This report presents a traffic impact analysis (TIA) for a proposed new 100,000 ft^2 shopping center development on the southeast corner of the study intersection of Huron Street and the cross street. The primary objectives are to forecast the traffic impacts of the new shopping center, evaluate the operation of the proposed access drive and study intersection after adding the site-generated traffic, and identify any necessary improvements to accommodate the additional volumes

The existing conditions at the study intersection were first evaluated based on turning movement counts. This established a baseline for traffic volumes, vehicle classifications, delays, and levels of service. Projected traffic volumes were then calculated accounting for a 2% annual growth rate over the 3-year horizon of the proposed development.

Using methodologies from the Institute of Transportation Engineers (ITE) Trip Generation Manual, forecasts were made for vehicle trips the 100,000 ft^2 shopping center is expected to generate during the PM peak hour. An assumption of a 50/50 split was applied to determine the entering/exiting volumes and their distribution among the turning movements at the proposed access drive location on Huron Street.

A detailed traffic impact analysis and design have been attached to demonstrate the improvement measures.

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Existing Conditions

Current Condition of Site

At present time, the site has residential buildings on the sides of each approach, it has crosswalks on each approach and Right turn on Red is not allowed on WB and EB directions, whereas, the NB and SB directions are only allowed thru movements.

	Lane width (ft)	Number of lanes	Parking Space	Bus Stop
North Bound	10	2	No	No
South Bound	11	2	No	No
East Bound	18	2	4	No
West Bound	12	2	3	No

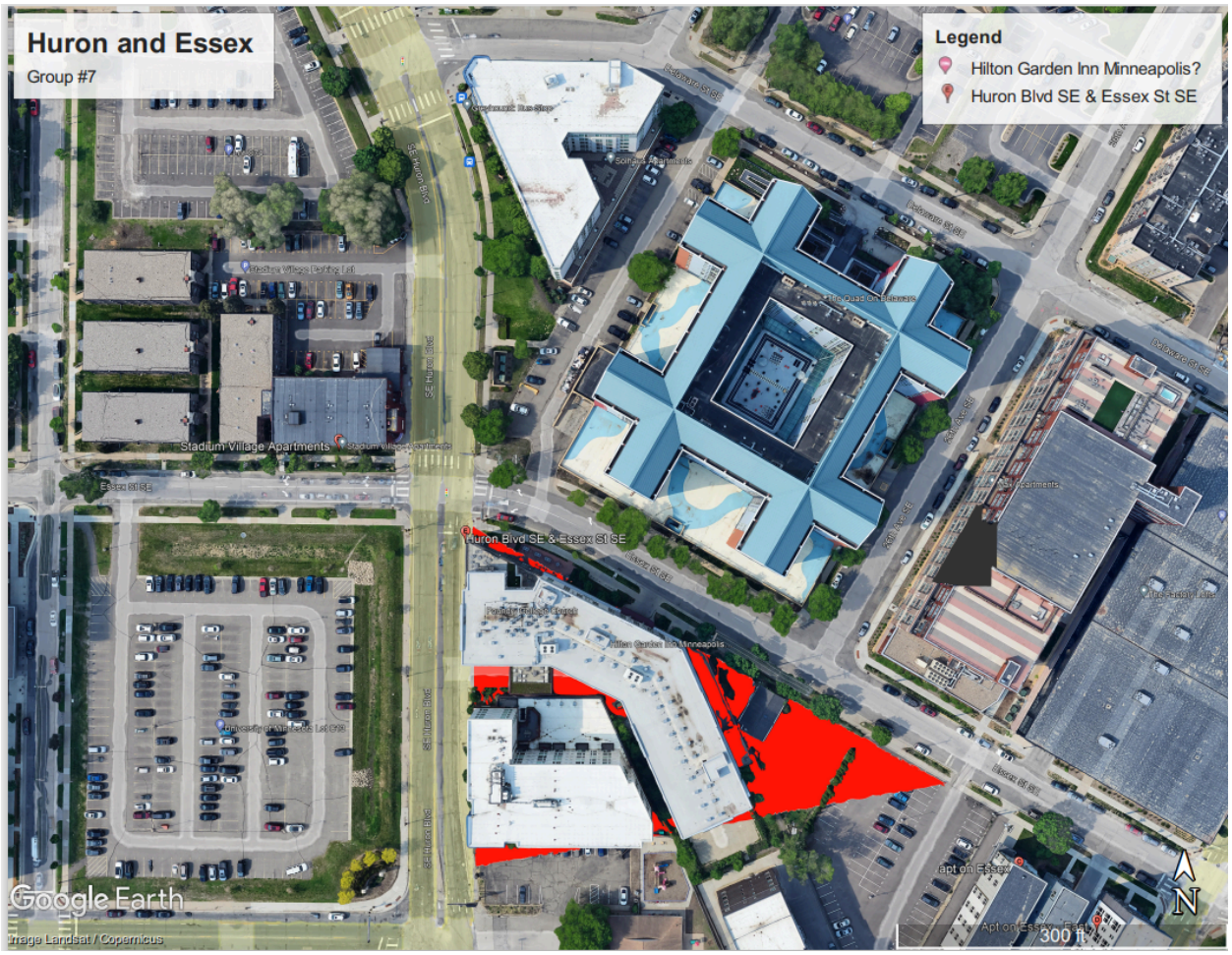


Figure: Aerial Map of Site

Traffic Volumes

The existing traffic volumes as counted in the PM peak hour is given below:

Approach Direction	RT	TH	LT
WB	14	0	232
EB	427	0	58
NB	0	611	0
SB	0	508	0

Complete data of the turning volumes are attached in Appendix A.

Vehicle Classification

According to the question, we are designing the driveway for a WB-50s vehicle.

