning movements from O-D movements.

Exhibit 34-162
O-D Flows for Each Interchange Configuration

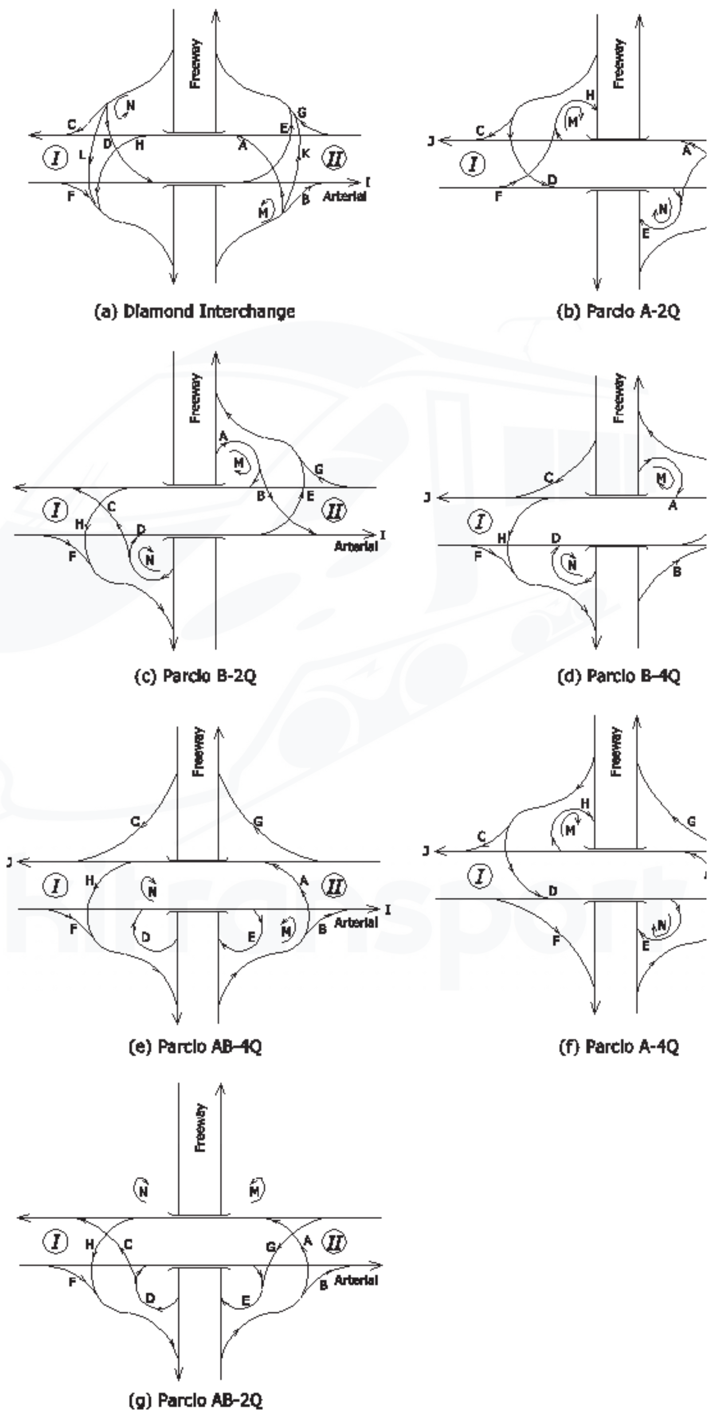


Exhibit 34-163
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo A-2Q Interchanges

Input					Output	
Approach	**Intersection I** Turning Movement	**Intersection I** Volume (veh/h)	**Intersection II** Turning Movement	**Intersection II** Volume (veh/h)	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	EXT-LT		LT		A = (NB LT) – (NB UT)	
	RT		INT-RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	LT		EXT-LT		D = (SB LT) – (SB UT)	
	INT-RT		RT		E = (EB INT-RT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-LT	
Northbound (NB)	LT		LT		G = WB EXT-LT	
	RT		RT		H = (WB INT-RT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (SB UT and NB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-164
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo A-4Q Interchanges

Input					Output	
Approach	**Intersection I** Turning Movement	**Intersection I** Volume (veh/h)	**Intersection II** Turning Movement	**Intersection II** Volume (veh/h)	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		LT		A = (NB LT) – (NB UT)	
	EXT-RT		INT-RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	LT		LT		D = (SB LT) – (SB UT)	
	INT-RT		EXT-RT		E = (EB INT-RT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT		G = WB EXT-RT	
	RT		RT		H = (WB INT-RT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (SB UT and NB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-165
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo AB-2Q Interchanges