Signalized Intersection Design

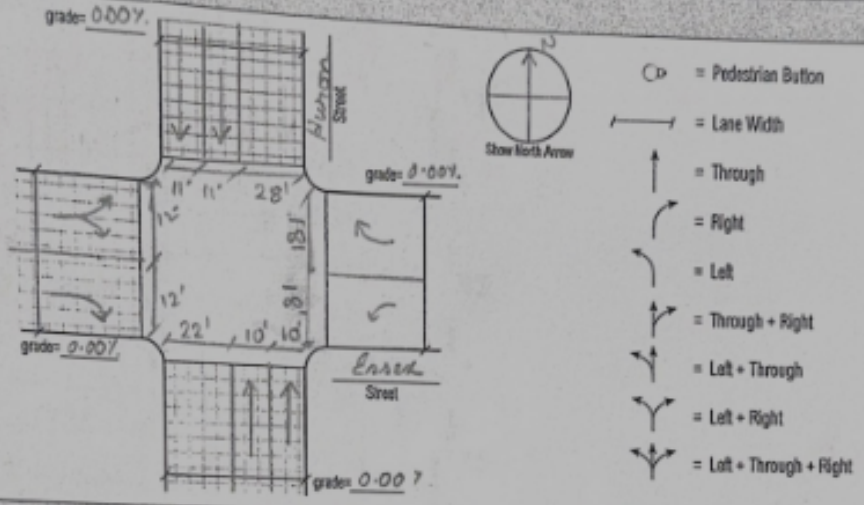
The existing intersection signal timing is as follows:

09:80 PM

Jurisdiction

Analysis Year

Intersection Geometry



Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]
Volume, V (veh/h)	52	–	442	221	–	19	–	603	–	–	501	–
% heavy vehicles, % HV	11.54	–	0.71	4.98	–	0	–	1.83	–	–	1.9	–
Peak-hour factor, PHF	0.91	–	0.95	0.88	–	0.50	–	0.98	–	–	0.97	–
Pretimed (P) or actuated (A)	P	–	P	P	–	P	–	P	–	–	P	–
Start-up lost time, l_1 (s)												
Extension of effective green time, e (s)												
Arrival type, AT	3	–	3	3	–	3	–	3	–	–	3	–
Approach pedestrian volume,[2] v_{ped} (p/h)	11			59			33			30		
Approach bicycle volume,[2] v_{bic} (bicycles/h)												
Parking (Y or N)	Y			Y			N			N		
Parking maneuvers, N_m (maneuvers/h)	8			7			–			–		
Bus stopping, N_B (buses/h)												
Min. timing for pedestrians,[3] G_p (s)	28			28			39			39		

Signal Phasing Plan

	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
DIAGRAM	↓ ↑	↘ ↙	↙					
Timing	G = 46 Y = 6 + 3	G = 23 Y = 2.5 + 5	G = 29 Y = 3.5 + 3	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =

Protected turns — Permitted turns — Pedestrian

Cycle length, C = 121 s

Notes

1. RT volumes, as shown, exclude RTOR.
2. Approach pedestrian and bicycle volumes are those that conflict with right turns from the subject approach.
3. Refer to Equation 16-2.

Intersection Delays

The total delays found in each approach are:

- For SB vehicles, the total delay is 960 veh-sec.
- For NB vehicles, the total delay is1635 veh-sec.
- For EB vehicles, the total delay is 1545 veh-sec.
- For WB vehicles, the total delay is 106 veh-sec.

The avg stopped delays per vehicle found in each approach are:

- For SB vehicles, the stopped delay is 36.92 seconds.
- For NB vehicles, the stopped delay is 39.88 seconds.
- For EB vehicles, the stopped delay is 12.77 seconds.
- For WB vehicles, the stopped delay is 4.82 seconds.

Average delay per approach vehicle

- SB 16.27 SEC
- EB 13.43 SEC
- NB 15.42 SEC
- WB 3.79 SEC

% VEHICLES STOPPED

- SB 44.1
- EB 80
- NB 38.7
- WB 78.6

From Synchro, the Approach LOS of EB, WB, NB,and SB are F,E,C and C respectively.

The control Delay for each turn are:

- 44.1s/veh for EB left turn
- 306.5s/veh for EB right turn
- 68 s/veh for WB left turn
- 37 s/veh for WB right turn
- 31.3s/veh for NB thru
- 30.3 s/veh for SB thru

The complete report from the Synchro simulation is added on Appendix C.

Proposed Development

Site Intersection:

The proposed development and the corresponding intersection are shown in the following drawing.

Projected Traffic

New development:

100,000 ft^2 shopping center,

1 site access, T-intersection abutting NB lane of Huron Street,

Access intersection may be stop-controlled from site, uncontrolled on Huron,

3 years until buildout,

2% annual growth rate,

Analysis for PM peak hour.

1. Turning movement volumes at access point T-intersection

Forecast of offsite traffic at buildout

From turning movement counts, the directional flows on Huron Street are currently

SB=437+232+508= 1177vph

NB=611 vph

2% annual growth rate for 3 years gives directional flows on Huron

SB=1177(1.02)(1.02)(1.02)= 1249 vph

NB=611(1.02)(1.02)(1.02)= 648 vph

2. Forecast of site traffic

Using ITE=s Trip Generation Manual, total PM peak hour traffic for

100,000 ft^2: ln(T)=(.66)1n(100)+3.4

T==626 vph

With 50/50 directional split:

Shopping Center Entering Flow=(.5)626=313 vph

Shopping Center Exiting Flow=(.5)626=313 vph

3. Assigning site traffic to turning movements

Assumptions:

- 0% Pass-by trips,
- Site traffic has same origin-destination characteristics as nonsite traffic proportions on Huron street

SB=1249/(1249+648)=0.66

NB=1-0.66=0.34

Thus, Entering Trips

Entering from SB direction via LT=(.66)(313)=207 vph

Entering from NB direction via RT=(.34)(313)=106 vph

Exiting Trips

Exiting to SB direction via LT=(.66)(313)=207 vph

Exiting to NB direction via RT=(.34)(313)=106 vph

Forecasted Turning Movement Volumes at Access Intersection:

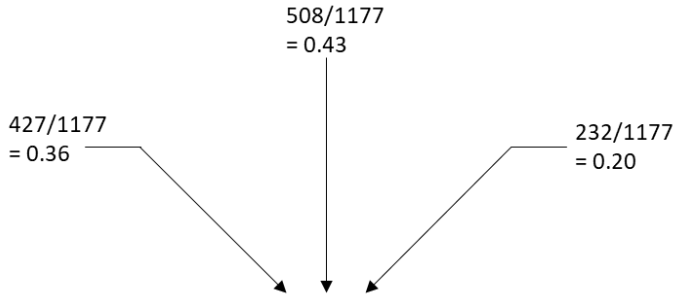
Movement			
Approach Direction	RT	TH	LT
NB	648	106	0
SB	0	1249	207
WB	207	0	106

Forecast of nonsite traffic by applying growth factor to turning movement counts

Approach Direction	RT	TH	LT
WB	14*(1.02)^3=15	0	232*(1.02)^3=246
EB	453	0	62
NB	0	648	0
SB	0	539	0

Distribution of site traffic using existing proportions: Origin

proportions of SB traffic are:



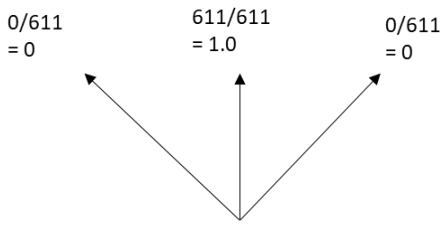
So for the 207 vph entering site from SB on Huron

(207)(.36)=75 vph will be EB RT

(207)(.43)=89 vph will be SB TH

(207)(.20)=41 vph will be WB LT.

Destination proportions for NB traffic are:



So for 106 vph entering site

106vph will be NB THs

Turning Movement Volumes at the driveway intersection:

Approach Direction	RT	TH	LT
WB	106	0	207
EB	0	0	0
NB	106	648	0
SB	0	539	207

Turning Movement Volumes at the signalized intersection:

Approach Direction	RT	TH	LT
WB	15	0	246+41=287
EB	453+75 = 528	0	62
NB	0	648+106=754	0
SB	0	539+89=628	0

Traffic Analysis at Build-out

Driveway Design

Pavement width determination

AASHTO Exhibit 9-10, p. 584

MnDOT WB-50s, with $\Delta=90^\circ$ R = 60 ft O = 4.0 ft Taper = 15:1

Taper angle: $\Theta=\tan-1(1/\text{Taper}) = \tan-1(1/15)=3.81^\circ$

Distance PI to start taper: $T=(R+O)\tan(\Delta/2)+\text{taper}[O+R(1-\cos\theta]-R\sin\theta$

$=(60+4)\tan(45)+15[4+60(1-\cos(3.81)]- 60\sin(3.81) = 122$ ft

Distance start taper to start curve:

$\text{Taper}[O+R(1-\cos\theta)] = (15)[4+60(1-\cos(3.81)] = 61.99 \sim 62$ ft

Offset at start curve: $O+R(1-\cos\theta) = 4 + 60[1-\cos(3.81)] = 4.13$ ft

Pavement width: W = 30 ft (AASHTO Exhibit 3-50)

The driveway is designed to accommodate a WB-50 semi-truck. Using the geometric design criteria from AASHTO Exhibit 9-10 for a 90-degree intersection with a 60-foot radius and 15:1 taper rate. The total pavement width required is 30 feet on both side per AASHTO Exhibit 3-50 for a two-lane, two-way driveway. This design allows the WB-50 trucks to enter and exit the driveway without encroaching on the opposing traffic lane or crossing over the edge lines.

Sight triangles for driveway

For intersection

Passenger car to turn left:

Required SD: $d_{ISD} = 1.47* V_{major} *t_g$

= 1.47*20*7.5 [assuming, passenger car for t_g]

= 220.5 ft

Available SD: $d_a = \frac{bda}{db-a} < 0$

b= 69 ft

a= 59ft

d_b= 23.5 ft

6.5 ft – AASHTO typical distance from PI to vehicle front end

8 ft = conservative distance from front end to driver eye

a> d_b so no sight obstruction.

For Driveway

Combo truck to turn left:

Required SD: $d_{ISD} = 1.47 * V_{major} * t_g$

= 1.47*20*11.5 [assuming, combo truck for t_g]

= 220.5 ft

Available SD: $d_a = \frac{bda}{db-a} < 0$

b= 78 ft

a= 81 ft

d_b= 32.5 ft

6.5 ft – AASHTO typical distance from PI to vehicle front end

8 ft = conservative distance from front end to driver eye

a> d_b so no sight obstruction.

Sight Distance Analysis: The sight distances required for turning maneuvers from the driveway onto Huron Street were calculated using the AASHTO formula based on the major road (Huron St) design speed of 20 mph.