Input					Output	
Approach	**Intersection I** Turning Movement	**Intersection I** Volume (veh/h)	**Intersection II** Turning Movement	**Intersection II** Volume (veh/h)	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		LT		A = (NB LT(II)) – (NB UT(II))	
	EXT-RT		INT-RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = NB LT(I)	
Westbound (WB)	INT-LT		EXT-LT		D = (NB RT(I)) – (NB UT(I))	
	RT		RT		E = (EB INT-RT) – (NB UT(I))	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT(I)		LT(II)		G = WB EXT-LT	
	RT(I)		RT(II)		H = (WB INT-LT) – (NB UT(II))	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT(I))	
	UT(I)		UT(II)		J = (WB INT-TH) – (NB LT(II)) + (NB UT(II))	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT(II)	
	UT		UT		N = NB UT(I)	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway [NB UT(I) and NB UT(II)] are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-166
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo AB-4Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		LT		A = (NB LT(II)) – (NB UT(II))	
	EXT-RT		INT-RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = SB RT(I)	
Westbound (WB)	INT-LT		LT		D = (NB RT(I)) – (NB UT(I))	
	RT		EXT-RT		E = (EB INT-RT) – (NB UT(I))	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT(II)		G = WB EXT-LT	
	RT(I)		RT(II)		H = (WB INT-LT) – (NB UT(II))	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT(I))	
	UT(I)		UT(II)		J = (WB INT-TH) – (NB LT(II)) + (NB UT(II))	
Southbound (SB)	LT		LT		K	
	RT(I)		RT		L	
	TH		TH		M = NB UT(II)	
	UT		UT		N = NB UT(I)	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway [NB UT(I) and NB UT(II)] are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-167
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo B-2Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (SB RT) – (SB UT)	
	EXT-RT		RT		B = SB LT	
	EXT-TH		INT-TH		C = NB LT	
Westbound (WB)	INT-LT		LT		D = (NB RT) – (NB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (NB UT)	
	INT-TH		EXT-TH		F = (EB EXT-RT)	
Northbound (NB)	L T		LT		G = (WB EXT-RT)	
	RT		RT		H = (WB INT-LT) – (SB UT)	
	TH		TH		I = (EB INT-TH) – (NB RT) + (NB UT)	
	UT		UT		J = (WB INT-TH) – (SB RT) + (SB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = SB UT	
	UT		UT		N = NB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-168
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo B-4Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (SB RT(II)) – (SB UT)	
	EXT-RT		RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = SB RT(I)	
Westbound (WB)	INT-LT		LT		D = (NB RT(I)) – (NB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (NB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	L T		LT		G = WB EXT-RT	
	RT(I)		RT(II)		H = (WB INT-LT) – (SB UT)	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT)	
	UT		UT		J = (WB INT-TH) – (SB RT(II)) + (SB UT)	
Southbound (SB)	LT		LT		K	
	RT(I)		RT(II)		L	
	TH		TH		M = SB UT	
	UT		UT		N = NB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-169
Worksheet for Obtaining O-D Movements from Turning Movements for Diamond Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (NB LT) – (NB UT)	
	EXT-RT		RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	INT-LT		LT		D = (SB LT) – (SB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	L T		LT		G = WB EXT-RT	
	RT		RT		H = (WB INT-LT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K = NB TH	
	RT		RT		L = SB TH	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-170
Worksheet for Obtaining O-D Movements from Turning Movements for SPUIs

Input			Output	
Approach	Turning Movement	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		A = NB LT	
	RT		B = NB RT	
	TH		C = SB RT	
Westbound (WB)	LT		D = SB LT	
	RT		E = EB LT	
	TH		F = EB RT	
Northbound (NB)	LT		G = WB RT	
	RT		H = WB LT	
	TH		I = EB TH	
	UT		J = WB TH	
Southbound (SB)	LT		K = NB TH	
	RT		L = SB TH	
	TH		M	
	UT		N	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through.
The flow of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-171
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo A-2Q and Parclo A-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	EXT-LT = F		LT	
B			RT		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	LT		EXT-LT = G	
E			INT-RT = H+M		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-172
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-2Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		EXT-LT = G	
E			RT		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT(I) = C		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-173
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-174
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-2Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT = C		LT	
H			RT = D+N		RT	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT = B	
L			RT		RT = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-175
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT(II) = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-176
Worksheet for Obtaining Turning Movements from O-D Movements for Diamond Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH = K	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH = L		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 34-177
Worksheet for Obtaining Turning Movements from O-D Movements for SPUIs

Input		Output		
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT = E	
B			RT = F	
C			TH = I	
D		Westbound (WB)	LT = H	
E			RT = G	
F			TH = J	
G		Northbound (NB)	LT = A	
H			RT = B	
I			TH = K	
J			UT	
K		Southbound (SB)	LT = D	
L			RT = C	
M			TH = L	
N			UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through.
Shading indicates movements that do not occur in this interchange form.

5. REFERENCES

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