For passenger cars making a left turn from the driveway:

- Required intersection sight distance is 220.5 feet
- Available sight distance was determined to be adequate in both directions with no obstructions

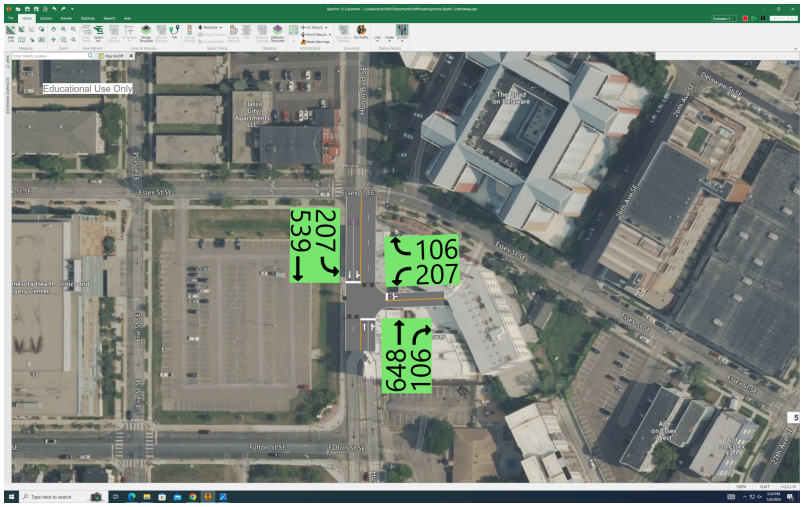
For WB-50 trucks making a left turn from the driveway:

- Required intersection sight distance is 335.5 feet
- Available sight distance was determined to be adequate in both directions with no obstructions

The sight distance analysis confirms that approaching vehicles on Huron Street will have a clear view of vehicles turning out of the driveway, meeting AASHTO safety standards. Likewise, vehicles exiting the driveway will have proper sight lines to identify gaps in traffic on Huron to complete their turning maneuvers.

LOS of Driveway and Intersection

Driveway Volume is as given below:



For the scenario of having stop controlled traffic in Driveway, the LOS of the Driveway is C.

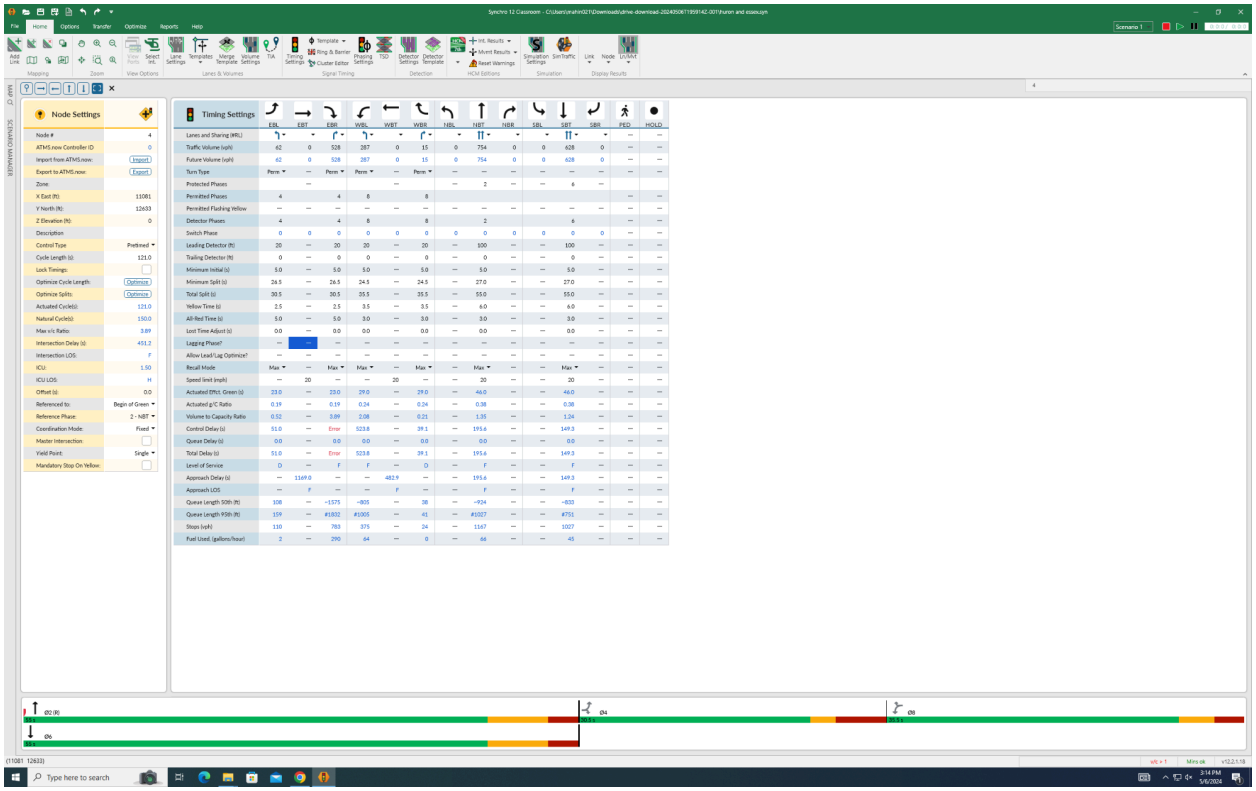
Lanes, Volumes, Timings
6: 05/06/2024

Lane Group	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (vph)	207	106	648	106	207	539
Future Volume (vph)	207	106	648	106	207	539
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Lane Util. Factor	1.00	1.00	0.95	0.95	0.95	0.95
Frt	0.954		0.979			
Flt Protected	0.968					0.986
Satd. Flow (prot)	1720	0	3465	0	0	3490
Flt Permitted	0.968					0.986
Satd. Flow (perm)	1720	0	3465	0	0	3490
Link Speed (mph)	30		20			30
Link Distance (ft)	132		70			173
Travel Time (s)	3.0		2.4			3.9
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	225	115	704	115	225	586
Shared Lane Traffic (%)						
Lane Group Flow (vph)	340	0	819	0	0	811
Enter Blocked Intersection	No	No	No	No	No	No
Lane Alignment	Left	Right	Left	Right	Left	Left
Median Width(ft)	12		0			0
Link Offset(ft)	0		0			0
Crosswalk Width(ft)	16		16			16
Two way Left Turn Lane						
Headway Factor	1.00	1.00	1.00	1.00	1.00	1.00
Turning Speed (mph)	15	9		9	15	
Sign Control	Stop		Stop			Stop

Intersection Summary			
Area Type:	Other		
Control Type: Unsignalized			
Intersection Capacity Utilization	70.2%	ICU Level of Service	C
Analysis Period (min)	15		

Whereas, projected nonsite traffic LOS with current signal timing:

The LOS of each approach with projected traffic and current signal phasing and timing has worse level of service, LOS F on each approach.



Possible Improvements

One improvement could be to optimize the signal timing for the forecasted traffic volumes at the nonsite intersection. Synchro has been used to optimize the phasing and the timing:

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	62	0	528	287	0	15	0	754	0	0	628	0
Future Volume (vph)	62	0	528	287	0	15	0	754	0	0	628	0
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width (ft)	16	12	16	12	12	12	12	10	12	12	11	12
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	1.00	1.00	0.95	1.00
Ped Bike Factor	0.97		0.96	0.85		0.84						
Frt			0.850			0.850						
Flt Protected	0.950			0.950								
Satd. Flow (prot)	1599	0	1559	1487	0	1397	0	3336	0	0	3455	0
Flt Permitted	0.950			0.950								
Satd. Flow (perm)	1555	0	1498	1268	0	1178	0	3336	0	0	3455	0
Right Turn on Red			No			No			No			No
Satd. Flow (RTOR)												
Link Speed (mph)		20			20			20			20	
Link Distance (ft)		346			365			243			363	
Travel Time (s)		11.8			12.4			8.3			12.4	
Confl. Peds. (#/hr)	11		11	59		59	33		33	30		30
Peak Hour Factor	0.81	0.25	0.95	0.88	0.25	0.50	0.25	0.88	0.25	0.25	0.77	0.25
Growth Factor	200%	200%	200%	200%	200%	200%	200%	200%	200%	200%	200%	200%
Heavy Vehicles (%)	10%	0%	1%	5%	0%	0%	0%	1%	0%	0%	1%	0%
Parking (#/hr)	8		8	7		7						
Adj. Flow (vph)	153	0	1112	652	0	60	0	1714	0	0	1631	0
Shared Lane Traffic (%)												
Lane Group Flow (vph)	153	0	1112	652	0	60	0	1714	0	0	1631	0
Enter Blocked Intersection	No	No	No	No	No	No	No	No	No	No	No	No
Lane Alignment	Left	Left	Right	Left	Left	Right	Left	Left	Right	Left	Left	Right
Median Width(ft)		16			16			0			0	
Link Offset(ft)		0			0			0			0	
Crosswalk Width(ft)		16			16			16			16	
Two way Left Turn Lane												
Headway Factor	1.03	1.00	1.03	1.20	1.00	1.20	1.00	1.09	1.00	1.00	1.04	1.00
Turning Speed (mph)	15		9	15		9	15		9	15		9
Turn Type	Perm		Perm	Perm		Perm		NA			NA	
Protected Phases								2			6	
Permitted Phases	4		4	8		8						
Minimum Split (s)	26.5		26.5	24.5		24.5		27.0			27.0	
Total Split (s)	58.0		58.0	44.0		44.0		48.0			48.0	
Total Split (%)	38.7%		38.7%	29.3%		29.3%		32.0%			32.0%	
Maximum Green (s)	50.5		50.5	37.5		37.5		39.0			39.0	
Yellow Time (s)	2.5		2.5	3.5		3.5		6.0			6.0	
All-Red Time (s)	5.0		5.0	3.0		3.0		3.0			3.0	
Lost Time Adjust (s)	0.0		0.0	0.0		0.0		0.0			0.0	
Total Lost Time (s)	7.5		7.5	6.5		6.5		9.0			9.0	
Lead/Lag												
Lead-Lag Optimize?												
Walk Time (s)	7.0		7.0	7.0		7.0		7.0			7.0	
Flash Don't Walk (s)	11.0		11.0	11.0		11.0		11.0			11.0	
Pedestrian Calls (#/hr)	0		0	0		0		0			0	

Lanes, Volumes, Timings
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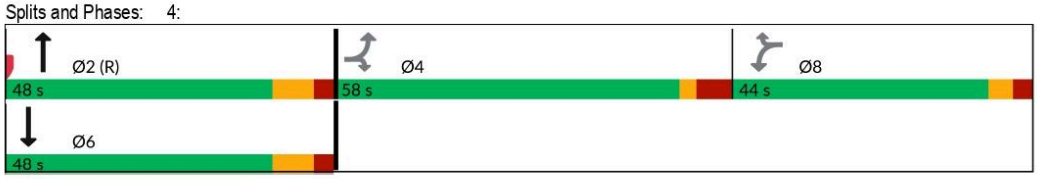
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Act Effct Green (s)	50.5		50.5	37.5		37.5		39.0			39.0	
Actuated g/C Ratio	0.34		0.34	0.25		0.25		0.26			0.26	
v/c Ratio	0.29		2.20	2.05		0.20		1.97			1.81	
Control Delay (s/veh)	38.5		574.3	514.6		46.7		472.9			403.3	
Queue Delay	0.0		0.0	0.0		0.0		0.0			0.0	
Total Delay (s/veh)	38.5		574.3	514.6		46.7		472.9			403.3	
LOS	D		F	F		D		F			F	
Approach Delay (s/veh)		509.6			475.2			473.0			403.4	
Approach LOS		F			F			F			F	
Queue Length 50th (ft)	110		~1744	~1001		47		~1366			~1261	
Queue Length 95th (ft)	153		#2010	#1210		48		#1461			#1128	
Internal Link Dist (ft)		266			285			163			283	
Turn Bay Length (ft)												
Base Capacity (vph)	523		504	317		294		867			898	
Starvation Cap Reductn	0		0	0		0		0			0	
Spillback Cap Reductn	0		0	0		0		0			0	
Storage Cap Reductn	0		0	0		0		0			0	
Reduced v/c Ratio	0.29		2.21	2.06		0.20		1.98			1.82	

Intersection Summary

Area Type: Other
Cycle Length: 150
Actuated Cycle Length: 150
Offset: 0 (0%), Referenced to phase 2:NBT, Start of Green
Natural Cycle: 150
Control Type: Pretimed
Maximum v/c Ratio: 2.21
Intersection Signal Delay (s/veh): 460.6 Intersection LOS: F
Intersection Capacity Utilization 150.1% ICU Level of Service H
Analysis Period (min) 15
~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 4:



However, it did not lead to any improvement in the LOS of the intersection.

Signalizing the Driveway

Another possible improvement could be to signalize the driveway intersection.

Lanes, Volumes, Timings
6: 05/06/2024

Lane Group	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (vph)	207	106	648	106	207	539
Future Volume (vph)	207	106	648	106	207	539
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Lane Util. Factor	1.00	1.00	0.95	0.95	0.95	0.95
Frt	0.954		0.979			
Flt Protected	0.968					0.986
Satd. Flow (prot)	1720	0	3465	0	0	3490
Flt Permitted	0.968					0.616
Satd. Flow (perm)	1720	0	3465	0	0	2180
Right Turn on Red		Yes		Yes		
Satd. Flow (RTOR)	26		20			
Link Speed (mph)	30		20			30
Link Distance (ft)	132		70			173
Travel Time (s)	3.0		2.4			3.9
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	225	115	704	115	225	586
Shared Lane Traffic (%)						
Lane Group Flow (vph)	340	0	819	0	0	811
Enter Blocked Intersection	No	No	No	No	No	No
Lane Alignment	Left	Right	Left	Right	Left	Left
Median Width(ft)	12		0			0
Link Offset(ft)	0		0			0
Crosswalk Width(ft)	16		16			16
Two way Left Turn Lane						
Headway Factor	1.00	1.00	1.00	1.00	1.00	1.00
Turning Speed (mph)	15	9		9	15	
Turn Type	Prot		NA		Perm	NA
Protected Phases	8		2			6
Permitted Phases					6	
Minimum Split (s)	22.5		22.5		22.5	22.5
Total Split (s)	22.5		28.0		39.5	39.5
Total Split (%)	25.0%		31.1%		43.9%	43.9%
Maximum Green (s)	18.0		23.5		35.0	35.0
Yellow Time (s)	3.5		3.5		3.5	3.5
All-Red Time (s)	1.0		1.0		1.0	1.0
Lost Time Adjust (s)	0.0		0.0			0.0
Total Lost Time (s)	4.5		4.5			4.5
Lead/Lag						
Lead-Lag Optimize?						
Walk Time (s)	7.0		7.0		7.0	7.0
Flash Dont Walk (s)	11.0		11.0		11.0	11.0
Pedestrian Calls (#/hr)	0		0		0	0
Act Effct Green (s)	18.0		23.5			35.0
Actuated g/C Ratio	0.20		0.26			0.39
v/c Ratio	0.93		0.89			0.95
Control Delay (s/veh)	67.7		44.8			50.4
Queue Delay	0.0		0.0			15.3
Total Delay (s/veh)	67.7		44.8			65.8

Lanes, Volumes, Timings
6: 05/06/2024

Lane Group	WBL	WBR	NBT	NBR	SBL	SBT
LOS	E		D			F
Approach Delay (s/veh)	67.8		44.8			95.8
Approach LOS	E		D			F
Queue Length 50th (ft)	178		231			231
Queue Length 95th (ft)	#345		#339			#360
Internal Link Dist (ft)	52		1			93
Turn Bay Length (ft)						
Base Capacity (vph)	364		919			847
Starvation Cap Reductn	0		0			359
Spillback Cap Reductn	0		0			0
Storage Cap Reductn	0		0			0
Reduced v/c Ratio	0.93		0.89			1.66

Intersection Summary			
Area Type:	Other		
Cycle Length: 90			
Actuated Cycle Length: 90			
Offset: 0 (0%), Referenced to phase 2:NBT, Start of Green			
Natural Cycle: 90			
Control Type: Pretimed			
Maximum v/c Ratio: 0.96			
Intersection Signal Delay (s/veh): 69.8		Intersection LOS: E	
Intersection Capacity Utilization 71.4%		ICU Level of Service C	
Analysis Period (min) 15			
# 95th percentile volume exceeds capacity, queue may be longer.			
Queue shown is maximum after two cycles.			

Splits and Phases: 6:

The signalization of the driveway intersection did not improve the LOS, however with a slightly longer queue.

Combined Nonsite and Driveway Intersection Optimization

To further improve, both intersection signal phasing and timing has been optimized simultaneously using synchro (including offset).

The complete result of the optimization has been attached on Appendix D.

This optimization lead to improvement of LOS from F to E on NB approach, but no other improvements elsewhere in the nonsite intersection.

Lanes, Volumes, Timings
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Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Actuated g/C Ratio	0.32		0.32	0.22		0.22		0.24			0.24	
v/c Ratio	0.15		1.16	1.13		0.11		1.07			0.98	
Control Delay (s/veh)	25.8		129.1	131.1		33.0		62.4			66.4	
Queue Delay	0.0		0.5	0.0		0.0		11.7			66.5	
Total Delay (s/veh)	25.8		129.7	131.1		33.0		74.2			133.0	
LOS	C		F	F		C		E			F	
Approach Delay (s/veh)		117.1			122.9			74.2			133.0	
Approach LOS		F			F			E			F	
Queue Length 50th (ft)	35		~424	~243		16		~100			273	
Queue Length 95th (ft)	63		#630	#400		22		m#388			#293	
Internal Link Dist (ft)		266			285			93			283	
Turn Bay Length (ft)												
Base Capacity (vph)	494		476	288		267		800			829	
Starvation Cap Reductn	0		0	0		0		54			0	
Spillback Cap Reductn	0		27	0		0		0			735	
Storage Cap Reductn	0		0	0		0		0			0	
Reduced v/c Ratio	0.16		1.24	1.13		0.11		1.15			8.68	

Intersection Summary			
Area Type:	Other		
Cycle Length: 100			
Actuated Cycle Length: 100			
Offset: 0 (0%), Referenced to phase 2:NBT, Start of Green			
Natural Cycle: 140			
Control Type: Pretimed			
Maximum v/c Ratio: 1.17			
Intersection Signal Delay (s/veh): 108.9		Intersection LOS: F	
Intersection Capacity Utilization 84.2%		ICU Level of Service E	
Analysis Period (min) 15			
~ Volume exceeds capacity, queue is theoretically infinite.			
Queue shown is maximum after two cycles.			
# 95th percentile volume exceeds capacity, queue may be longer.			
Queue shown is maximum after two cycles.			
m Volume for 95th percentile queue is metered by upstream signal.			

Splits and Phases: 4:

In the driveway intersection, this optimization shows improvement of LOS on one approach while worsening on another, leading to an intersection LOS of C. Thus, not resulting in any overall improvement.

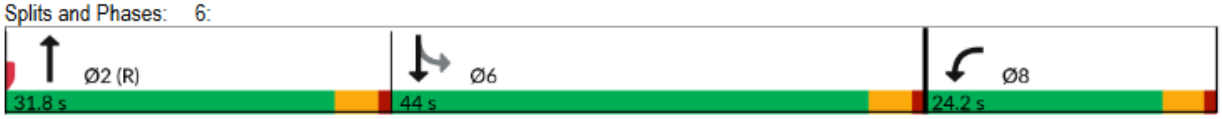
Lanes, Volumes, Timings
6: 05/06/2024

Lane Group	WBL	WBR	NBT	NBR	SBL	SBT
LOS	F		D			E
Approach Delay (s/veh)	121.3		54.8			78.1
Approach LOS	F		D			E
Queue Length 50th (ft)	203		255			255
Queue Length 95th (ft)	#379		#354			m231
Internal Link Dist (ft)	52		1			93
Turn Bay Length (ft)						
Base Capacity (vph)	357		959			857
Starvation Cap Reductn	0		0			430
Spillback Cap Reductn	116		128			0
Storage Cap Reductn	0		0			0
Reduced v/c Ratio	1.41		0.99			1.90

Intersection Summary			
Area Type:	Other		
Cycle Length: 100			
Actuated Cycle Length: 100			
Offset: 97 (97%), Referenced to phase 2:NBT, Start of Green			
Natural Cycle: 90			
Control Type: Pretimed			
Maximum v/c Ratio: 0.95			
Intersection Signal Delay (s/veh): 75.8		Intersection LOS: E	
Intersection Capacity Utilization 71.4%		ICU Level of Service C	
Analysis Period (min) 15			

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.
m Volume for 95th percentile queue is metered by upstream signal.

Splits and Phases: 6:



Driveway Intersection Unsignalized, Nonsite Intersection Signalized:

Further optimization has been performed with keeping the nonsite intersection signalized and the driveway intersection operated as stop controlled. Appendix E features the complete report from Synchro.

This has led to similar results with SB approach in nonsite intersection showing LOS E and other approaches has an LOS F, while the LOS of the unsignalized driveway has an LOS of C. Thus not resulting in any significant difference from having the driveway signalized.

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Conclusion and Recommendation

Optimization of the the nonsite intersection and the site driveway intersection has led to the conclusion that the nonsite intersection should have a signal timing of:

			
G	33	39	28
Y	2.5	3.5	6
AR	5	3	3

Additionally, keeping the driveway intersection unsignalized.

Recommendation

The driveway entry and exit on Huron Street is causing additional queue length increase on an already congested road. Moving the Entry/Exit of the site to the Essex Street (Minor Street) that has less traffic volume may exert less pressure on the overall traffic network.

Appendix

Appendix A

Turning Movement Volume Datasheet

Passenger car movements-15 minutes interval																	
Start Time	Southbound				Westbound				Northbound				Eastbound				Total
	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	
Movement	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
3:00	4	0	115	0	14	4	0	48	8	0	138	0	1	103	0	16	435
3:15	11	0	120	0	9	7	0	63	11	0	137	0	3	98	0	10	459
3:30	4	0	104	0	19	2	0	62	10	0	171	0	3	111	0	11	486
3:45	11	0	162	0	17	1	0	48	4	0	157	0	4	112	0	15	516
Hour Total	30	0	501	0	59	14	0	221	33	0	603	0	11	424	0	52	1896

Trucks and buses movements-15 minutes interval																	
Start Time	Southbound				Westbound				Northbound				Eastbound				Total
	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	
Movement	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
3:00	4	0	1	0	14	0	0	1	8	0	1	0	1	1	0	1	31
3:15	11	0	3	0	9	0	0	1	11	0	2	0	3	0	0	3	40
3:30	4	0	3	0	19	0	0	2	10	0	4	0	3	0	0	0	45
3:45	11	0	0	0	17	0	0	7	4	0	1	0	4	2	0	2	46
Hour Total	30	0	7	0	59	0	0	11	33	0	8	0	11	3	0	6	162

Appendix B

Peak Hour Analysis - Passenger Cars																	
	Southbound				Westbound				Northbound				Eastbound				Total
	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	
1. Peak Hour of Intersection	30	0	501	0	59	14	0	221	33	0	603	0	11	424	0	52	1896
Peak Hour Factor (Method 1)	**0.68**		**0.77**		**0.78**	**0.50**		**0.88**	**0.75**		**0.88**		**0.69**	**0.95**		**0.81**	**0.92**
2. Peak 15 min x 4	44	0	648	0	76	28	0	252	44	0	684	0	16	448	0	64	2064
Peak Hour Factor (Method 2)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Peak Hour Analysis - Trucks and Buses																	
	Southbound				Westbound				Northbound				Eastbound				Total
	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	Peds	Right	Thru	Left	
1. Peak Hour of Intersection	30	0	7	0	59	0	0	11	33	0	8	0	11	3	0	6	162
Peak Hour Factor (Method 1)	**0.68**		**0.58**		**0.78**			**0.39**	**0.75**		**0.50**		**0.69**	**0.38**		**0.50**	**0.88**
2. Peak 15 min x 4	44	0	12	0	76	0	0	28	44	0	16	0	16	8	0	12	184
Peak Hour Factor (Method 2)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Appendix C

Lanes, Volumes, Timings
4:

05/06/2024

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	58	0	427	232	0	14	0	611	0	0	508	0
Future Volume (vph)	58	0	427	232	0	14	0	611	0	0	508	0
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width (ft)	16	12	16	12	12	12	12	10	12	12	11	12
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	1.00	1.00	0.95	1.00
Ped Bike Factor	0.97		0.96	0.88		0.87						
Frt			0.850			0.850						
Flt Protected	0.950			0.950								
Satd. Flow (prot)	1599	0	1559	1487	0	1397	0	3336	0	0	3455	0
Flt Permitted	0.950			0.950								
Satd. Flow (perm)	1564	0	1506	1310	0	1217	0	3336	0	0	3455	0
Right Turn on Red			No			No			No			No
Satd. Flow (RTOR)												
Link Speed (mph)		20			20			20			20	
Link Distance (ft)		346			365			173			363	
Travel Time (s)		11.8			12.4			5.9			12.4	
Confl. Peds. (#/hr)	11		11	59		59	33		33	30		30
Peak Hour Factor	0.81	0.25	0.95	0.88	0.25	0.50	0.25	0.88	0.25	0.25	0.77	0.25
Heavy Vehicles (%)	10%	0%	1%	5%	0%	0%	0%	1%	0%	0%	1%	0%
Parking (#/hr)	8		8	7		7						
Adj. Flow (vph)	72	0	449	264	0	28	0	694	0	0	660	0
Shared Lane Traffic (%)												
Lane Group Flow (vph)	72	0	449	264	0	28	0	694	0	0	660	0
Enter Blocked Intersection	No	No	No	No	No	No	No	No	No	No	No	No
Lane Alignment	Left	Left	Right	Left	Left	Right	Left	Left	Right	Left	Left	Right
Median Width(ft)		16			16			0			0	
Link Offset(ft)		0			0			0			0	
Crosswalk Width(ft)		16			16			16			16	
Two way Left Turn Lane												
Headway Factor	1.03	1.00	1.03	1.20	1.00	1.20	1.00	1.09	1.00	1.00	1.04	1.00
Turning Speed (mph)	15		9	15		9	15		9	15		9
Turn Type	Perm		Perm	Perm		Perm		NA			NA	
Protected Phases								2			6	
Permitted Phases	4		4	8		8						
Minimum Split (s)	26.5		26.5	24.5		24.5		27.0			27.0	
Total Split (s)	30.5		30.5	35.5		35.5		55.0			55.0	
Total Split (%)	25.2%		25.2%	29.3%		29.3%		45.5%			45.5%	
Maximum Green (s)	23.0		23.0	29.0		29.0		46.0			46.0	
Yellow Time (s)	2.5		2.5	3.5		3.5		6.0			6.0	
All-Red Time (s)	5.0		5.0	3.0		3.0		3.0			3.0	
Lost Time Adjust (s)	0.0		0.0	0.0		0.0		0.0			0.0	
Total Lost Time (s)	7.5		7.5	6.5		6.5		9.0			9.0	
Lead/Lag												
Lead-Lag Optimize?												
Walk Time (s)	7.0		7.0	7.0		7.0		7.0			7.0	
Flash Dont Walk (s)	11.0		11.0	11.0		11.0		11.0			11.0	
Pedestrian Calls (#/hr)	0		0	0		0		0			0	
Act Effct Green (s)	23.0		23.0	29.0		29.0		46.0			46.0	

Lanes, Volumes, Timings
4: 05/06/2024

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Actuated g/C Ratio	0.19		0.19	0.24		0.24		0.38			0.38	
v/c Ratio	0.24		1.56	0.84		0.09		0.54			0.50	
Control Delay (s/veh)	44.1		306.5	68.0		37.0		31.3			30.3	
Queue Delay	0.0		0.0	0.0		0.0		0.0			0.0	
Total Delay (s/veh)	44.1		306.5	68.0		37.0		31.3			30.3	
LOS	D		F	E		D		C			C	
Approach Delay (s/veh)		270.3			65.1			31.4			30.4	
Approach LOS		F			E			C			C	
Queue Length 50th (ft)	48		~497	198		17		221			205	
Queue Length 95th (ft)	83		#702	#332		23		273			215	
Internal Link Dist (ft)		266			285			93			283	
Turn Bay Length (ft)												
Base Capacity (vph)	297		286	313		291		1268			1313	
Starvation Cap Reductn	0		0	0		0		0			0	
Spillback Cap Reductn	0		0	0		0		0			0	
Storage Cap Reductn	0		0	0		0		0			0	
Reduced v/c Ratio	0.24		1.57	0.84		0.10		0.55			0.50	

Intersection Summary			
Area Type:	Other		
Cycle Length: 121			
Actuated Cycle Length: 121			
Offset: 6 (5%), Referenced to phase 2:NBT, Start of Green			
Natural Cycle: 90			
Control Type: Pretimed			
Maximum v/c Ratio: 1.57			
Intersection Signal Delay (s/veh): 93.0		Intersection LOS: F	
Intersection Capacity Utilization 73.0%		ICU Level of Service C	
Analysis Period (min) 15			
~ Volume exceeds capacity, queue is theoretically infinite.			
Queue shown is maximum after two cycles.			
# 95th percentile volume exceeds capacity, queue may be longer.			
Queue shown is maximum after two cycles.			

Splits and Phases: 4: