APPENDIX A

TRAFFIC OPERATIONS STUDY

Highway No. 21
Connecting Link Analysis
Village of Grand Bend
Municipality of Lambton Shores

Final Report
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Submitted by

Dillon Consulting Limited

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

1.0 BACKGROUND

The Village of Grand Bend is one of five main communities within the Municipality of Lambton Shores, Ontario. Grand Bend is situated along Highway 21 approximately 46 kilometres north of Highway 402 (stretching between Sarnia and London). Located on the shore of Lake Huron, the village is a heavily tourist-oriented locale, with popular beaches and attractions within the town, cottage areas to the north and south along Lake Huron, and the Pinery Provincial Park to the south. The main business and tourist district is still situated downtown, in the vicinity of the intersection of Highway 21 and former Highway 81, although new commercial development is planned for Main Street east of the village limits. In the summer period, Friday evenings and Saturdays are typically the times of greatest vehicular activity. Within the main urban section of the Village of Grand Bend, Highway 21 (also known as Ontario Street) has a non-standard four-lane cross-section, consisting of two southbound lanes, one northbound lane, and one two-way continuous left-turn lane; to the north and south of the village, Highway 21 is a standard rural two-lane highway.

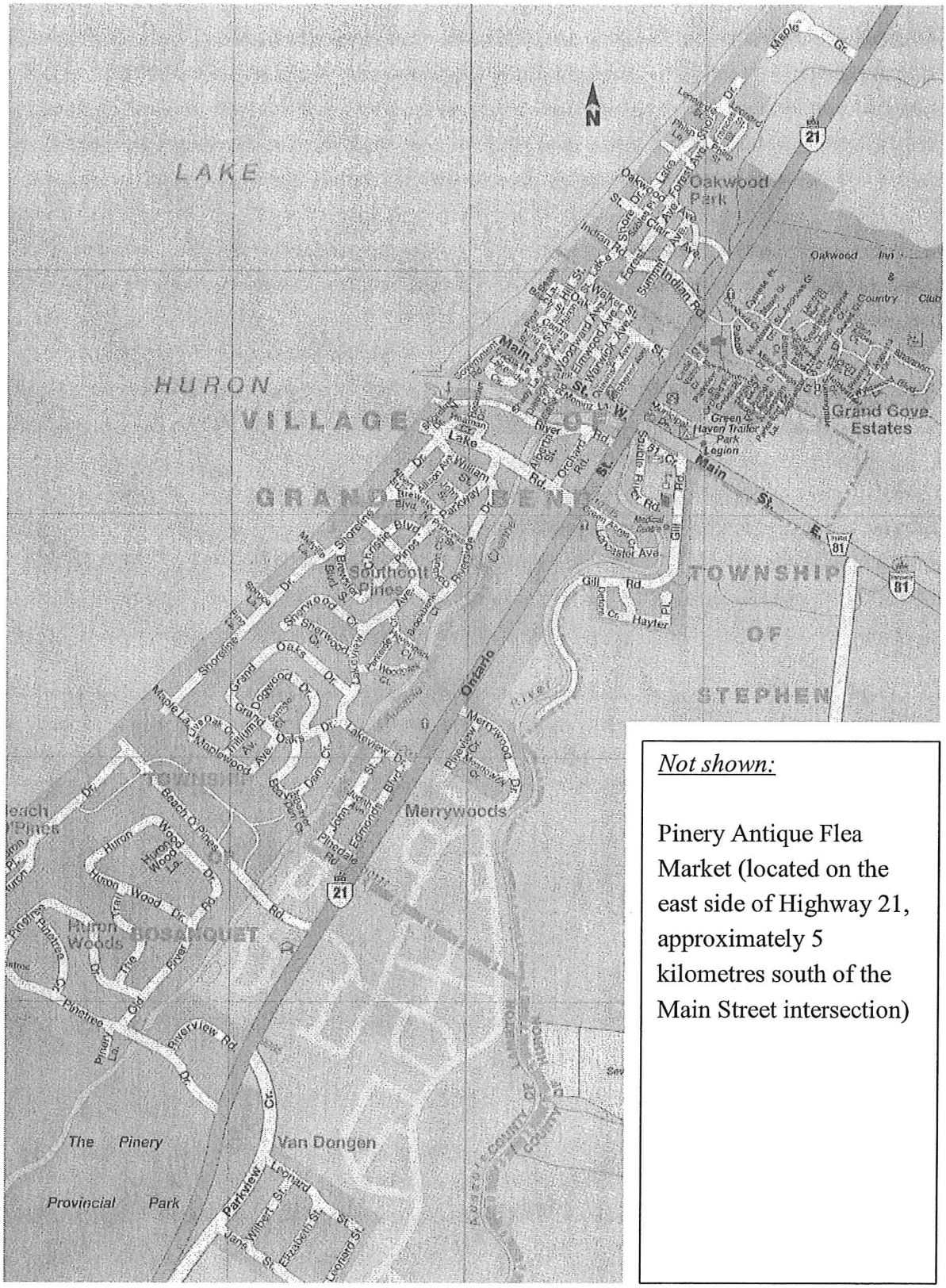
This Highway 21 Connecting Link study evaluated the current operation of the Highway 21 (Ontario Street) corridor within Grand Bend, including three key intersections, and evaluated a growth scenario assuming a significant level of development immediately to the south and to the east of the village. The analysis also identified any necessary improvements.

2.0 STUDY AREA

The primary study area of this assignment is the main portion of Highway 21 (Ontario Street) designated as a Connecting Link within the Village of Grand Bend (between Main Street and the south village limits, approximately Pinedale Road). Within this corridor, particular attention was paid to the intersections of Highway 21 with Main Street (former Highway 81), Lake Road/ Green Acres Crescent, and Lakeview Drive. This study area is shown in ***Figure 2.1***.

Although outside of the formal study area, additional analyses were conducted at the entrance to the Pinery Antique Flea Market, located on Highway 21 some 3 kilometres south of the southerly village limits.

Figure 2.1 - Study Area



Not shown:

Pinery Antique Flea Market (located on the east side of Highway 21, approximately 5 kilometres south of the Main Street intersection)

3.0 EXISTING CONDITIONS

3.1 Existing Roadway Conditions

Throughout the majority of the Village of Grand Bend, Highway 21 has a four-lane cross-section with curbs/gutters and sidewalks. This cross-section is made up of a non-standard lane arrangement, consisting of two southbound lanes, one northbound lane, and one two-way continuous left turn lane. This cross-section extends from the north village limits to a point approximately 450 metres south of Lake Road; to the north and south, the cross-section narrows to two lanes with gravel shoulders. No on-street parking is permitted on Highway 21 throughout the study area. The speed limit is 50 km/h through the main portion of the village, 60 km/h in the rural section of the village to the south, and 80 km/h outside the village limits. Within the village, development is most concentrated along the four-lane section and consists predominantly of automobile-oriented commercial development with numerous accesses; direct access is more limited along the two-lane section to the south, which passes through predominantly low-density residential and undeveloped land.

Within the study area, three intersections were considered:

Highway 21 at Main Street (former Highway 81): This signalized intersection is the primary village intersection. To the west, Main Street is the centre of the village's traditional Central Business District (CBD), with a number of pedestrian- and tourist-oriented commercial facilities. On-street parallel parking is permitted, and access is provided to the main public beach area and village parking facilities. To the east, Main Street becomes Huron County Road 81, heading southeast past a developing suburban commercial area just east of the village limits, to Strathroy and Highway 402. Immediately to the south of the intersection is the Highway 21 bridge over the Ausable River, which is four lanes wide plus sidewalks on each side. There are advance left-turn signal phases for all four directions of traffic.

Highway 21 at Lake Road/Green Acres Crescent: This signalized intersection provides access to neighbourhoods to the west and east of Highway 21, south of downtown. Lake Road and Green Acres Crescent are both two-lane local roadways. There is an advance left-turn signal phase (green arrow) for northbound traffic.

Highway 21 at Lakeview Avenue: This unsignalized intersection (stop control on the Lakeview Avenue approach) is located to the south of the main urban village area, although within the village limits and designated Connecting Link area. Lakeview Avenue is a two-lane local roadway providing access to the adjacent residential subdivision. Immediately to the west, Lakeview Avenue intersects with Edmonds Boulevard, which functions as a service road for residences on the west side of Highway 21.

Figure 3.1 illustrates the lane configurations and traffic control at the study area intersections.

3.2 Existing Traffic Conditions

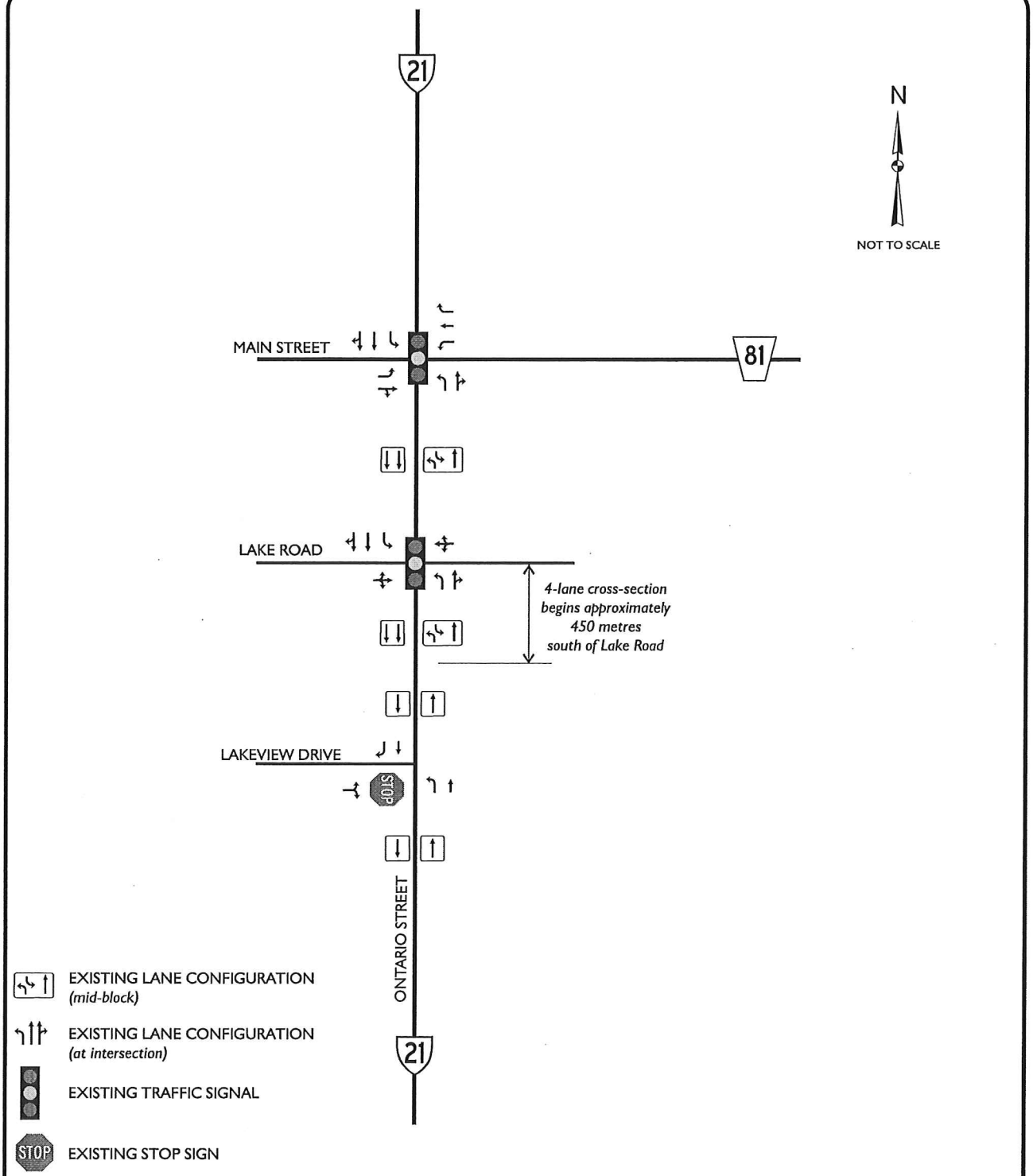
Existing traffic volumes were determined through turning movement counts commissioned by Dillon Consulting Limited at the following locations:

- Highway 21 at Main Street;
- Highway 21 at Lake Road; and
- Highway 21 at Lakeview Avenue.

Surveys were conducted on Friday, August 9, 2002, between 3:00 and 7:00 PM, and Saturday, August 10, 2002, between 10:00 AM and 3:00 PM. These counts represent typical summer weekend traffic levels (i.e., regular two-day weekend). The peak hours of traffic along the corridor were found to be approximately 4:15-5:15 PM on Friday, and 11:30 AM-12:30 PM on Saturday. Peak hour traffic volumes are illustrated in ***Figure 3.2***.

3.3 Existing Intersection Operations

The three study area intersections were analyzed to determine the current operational characteristics in the Friday PM and Saturday mid-day peak hours. These analyses were conducted using the Synchro 6.0 software package.



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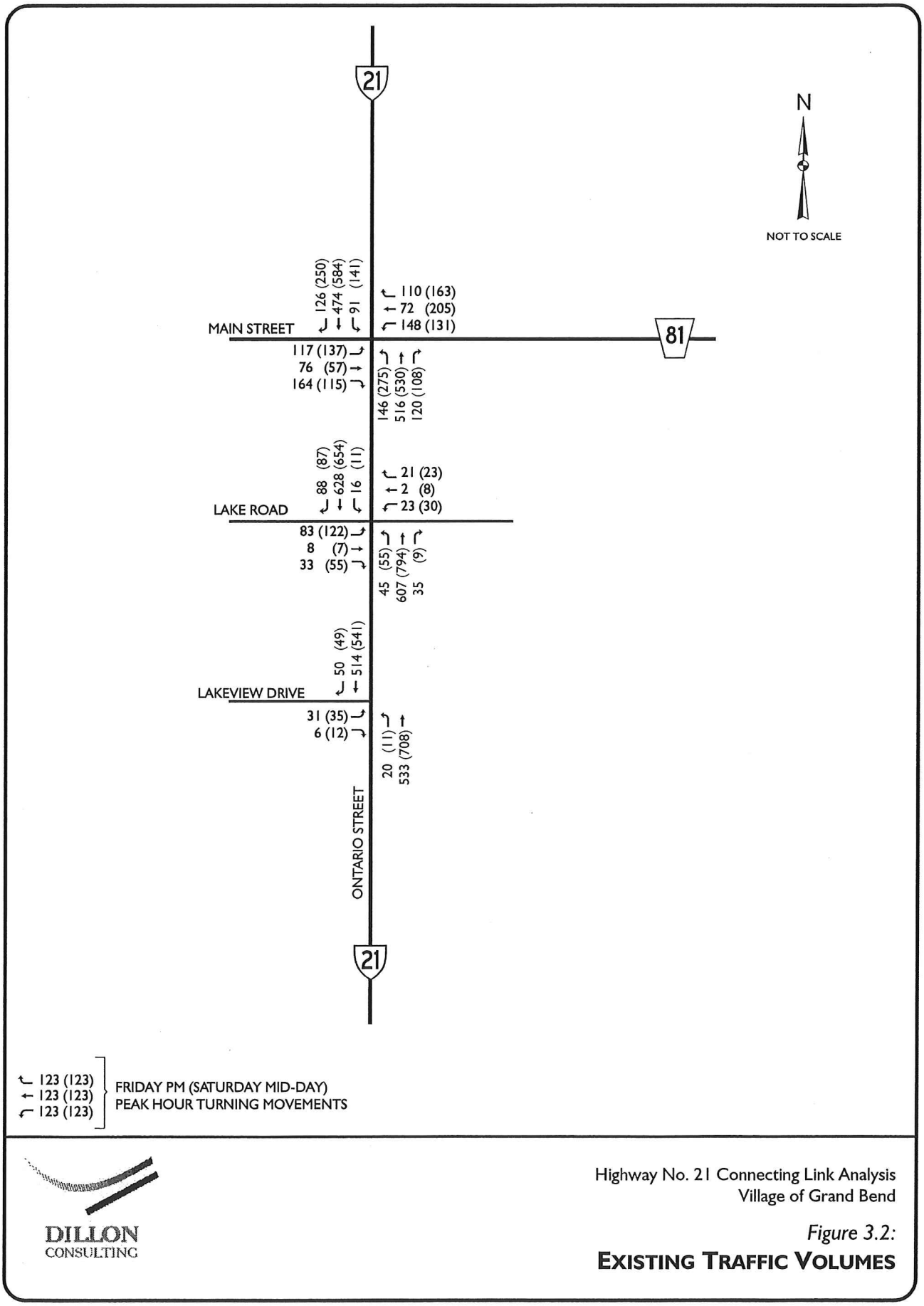
Village of Grand Bend

Figure 3.1:

EXISTING STUDY AREA ROAD NETWORK
LANE CONFIGURATIONS & INTERSECTION CONTROL



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Village of Grand Bend

Figure 3.2:

EXISTING TRAFFIC VOLUMES

Highway 21 Signalized Intersections

Friday PM peak hour and Saturday mid-day peak hour analyses were conducted for the two signalized intersections within the study area (Highway 21 at Main Street and at Lake Road), using existing peak hour traffic volumes and existing traffic control signal timings. For signalized intersection analyses, operational characteristics noted consisted of the following:

- The overall Level of Service (LOS) of the intersection;[1] and
- The average vehicular delay (in seconds) at the intersection.

In addition, the LOS, delay, and volume-to-capacity ratio (v/c ratio, or degree of saturation) were noted for any critical movements at the intersection. Critical movements are defined as:

- any through movement, or any shared through/turning movement, with a v/c ratio of 0.85 or higher; and
- any exclusive turning lane with a v/c ratio of 1.00 or higher.

The results of the analyses are presented in ***Table 3.1*** below.

Table 3.1 - Existing Signalized Intersection Operations

Intersection	Peak Hour	Overall Intersection		Critical Movements			
		LOS	Delay (sec.)	Movement	LOS	Delay	v/c
Highway 21 @ Main Street	Friday PM	E	62.9	NB Through/Right	F	147	1.23
	Saturday	D	50.8	NB Through/Right	F	89.9	1.07
Highway 21 @ Lake Road	Friday PM	B	10.7	N/A	—	—	—
	Saturday	B	17.2	EB Left/Through/Right	E	57.7	0.87

Note: NB = Northbound; EB = Eastbound.

The intersection of Highway 21 and Main Street operates at a poor overall level of service in both the Friday PM and Saturday peak hours, primarily due to the single-lane northbound approach operating over capacity for through/right-turning traffic during the busiest intervals within the peak hour (the "peak within a peak") and due to a number of moderate- to high-demand turning movements.

[1] Level of Service (LOS), applied to an intersection, is a measure qualifying the amount of delay experienced by motorists, expressed either for specific turning movements at the intersection or for the intersection as a whole. Level of Service can range from LOS A (excellent operations, with motorists experiencing little or no delay) to LOS F (failure of the movement or intersection, with motorists experiencing significant delays). A more detailed explanation of LOS is provided in ***Appendix A***.

The intersection of Highway 21 and Lake Road operates at a good overall level of service in both the Friday PM and Saturday peak hours, since lower traffic volumes on Lake Road allow more green time to be allocated to Highway 21. The eastbound approach on Lake Road is identified as being critical in the Saturday peak hour, when eastbound volumes are somewhat higher and demand begins to approach the capacity of the movement.

Highway 21 Unsignalized Intersection

For the unsignalized study area intersection (Highway 21 at Lakeview Avenue), operating conditions were noted for traffic exiting Lakeview Avenue, including the LOS and average delay experienced by motorists, and the v/c ratio for that movement. The analysis results are presented in ***Table 3.2*** below.

Table 3.2 - Existing Unsignalized Intersection Operations

Peak Hour	Lakeview Avenue Eastbound Approach		
	LOS	Delay	v/c
Friday PM	C	21.8	0.22
Saturday	D	32.2	0.31

In both the Friday PM and Saturday peak hours, traffic exiting Lakeview Avenue experiences a moderate (but not inappropriate) level of delay waiting for a gap in traffic on Highway 21. Sufficient gaps are available to service the low volume of eastbound traffic during these time periods.

3.4 Existing Corridor Operations

In addition to the intersections analyzed above, the traffic volumes along the corridor as a whole were assessed at a "planning level," based on assumed lane capacities. Corridor operations were assessed at two locations within the study area:

- Highway 21 south of Lake Road (in town); and
- Highway 21 south of Lakeview Avenue (south limits).

Lane capacities were assumed based on planning capacities used in similar jurisdictions in southern Ontario. For the rural section to the south of the main built-up area, a lane capacity of 1,000 vehicles per hour per lane (veh/h/la) was assumed, or an average of one vehicle every 3.6 seconds; this is a typical planning capacity for a rural two-lane highway. For the four-lane section within the village, a lower capacity of 800 veh/h/la was assumed (one vehicle on average every 4.5 seconds), which is a typical planning capacity for an arterial roadway through an urban area. This lower capacity reflects the increased level of conflict inherent in an urban environment (due to traffic signals and interference from unsignalized intersections and driveways).

At these two locations, traffic volumes were compared to existing road capacity to determine the appropriateness of the roadway cross-section, and identify any lane deficiencies. For the Lake Road location, two lanes of southbound capacity and one lane of northbound capacity were modeled, reflecting the existing cross-section. At the Lakeview Avenue location, a single lane of capacity was assumed in each direction.

The results are summarized in ***Table 3.3*** below.

Table 3.3 - Corridor Analysis - Existing Volumes

Location	Peak Hour	Northbound			Southbound		
		Volume	Capacity	v/c	Volume	Capacity	v/c
South of Lake Rd.	Friday PM	687	800	0.86	684	1,600	0.43
	Saturday	858	800	1.07	739	1,600	0.46
South of Lakeview Ave.	Friday PM	553	1,000	0.55	520	1,000	0.52
	Saturday	719	1,000	0.72	553	1,000	0.55

In practice, it is possible to achieve a v/c somewhat greater than 1.0 (i.e., volumes greater than capacity) through measures such as adjustments to signal timing, and with poorer operations for traffic exiting driveways and side streets (longer delays due to fewer gaps in main street traffic). It is also possible that, in practice, the lane capacity is higher than the assumed theoretical capacity. For planning purposes, however, a v/c ratio of 0.90 or greater is typically an indicator that some form of additional capacity is required. Analyses of existing traffic volumes under the existing roadway cross-section indicate that the northbound direction just south of Lake Road likely requires some additional capacity to meet existing demands. This additional capacity can be provided through new or widened roadways, or by reducing or managing the traffic demand (through such measures as carpooling, public transit, or adjusting trip times to occur outside of the peak). Since the latter is not a realistic option to reduce vehicular demands in the village, additional capacity will likely take the form of road widenings or new roads.

4.0 FUTURE TRAFFIC VOLUMES

Traffic volumes were estimated for a ten-year build-out scenario. These future volumes are comprised of existing traffic volumes, increased by a growth factor to account for general traffic growth along Highway 21, plus traffic generated by anticipated developments in the immediate vicinity.

4.1 Background Traffic Growth

Historical traffic counts were examined from 1998 and 2001. These counts indicated an increase in daily summer weekend traffic on Highway 21 of approximately 1.5% per year. This growth factor was applied to future ten-year horizon north/south traffic volumes on Highway 21, to estimate the effect of increased development outside the immediate vicinity of Grand Bend.

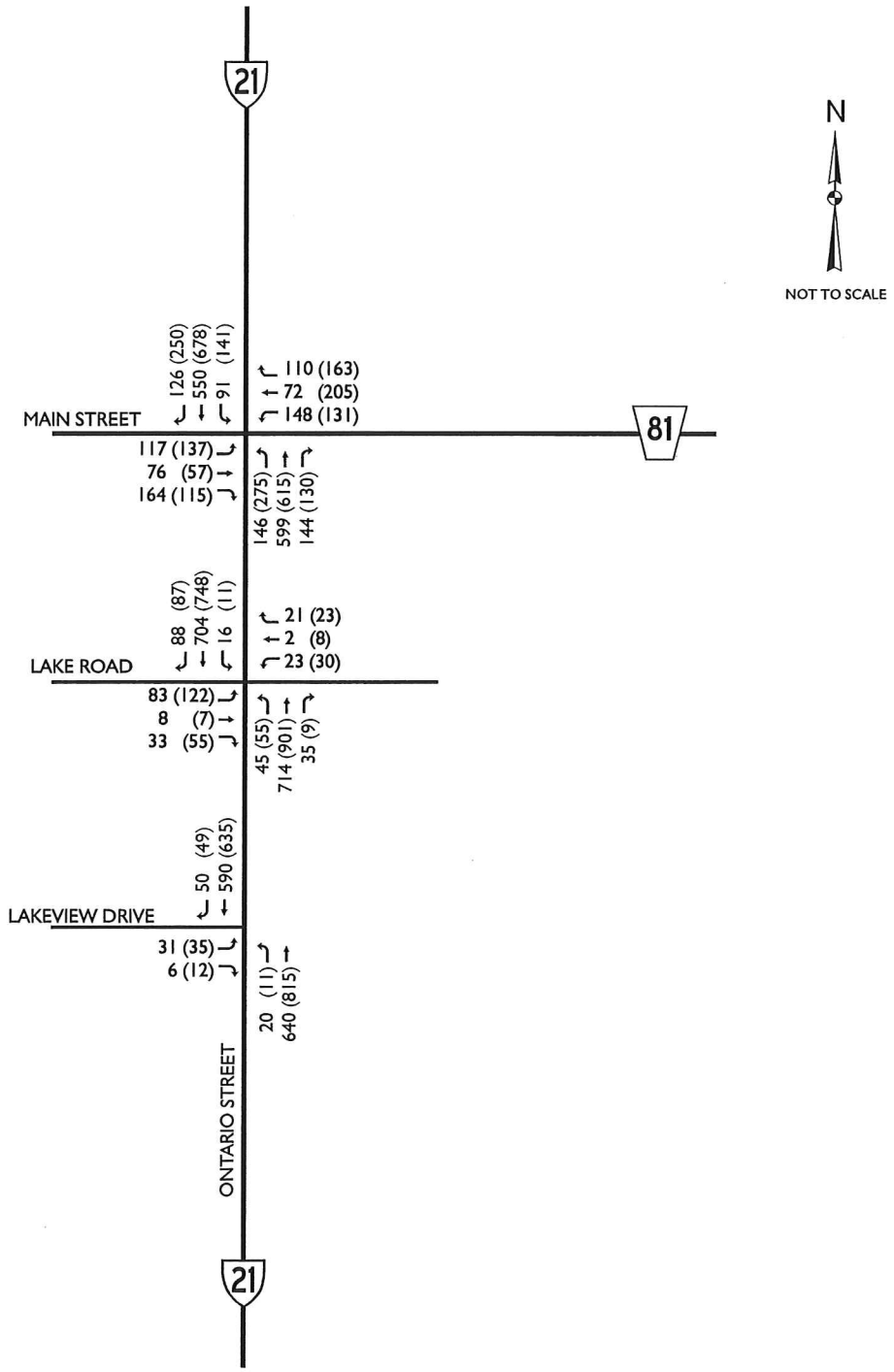
In addition, a growth factor was applied to account for possible future expansion of the Grand Bend Motorplex, which is a significant trip generator on County Road 81 approximately 5 kilometres southeast of Grand Bend. As no specific traffic volumes or trip generation data were available for the Motorplex, a 20% increase in northbound right turns was assumed at the Highway 21/Main Street intersection, reflecting a possible increase in competitors originating from the south on Friday evening, and attendees during the day on Saturday.

The resulting background traffic volumes are illustrated in ***Figure 4.1***.

4.2 Main Street East Development

A March 1995 draft report prepared by MIG Engineering Ltd. in association with McCormick Rankin (*Village of Grand Bend Collector Road Study*) assessed the Friday PM peak hour traffic impact of a significant amount of development expected along the Main Street East corridor, east of the Ausable River and Walker Drain, consisting of:

- 340,000 square feet of highway commercial development along both sides of Main Street East;
- 1,057 medium-density residential units (townhouse, multi-family, and seniors' apartments) to the north of the proposed highway commercial strip; and
- 1,404 single-family residential units to the south of the proposed highway commercial strip.



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Village of Grand Bend

Figure 4.1:

2012 FUTURE BACKGROUND TRAFFIC VOLUMES

Although some of the commercial development assessed in that report has since opened, this development was not existing during the August 2002 counts conducted for the present study existing conditions, and therefore the August 2002 counts would not reflect any of the future site traffic generated by MIG.

The March 1995 MIG report analyzed a long-term development scenario, to occur in an unspecified year. It is considered unlikely that either the full development, or the supporting roadway infrastructure, will be in place by the 2012 analysis horizon year used in the present assessment. Within the ten-year horizon, it is anticipated that of the development assessed in the MIG report, 70% of the commercial and 20% of the residential development on the north side of Main Street East will be built out, and no development will occur on the south side of Main Street East. Thus, for the purposes of this analysis, the future development on Main Street East is assumed to consist of:

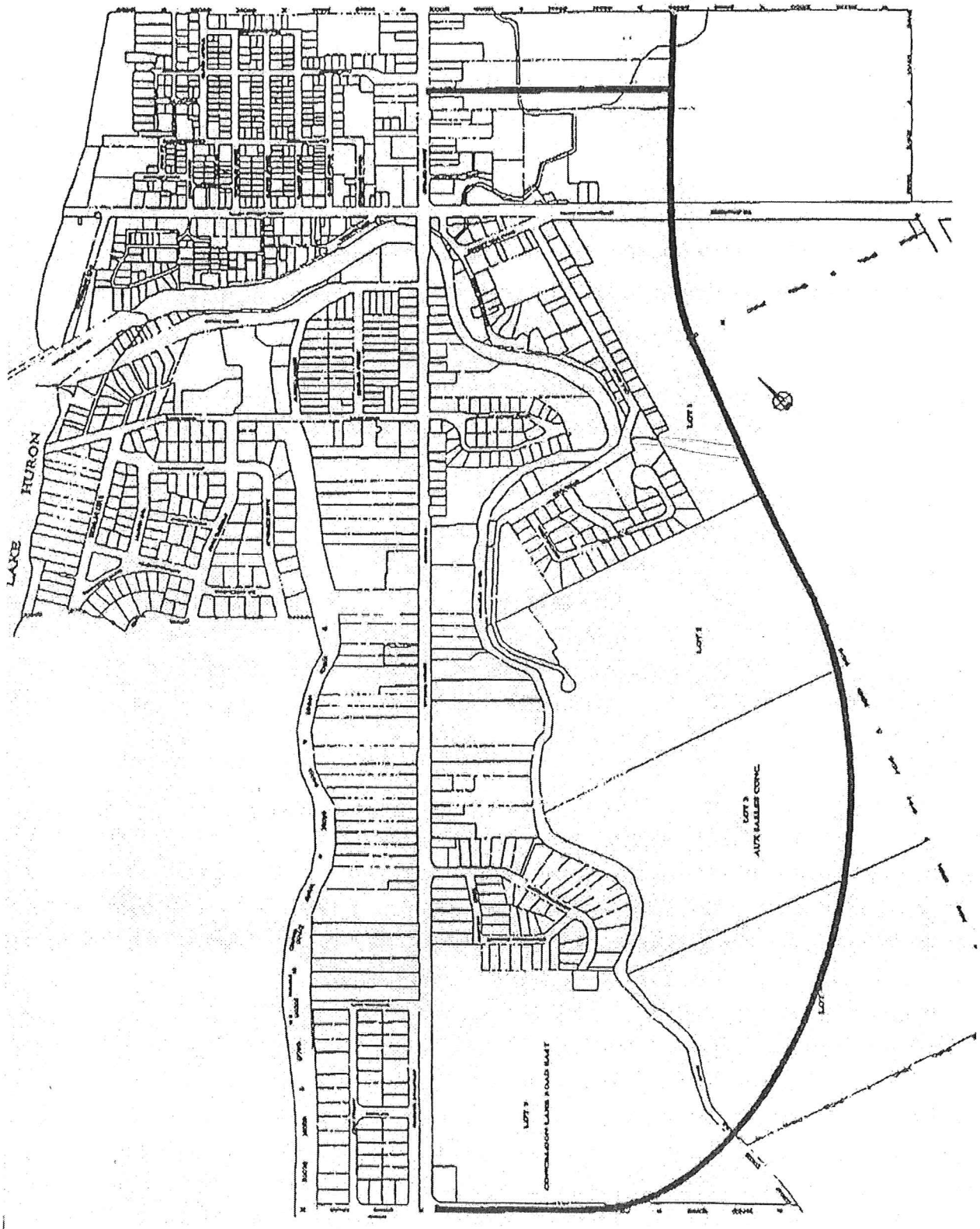
- 119,000 square feet of highway commercial development along the north side of Main Street East (of which 85,000 square feet already exist); and
- 211 medium-density residential units (townhouse, multi-family, and seniors' apartments) to the north of the proposed highway commercial strip.

In addition, the MIG report assumed some trips generated by this new development area would be accommodated through a new collector ring road (illustrated in ***Figure 4.2***). It has since become apparent that a collector ring road is unlikely to be built within the present study's 10-year timeframe, if at all; consequently, the trips assigned to the proposed collector road in the MIG report have been re-assigned for the purpose of this study to the existing roadway network.

Since the MIG report only assessed the Friday PM peak hour, the trip generation was updated to include the Saturday peak hour scenario. Trips were generated using rates and equations published in the Institute of Transportation Engineers' (ITE) *Trip Generation* (7^{th} edition). Trip generation was also re-calculated for the Friday PM peak hour due to the reduced level of development anticipated by 2012.

Although the previous report reduced the generated trips by 20% to reflect site-generated trips with both origin and destination within the study area (traffic not having to use Ontario or Main Streets to access the site), the current trip generation assumes no reduction (reflecting the reduced magnitude of development, and the reduced connectivity due to the elimination of the collector ring road).

Figure 4.2 - Collector Ring Road Recommended in 1995 MIG Report
(Source: MIG Engineering Ltd. : *Village of Grand Bend Future Collector Road Study*, March 1995)



The MIG trip generation did not account for any pass-by traffic related to the commercial development. It is noted in the ITE Manual (*ITE Trip Generation Manual*, 6th Edition) that in the case of new commercial development, trips are attracted from the passing traffic on the adjacent streets, that is, traffic already "passing by" the site on the way from one location to another. In this case, the driveway volume is different from the amount of traffic added to the street system. For the updated Main Street East development, the trip generation for the commercial component was adjusted to reflect 20% pass-by trips (traffic already on the road network), which is a typical rate for general commercial developments, and 80% primary trips (new traffic to the road network).

Table 4.1 summarizes the trips generated for the Saturday mid-day peak hour, and the recalculated trip generation for the Friday PM peak hour.

Table 4.1 - Trip Generation - Main Street East Development

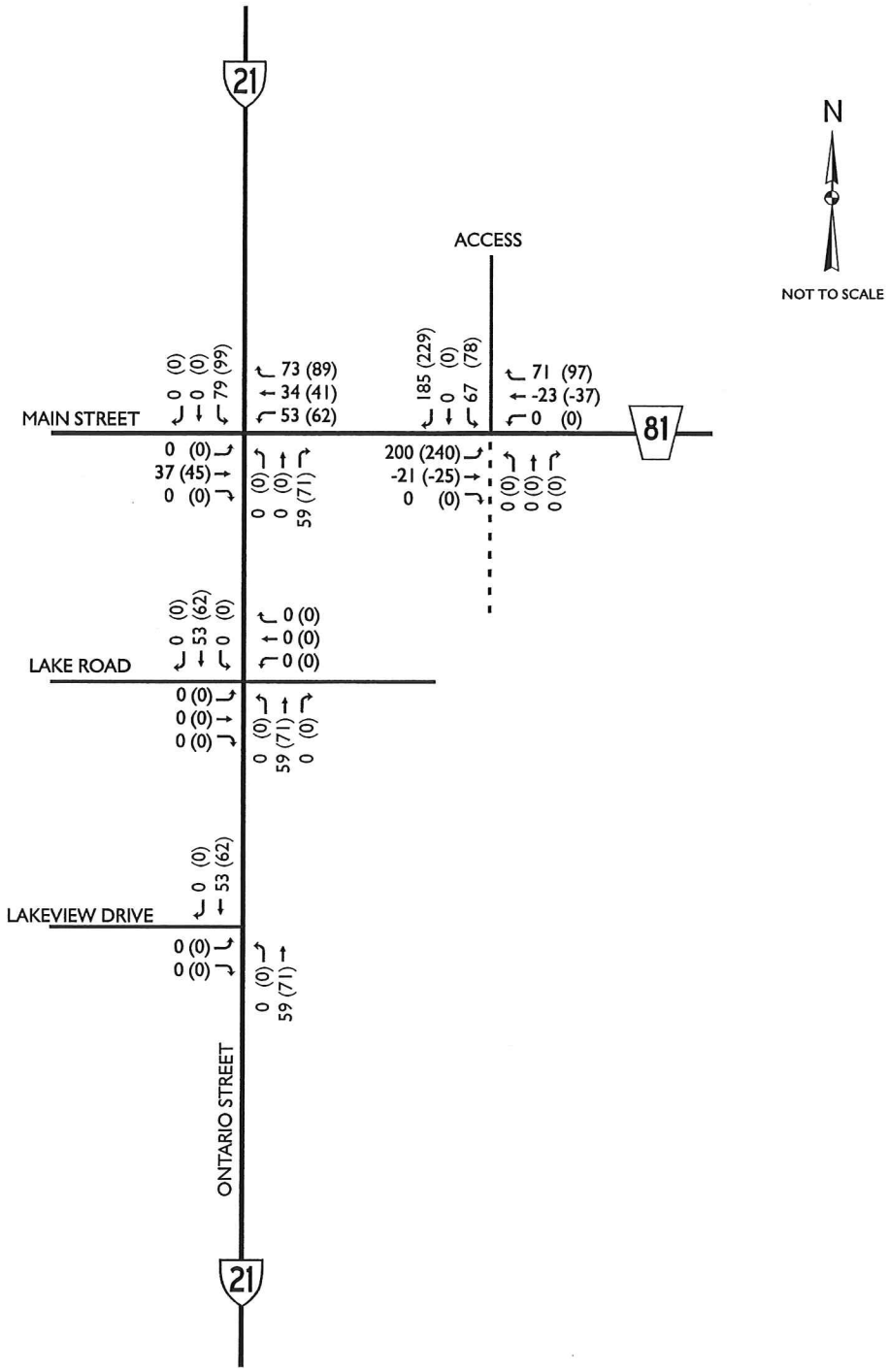
Land Use	ITE Code	Friday PM Peak			Saturday Peak		
		In	Out	Total	In	Out	Total
Residential North of Main Street (211 units)	230	70	40	110	56	48	104
Commercial North of Main Street — Primary (119,000 ft^2)	820	171	186	357	246	226	472
Commercial North of Main Street — Pass-By (119,000 ft^2)	820	43	46	89	62	57	119
TOTAL:		**284**	**272**	**556**	**364**	**331**	**695**

The new Saturday peak hour trips were distributed to the existing roadway network (not including the aforementioned proposed collector road) using a similar distribution to that of the previously distributed Friday trips, as shown in the March 1995 report. This distribution is summarized in ***Table 4.2*** below.

Table 4.2 - Trip Distribution - Main Street East Development

Trips To/From	Percent
Highway 21 North	33%
Highway 21 South	32%
Main Street West	15%
County Road 81 East	20%

The distributed trips were adjusted a final time to account for synergy between the Main Street East development and additional development on Highway 21 South. It is likely that some of the trips generated by the commercial development will be attracted from the new residential areas to the south of the village limits, and would be accounted for in the trip generation for that development (as outlined below in Section 4.3). To account for this "double counting," primary trips generated by the Main Street East commercial development destined to and from the south were reduced by one-third (approximately 10% of the total commercial development trip generation). The resulting new traffic generated by the proposed Main Street East commercial and residential development is illustrated in ***Figure 4.3***.



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Village of Grand Bend

Figure 4.3:

MAIN STREET EAST DEVELOPMENT
TRAFFIC VOLUMES

4.3 Highway 21 South Development

A second major development area has been identified in the immediate vicinity of Grand Bend, located generally at the southerly village limits between Highway 21 and the Ausable River. This development is envisioned to consist of:

- 600 single-family residential units;
- 50 townhouse condominium units;
- an 18-hole golf course; and
- a seven-storey hotel (assumed to contain 200 guest rooms).

Trips were generated for the Friday PM and Saturday mid-day peak hours, using rates and equations published in ITE's *Trip Generation* (7th edition). The trips generated by this development are summarized in ***Table 4.3*** below.

Table 4.3 - Trip Generation - Highway 21 South Development

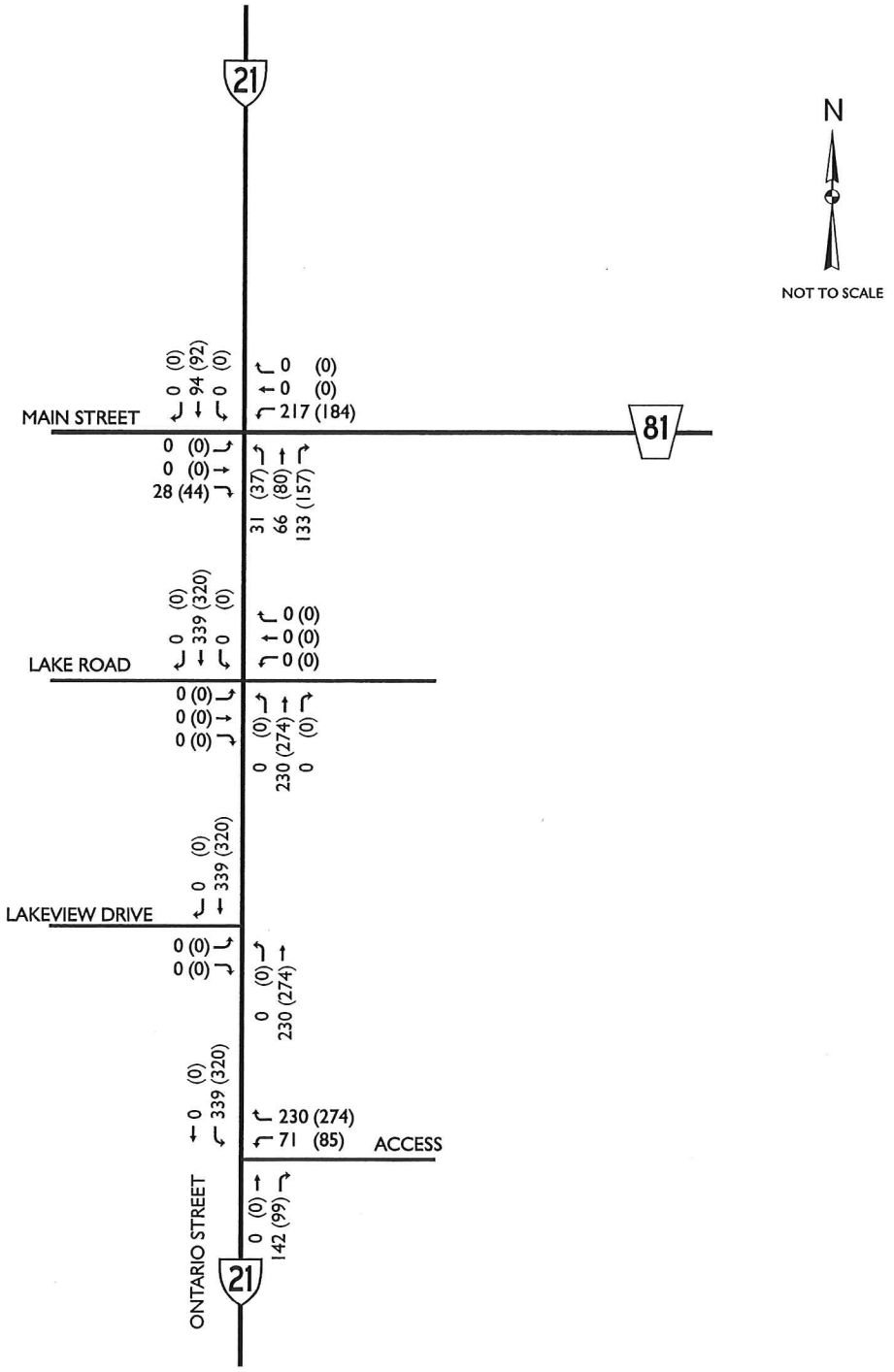
Land Use	ITE Code	Friday PM Peak			Saturday Peak		
		In	Out	Total	In	Out	Total
Single-Family Detached Residential (600 units)	210	382	224	606	305	259	564
Condominium Residential (50 units)	230	17	9	26	13	11	24
18-Hole Golf Course	430	22	27	49	41	42	83
Hotel (200 rooms)	310	63	55	118	81	63	144
TOTAL:		**484**	**315**	**799**	**440**	**375**	**815**

These site trips were then assigned to the existing roadway network (assuming no new collector ring road) based on anticipated catchment areas. ***Table 4.4*** below summarizes the directional distribution assumed for each land use. ***Figure 4.4*** illustrates the Friday PM and Saturday mid-day traffic volumes generated by the Highway 21 South development.

Table 4.4 - Trip Distribution - Highway 21 South Development

Land Use	Scenario	Highway 21		Main Street		Local*
		North	South	West	East	
Single-Family Residential	All	20%	25%	5%	50%	0%
Condominium Residential	All	20%	25%	5%	50%	0%
Golf Course	All	40%	20%	10%	20%	10%
Hotel	Friday PM Inbound	10%	60%	10%	20%	0%
	All others	15%	15%	30%	20%	20%

**Indicates trip to/from north not passing through Main Street intersection.*



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Village of Grand Bend

Figure 4.4:

HIGHWAY 21 SOUTH DEVELOPMENT TRAFFIC VOLUMES

4.4 Total Future Traffic Volumes

The total volume of traffic expected on the area roadway network in the future scenario was determined by adding the background traffic growth and traffic generated by the two new development areas to existing traffic volumes. The resulting future total traffic volumes are illustrated in ***Figure 4.5***.

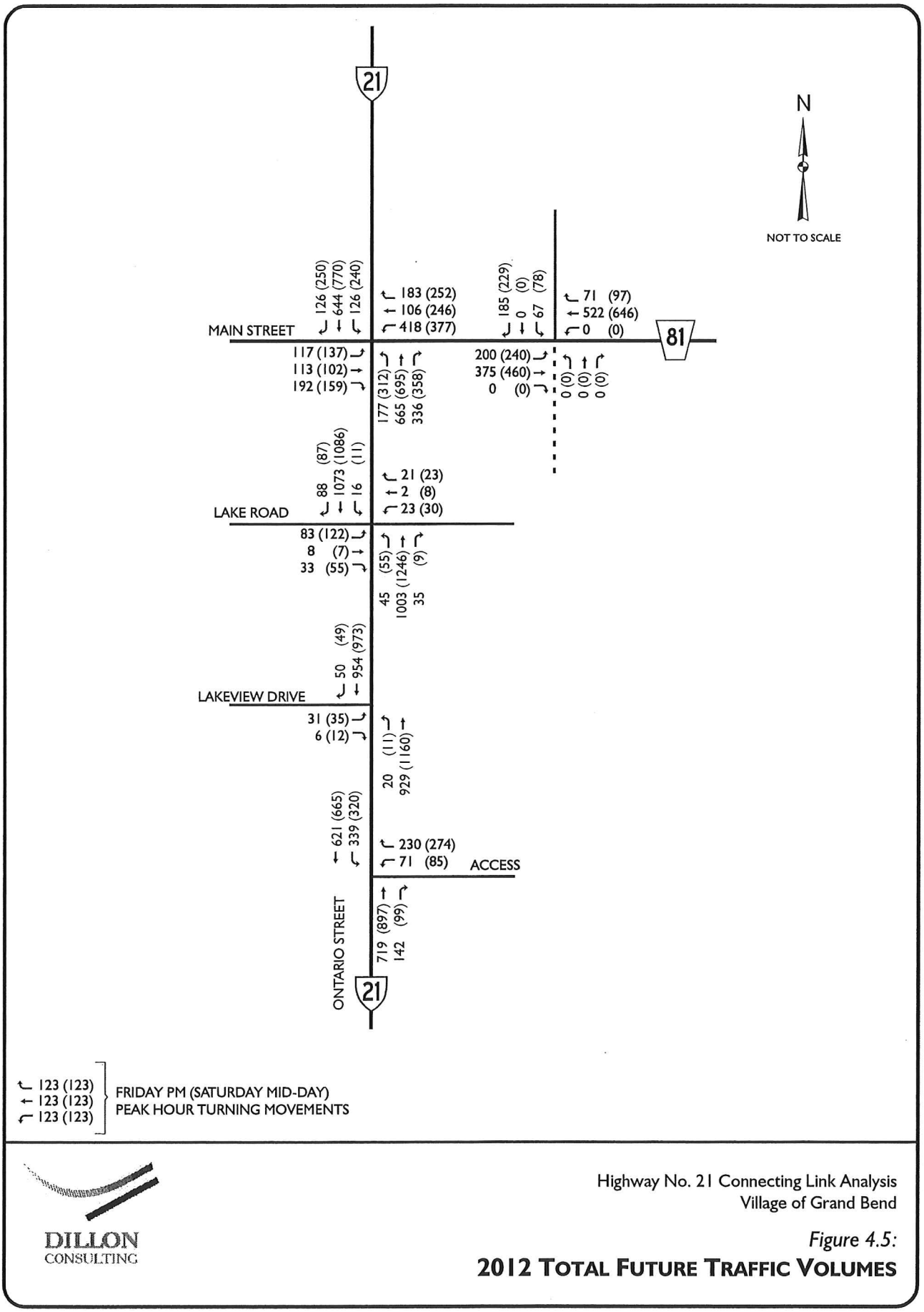

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Figure 4.5:

2012 TOTAL FUTURE TRAFFIC VOLUMES

5.0 FUTURE TRAFFIC OPERATIONS

5.1 Future Intersection Operations

The three study area intersections were analyzed again to determine the operational characteristics expected in the Friday PM and Saturday mid-day peak hours under the calculated future total traffic volumes, assuming no changes to the existing area roadway network. These analyses were conducted using the Synchro 6.0 software package.

Highway 21 Signalized Intersections

Friday PM peak hour and Saturday mid-day peak hour analyses were conducted for the two signalized intersections within the study area (Highway 21 at Main Street and at Lake Road), using future total peak hour traffic volumes and optimized traffic control signal timing. Noted were the overall intersection LOS and average vehicular delay, and the LOS, delay, and v/c ratio for any critical movements at the intersection. The results of the analyses are presented in ***Table 5.1*** below.

Table 5.1 - Future Signalized Intersection Operations

Intersection	Peak Hour	Overall Intersection		Critical Movements			
		LOS	Delay (sec.)	Movement	LOS	Delay	v/c
Highway 21 @ Main Street	Friday PM	F	163	EB Through/Right	F	108	1.07
				WB Left	F	>200	1.72
				NB Through/Right	F	>200	1.53
				SB Left	F	>200	1.32
	Saturday	F	>200	EB Through/Right	F	118	1.11
				WB Left	F	>200	2.09
				NB Left	D	49.2	1.01
				NB Through/Right	F	>200	1.75
				SB Left	F	>200	1.63
				SB Through/Right	E	77.7	1.05
Highway 21 @ Lake Road	Friday PM	A	10.0	N/A	—	—	—
	Saturday	E	41.4	EB Left/Through/Right	F	118	1.06
				NB Through/Right	E	69.0	1.08

Note: NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound.

With additional development and background traffic added to existing volumes, operating conditions are expected to worsen significantly at the Highway 21/Main Street intersection in both the Friday PM and Saturday peak hours, with a number of turning movements operating with high volumes (including the northbound right-turn and westbound left-turn at over 400 vehicles per hour) and high volumes in the single northbound through/right-turn lane. Turning movements on all approaches are expected to be significantly over capacity, and delays to traffic are expected to significantly exceed three minutes, both for those over-capacity movements and for the entire intersection on average.

At the Highway 21/Lake Road intersection, the single-lane northbound approach is expected to be nearing capacity during the Friday PM peak hour, and well over capacity in the Saturday peak hour. The eastbound approach is also expected to be over capacity in the Saturday peak hour.

Highway 21 Unsignalized Intersection

For the unsignalized study area intersection (Highway 21 at Lakeview Avenue), operating conditions were determined for traffic exiting Lakeview Avenue, including the LOS and average delay experienced by motorists, and the v/c ratio for that movement. The analysis results are presented in ***Table 5.2*** below.

Table 5.2 - Future Unsignalized Intersection Operations

Peak Hour	Lakeview Avenue Eastbound Approach		
	LOS	Delay	v/c
Friday PM	F	148	0.69
Saturday	F	>200	1.22

In both the Friday PM and Saturday peak hours, increased through traffic on Highway 21 will reduce the number of gaps available to traffic turning from Lakeview Avenue, and increase delays to levels considered unacceptable. The eastbound approach would be approaching capacity in the Friday PM peak hour, and well over capacity in the Saturday peak hour, with few gaps available for eastbound traffic.

5.2 Future Corridor Operations

As in the existing scenario, the predicted future traffic volumes along the corridor as a whole were assessed at a "planning level." This assessment assumed no change in roadway network (number of lanes, new roadway corridors) and continued to assume a maximum practical capacity of 1,000 passenger cars per hour per lane (pc/h/la) within the rural area south of the village, and of 800 pc/h/la on the four-lane section within the village.

At the same two locations as assessed for existing conditions above (south of Lake Road, and south of Lakeview Avenue), predicted future traffic volumes were compared to existing road capacity to determine the appropriateness of the existing roadway cross-section, and to identify any lane deficiencies. The results are summarized in ***Table 5.3***.

Table 5.3 - Corridor Analysis - Future Volumes

Location	Peak Hour	Northbound			Southbound		
		Volume	Capacity	v/c	Volume	Capacity	v/c
South of Lake Rd.	Friday PM	1,083	800	1.35	1,124	1,600	0.70
	Saturday	1,310	800	1.64	1,172	1,600	0.73
South of Lakeview Ave.	Friday PM	949	1,000	0.95	960	1,000	0.96
	Saturday	1,171	1,000	1.17	986	1,000	0.99

Based on calculated future volumes and assumed lane capacities, the single-lane section of Highway 21 south of Lakeview Avenue is expected to operate either at or above capacity in both the Friday PM and Saturday peak hours.

6.0 ALTERNATIVE SOLUTIONS

Based on the analysis of future conditions in the study area, and the problems identified, four options have been identified for the Municipality of Lambton Shores to consider in addressing likely future traffic problems.

Option 1 – "Do Nothing"

This option represents the status quo. It is recognized that this option does not solve the future problems anticipated for the area. The municipality, and its residents, can select this option and continue to accept the fact that during the summer there are traffic problems on the weekend in the downtown area (i.e., continue to live with the problem). However, with the anticipated development along Main Street East and in south Grand Bend, there will be more pressures placed on the Highway 21/Main Street intersection, not only in the summer peak period but also in the off-peak periods during the year. The Village of Grand Bend is certainly vulnerable to future gridlock conditions if this option is selected, and additional development continues to proceed in the village.

Option 2 – Reallocation of Existing Lanes

As noted above, the four-lane section of Highway 21 currently has a non-traditional lane configuration (two southbound lanes, one centre two-way left-turn lane, and one northbound lane). Corridor capacity analyses indicate that, while the two southbound lanes will provide sufficient capacity for future traffic demand, the single northbound lane will not be sufficient to accommodate the projected northbound traffic. However, the excess southbound capacity, and the difference between northbound and southbound volumes, are not sufficient to reverse the lane arrangement (i.e., a reversal to one southbound lane plus two northbound lanes). Such a reversal would instead shift the capacity problems from the northbound to the southbound direction. In addition, this change would not address capacity problems at the Main Street intersection, which would continue to be a bottleneck point.

Other related options include the following:

- **Reverting to two through lanes per direction** (i.e., removal of the two-way left-turn lane). This would provide an extra lane for northbound traffic; however, the inside lanes for each direction would have reduced capacity, depending on the volume of left turns that blocked through traffic. The two-way left-turn lane is generally preferable for accommodating mid-block left-turning traffic, from an operational and safety perspective.

- **Installing overhead lane control signals.** A number of municipalities in Ontario (e.g., Fort Erie [Peace Bridge]; Toronto [Jarvis Street]; Hamilton [mountain accesses]) have roadways where certain lanes switch directions during peak periods to accommodate heavier traffic flows in one direction. This lane reversal is controlled through signage and overhead signals (showing a green downward arrow or a red "X"). As noted above, the projected traffic volumes on Highway 21 are not unbalanced enough to warrant such a measure; in addition, this measure would raise safety issues due to the two-way left-turn lane and the high proportion of tourist traffic less familiar to the area.
- **Widening Highway 21 by one lane.** This measure would result in sufficient link capacity through the urban area of Grand Bend, based on assumed lane capacities (as discussed above in Section 5.2). This option may not be physically realistic, however, depending on right-of-way constraints, and would still leave the key Main Street intersection well over capacity during the Saturday peak hour.

Option 3 – Major Improvements to the Highway 21/Main Street Intersection

The Highway 21/Main Street intersection is the primary constraint within the study area road network. The identified future developments will add a significant amount of traffic to an already saturated intersection, and in particular to low-capacity turning movements. In order to meet future traffic demands at the intersection, additional capacity would have to be constructed, primarily in the form of additional exclusive turning lanes.

In order to fully accommodate the projected Friday PM peak hour traffic volumes, the intersection would need to be widened to add the following:

- new exclusive eastbound, northbound, and southbound right-turn lanes; and
- a second northbound through traffic lane.

This configuration would adequately handle the projected Friday PM peak hour traffic volumes, but would still result in the southbound through movement and westbound left-turn movement being over capacity in the busier Saturday mid-day peak hour. In order to fully accommodate the Saturday peak hour volumes, the intersection would need to be widened to add the following (compared to existing conditions):

- new exclusive eastbound, northbound, and southbound right-turn lanes;
- a second northbound through traffic lane; and
- second northbound and westbound left-turn lanes (i.e., creating dual northbound and westbound left-turn lanes, including a second lane on the westbound departure), with conversion of those movements to fully-protected traffic signal control.

This option would involve shifting and reconstructing the intersection of Highway 21 and Main Street. The most likely future location of the intersection would be to the east, since the topography to the west and south produces a significant constraint. For this intersection relocation to become viable, there would be a need to acquire properties on the northeast and southeast corner of the existing intersection. Also, there will be a need to widen or shift the bridge on the south approach to the intersection. With a relocated intersection, an improved level of service could be provided at this intersection. Given that this intersection is the main source of operational problems in the study area, improvements here would go a long way towards providing a better level of service for traffic as a whole in the study area.

Option 4 – Protect for a Future Road Connection to By-Pass the Existing Highway 21/Main Street Intersection (Ring Road Concept)

While Option 3 will certainly provide some relief to future traffic problems, Option 4 is ultimately what is needed for the area in the long term, given the emphasis on developing along Main Street East. By redirecting some of the turning movements away from the Highway 21/Main Street intersection, a by-pass could also eliminate the need to provide additional capacity at this intersection (widenings identified in Option 3).

Through this option, the Municipality should start (if they haven't done so already) protecting for a future right-of-way to the southeast of the intersection (similar to the ring road identified in the 1995 MIG report). This acquisition of property/right-of-way could be done in phases, where the south by-pass is done first, followed by the north by-pass (north of Main Street East) at a later date.

As with the creation of any by-pass, there will likely be some concern from businesses in the area. For those businesses that are dependent on through traffic, the municipality could offer to relocate them out to the by-pass. Those businesses that are local in nature will no longer have to contend with heavy volumes of through traffic (consisting of aggregate trucks, etc.). With a by-pass, there will be an opportunity to transform Ontario Street into a low-volume "local" road, which can cater to local residents and tourists in an environment that is not bisected by heavy through traffic volumes. "New" transportation options on this section of Ontario Street may be viable that weren't options previously; for example, with lower traffic volumes on Highway 21, it may be possible to reallocate the existing pavement width to include one through lane per direction, one two-way left-turn lane, and one bicycle lane per direction. Residents, customers, and tourists would then have the realistic option of walking or bicycling along Ontario Street in the summer months without fear of high traffic volumes, thereby increasing their enjoyment of the community.

7.0 PINERY ANTIQUE FLEA MARKET

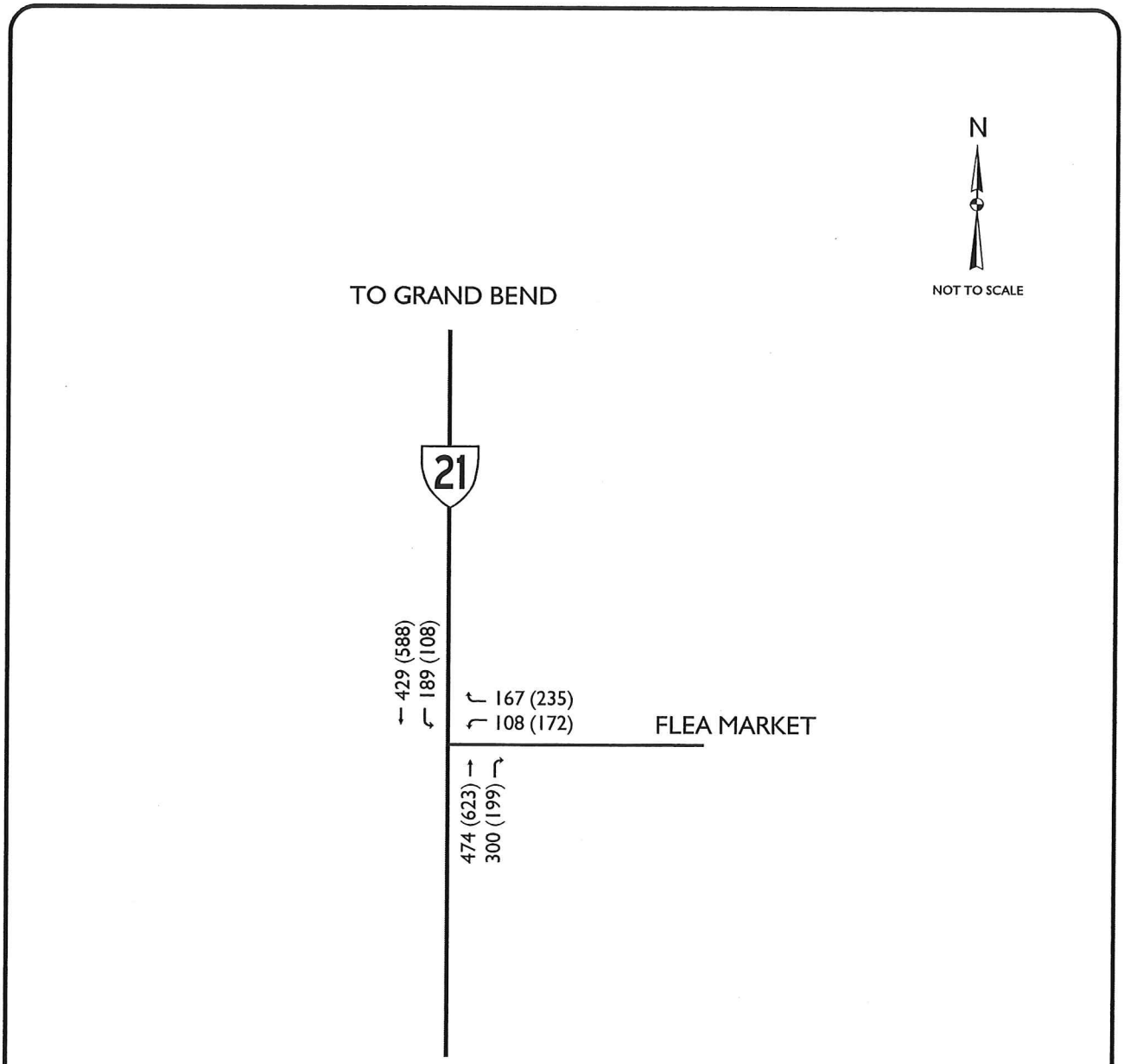
The Pinery Antique Flea Market is a significant trip generator on Sundays, located on the east side of Highway 21 approximately 5 kilometres south of Main Street. Although the flea market is situated approximately 3 kilometres south of the Connecting Link portion of Highway 21, the scope of the present study was expanded at the request of the Municipality of Lambton Shores to include an assessment of traffic operations at the Highway 21 intersection with the flea market driveway.

7.1 Existing Conditions

The flea market driveway is an unsignalized access to Highway 21; no exclusive turning lanes are provided on Highway 21. The intersection typically experiences a significant level of congestion on Sundays, due to a combination of heavy through traffic volumes on Highway 21 and high-demand turning movements into and out of the flea market. To help control traffic and allow the large volumes of flea market traffic to enter and exit the site driveway, the flea market hires a paid duty Ontario Provincial Police (OPP) officer to direct traffic during the busiest hours of traffic (typically no more than four hours per Sunday).

Existing traffic volumes at this intersection were determined through turning movement counts commissioned by Dillon Consulting Limited and conducted on Sunday, August 17, 2003, between 8:00 AM and 5:00 PM. The resulting count data reflect typical traffic volumes generated by the flea market on a summer weekend. Sunday mid-day and afternoon peak hour traffic volumes are illustrated in ***Figure 7.1***.

The turning movement count data indicated a high volume of traffic generated by the flea market (during the mid-day period, over 350 vehicles both entering and exiting the flea market), combined with a reasonably high volume of northbound and southbound through traffic on Highway 21 (typically between 400 and 600 vehicles per hour). These volumes are well above what can be reasonably handled by an unsignalized intersection, and satisfy the Ministry of Transportation of Ontario (MTO) combination signal warrant (minimum vehicular volume plus delay to cross traffic). Although Sunday trips were not generated for the additional development identified in Section 4 above, it is safe to assume that these future developments plus any additional background traffic growth on Highway 21 will put additional pressure on the intersection (both in terms of northbound and southbound through traffic, and additional potential flea market patrons.



↰ 123 (123)
← 123 (123)
↱ 123 (123)
} SUNDAY AM (PM) PEAK HOUR TURNING MOVEMENTS

DILLON CONSULTING

Highway No. 21 Connecting Link Analysis
Village of Grand Bend

Figure 7.1:

EXISTING TRAFFIC VOLUMES AT FLEA MARKET DRIVEWAY

7.2 Potential Future Options

Four potential options were identified to control traffic at the flea market location in the future.

Option 1 - "Do Nothing"

This scenario represents the status quo, with roadway geometry and traffic control remaining as existing, and with the flea market continuing to use paid duty officers to control traffic for at least part of the day. Currently, the rate for a paid duty officer is approximately $50 per hour, with a minimum of four hours, or $200 per Sunday. Assuming an officer is required on Sundays over approximately one-third of the year (late May to early September), this works out to approximately $3,500 per year in traffic control costs. Traffic operations would continue to be poor, and would deteriorate further as either traffic on Highway 21 or activity at the flea market increase.

Option 2 - Build Exclusive Turning Lanes

In this scenario, Highway 21 and the flea market driveway would be widened to allow for an exclusive southbound left-turn lane and an exclusive northbound right-turn lane into the flea market, and an exclusive westbound right-turn lane out of the flea market. This would allow through traffic on Highway 21 to continue unimpeded, and would allow right turns out of the flea market to bypass left-turning vehicles waiting for a gap in northbound and southbound traffic. However, there would still be a significant capacity deficiency for westbound traffic leaving the flea market, and the delays for these movements could actually increase as free-flow vehicles on Highway 21 would be less likely to allow "courtesy gaps" (i.e. waving motorists in).

Estimated construction costs for these exclusive turning lanes are estimated based on a general benchmark cost of $500 per metre of single-lane widening, including tapers. Taper lengths, parallel (deceleration) lane lengths, and storage lengths for Highway 21 are based on the Ministry of Transportation, Ontario (MTO) manual *Geometric Design Standards for Ontario Highways*, and assuming a 100 km/h design speed. For the flea market exit, since the left turn movement is well over capacity, the left turn queue (and right-turn lane length required to bypass the left turn queue) is theoretically infinite. For costing purposes, a 50-metre storage lane and 40-metre taper were assumed.

Direction	Description	Length (m)	Cost *(×$1,000)*
Southbound Left Turn	Entry Taper	160	$80
	Parallel Lane	70	$35
	Storage Lane	70	$35
	Widening Through Intersection	45	$23
	Runout Taper	160	$80
Northbound Right Turn	Right Turn Taper	80	$40
Westbound	Entry Taper	40	$20
	Storage Lane	50	$25
		TOTAL:	**$338**

Based on the aforementioned assumptions, geometric modifications to the intersection would cost approximately $338,000. Of this figure, $253,000 (or roughly 75% of the total cost) is related to the southbound left turn lane, which would result in the most benefits of the three new turning lanes identified.

Option 3 - Install Traffic Control Signals (with existing lane configuration)

In this scenario, traffic control signals would be installed at the Highway 21/flea market driveway intersection. Even though it is typical MTO practice to provide exclusive left-turn lanes at signalized intersections, this option assumes no additional construction — i.e., the intersection geometry would remain as existing, with no exclusive turning lanes provided. This solution would significantly increase capacity and improve operating conditions compared to Options 1 and 2 for traffic exiting the flea market, as these vehicles would no longer need to wait for a suitable gap in northbound and southbound traffic. Option 3 would decrease capacity on Highway 21, however, since southbound left turns would continue to block southbound through traffic, and also since traffic exiting the flea market would require green time to be taken from Highway 21. The southbound approach would be operating at capacity based on existing traffic volumes, not including any future background growth within Grand Bend or outside the study area, or any increase in traffic generated by the flea market. It is estimated that this option would cost approximately $80,000 for installation, plus annual maintenance costs of approximately $4,000.

Option 4 - Install Traffic Control Signals (with new turning lanes)

This scenario combines Options 2 and 3, and assumes the installation of traffic control signals plus the construction of an exclusive southbound left-turn lane, a northbound right-turn taper, and an exclusive westbound left-turn lane. This option allows southbound through traffic to bypass vehicles waiting to turn left into the flea market as in Option 2, but also provides protection for vehicles waiting to exit the flea market as in Option 3. As such, there would be sufficient capacity for all movements based on existing volumes, with room for growth in traffic both on Highway 21 and to/from the flea market. Costs for the two components in this option (widening and signalization) would likely be as listed individually in Options 2 and 3, for a total of approximately $415,000 in construction costs plus $4,000 per year in signal maintenance.

7.3 Summary

Four different traffic control and geometric alternatives were identified to address existing and future traffic congestion related to the Pinery Antique Flea Market, located on Highway 21 south of Grand Bend. ***Table 7.1*** below summarizes the four options, ranks from 1 to 4 (best to worst) the expected operations for both traffic on Highway 21 and traffic exiting the flea market, and shows the approximate costs (as calculated based on assumptions outlined above).

Table 7.1 - Flea Market Options

Option	Description	Operations		Approximate Cost	
		Highway 21 Through Traffic	Exiting Flea Market	Construction/ Installation	Annual Operating
1	"Do Nothing" (Police Control)	4*	3*	—	$3,500
2	Exclusive Turning Lanes	1	4	$338,000	—
3	Traffic Control Signal Only	3	2	$80,000	$4,000
4	Signal Plus Exclusive Turning Lanes	2	1	$415,000	$4,000

**Note: Ranked from 1 to 4 (1=best; 4=worst).*

8.0 CONCLUSIONS/RECOMMENDATIONS

This study investigated existing and future operations along the Highway 21 corridor in the former Village of Grand Bend for the section between the Highway 21/Main Street intersection and the south limit of the Connecting Link designation. The conclusions and recommendations of this study are summarized below.

8.1 Conclusions

Existing Operations

During the Friday PM peak hour and Saturday midday peak hour in summer months:

- the intersection of Highway 21 and Main Street currently operates at a poor level of service;
- the intersection of Highway 21 and Lake Road currently operates at good levels of service; and
- the intersection of Highway 21 and Lakeview Avenue currently operates at acceptable levels of service.

From a corridor operations perspective, there is a need for some additional northbound capacity on Highway 21 south of Lake Road (within the four-lane cross section).

Future Operations

Based on the anticipated growth that is planned for the Grand Bend area, analyses of future traffic operations indicate:

- The intersection of Highway 21 and Main Street will fail (operate at Level of Service 'F'). Significant delays will be experienced at this intersection if the existing intersection remains unchanged.
- The intersection of Highway 21 and Lake Road will operate at capacity during a Saturday midday peak hour.
- The intersection of Highway 21 and Lakeview Avenue will fail. Delays on the Lakeview Avenue approach will be excessive, given that very few gaps in traffic on Highway 21 will be available.

- Along the Highway 21 corridor, additional capacity will still be required for the northbound direction south of Lake Road (within the four-lane cross section). In the two-lane section of Highway 21, the southbound direction will be operating at capacity, while the northbound direction will be over capacity (i.e., requiring more capacity).

Of the future traffic problems, the one which has the most detrimental effect is the operation of the Highway 21/Main Street intersection. Therefore, four options to address future traffic problems (focussing mainly on the Highway 21/Main Street intersection) have been identified. They include:

- "Do Nothing" – status quo;
- adjusting the lane configuration within the existing four-lane pavement width;
- constructing major improvements to the Highway 21/Main Street intersection; and
- protecting for a future road connection to by-pass the existing Highway 21/Main Street intersection (Ring Road Concept).

The Pinery Antique Flea Market, located south of the connecting link, currently experiences operational problems on Sundays related to vehicles accessing and egressing the site. Currently, the Flea Market has paid duty OPP officers on site during peak times to control traffic. Four options were identified during this study for the municipality to consider in addressing the issue of traffic operations at the Flea Market driveway intersection with Highway 21. They include:

- "Do Nothing" – continue to have paid duty OPP officers control traffic;
- Construct exclusive turning lanes at the intersection;
- Install a traffic control signal at the intersection; and
- Install a traffic control signal and construct exclusive turning lanes at the intersection.

8.2 Recommendations

The former Village of Grand Bend needs to take action to be prepared for future increases in traffic volumes and demands on the road network resulting from anticipated future development in the area. Any future improvements to the road network need to recognize not only local operations in and around the village, but also the role that Highway 21 plays in moving commercial, recreational and other traffic throughout the Municipality of Lambton Shores and adjacent Ontario municipalities along the coast of Lake Huron.

To address anticipated future traffic problems in the study area, the Municipality of Lambton Shores needs to undertake a more comprehensive area-wide study that takes into account the travel characteristics and road network of the Municipality and area as a whole, and not just the connecting link portion of Highway 21 in Grand Bend. Such a study should be initiated as soon as possible, since the time to establish the needs of the larger area, acquire properties, and plan/design/construct new roadways is considerable.

In addressing the traffic issues associated with the Pinery Flea Market, the best option may be for the Flea Market to continue to use paid duty officers to control traffic in the short term. In the longer term, the Ministry of Transportation will eventually revisit this section of Highway 21 (south of the connecting link), either through a resurfacing or reconstruction project. At that time, the need for any exclusive turning lanes or additional traffic control at the Pinery Flea Market driveway can be identified and incorporated, where possible, into the MTO project.

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APPENDIX A
Level of Service Definitions

LEVEL OF SERVICE[1]

Level of Service (LOS) is defined as a qualitative measure describing operational conditions within a traffic stream, and their perception by motorists and/or passengers. This concept was introduced in the 1965 *Highway Capacity Manual* as a criteria for interrupted flow conditions. The 2000 *Highway Capacity Manual* changed the basis for measuring Level of Service at intersections to control delay[2].

Six Levels of Service are defined with LOS A representing the best operating conditions, and LOS F the worst (briefly described below). It should be noted that there is often significant variability in the amount of delay experienced by individual drivers.

LOS A: This Level of Service describes the highest quality of traffic flow and is referred to as free flow. The approach appears open, turning movements are easily made and drivers have freedom of operation. Control delay is less than 10 seconds/vehicle.

LOS B: This Level of Service is referred to as a stable flow. Drivers feel somewhat restricted and occasionally may have to wait to complete the minor movement. Control delay is 10-15 seconds/vehicle for unsignalized intersections and 10-20 seconds/vehicle for signalized intersections.

LOS C: At this level, the operation is stable. Drivers feel more restricted and may have to wait, with queues developing for short periods. Control delay is 15-25 seconds/vehicle at unsignalized intersections and 20-35 seconds/vehicle at signalized intersections.

LOS D: At this level, traffic is approaching unstable flow. The motorist experiences increasing restriction and instability of flow. There are substantial delays to approaching vehicles during short peaks within the peak period, but there are enough gaps to lower demand to permit occasional clearance of developing queues and prevent excessive back-ups. Control delay is 25-35 seconds/vehicle at unsignalized intersections and 35-55 seconds/vehicle at signalized intersections.

LOS E: At this level capacity occurs. Long queues of vehicles exist and delays to vehicles may extend. Control delay is 35-50 seconds/vehicle at unsignalized intersections and 55-80 seconds/vehicle at signalized intersections.

LOS F: At this Level of Service, the intersection has failed. Capacity of the intersection has been exceeded. Control delay exceeds 50 seconds/vehicle at unsignalized intersections and exceeds 80 seconds/vehicle at signalized intersections.

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1 *Transportation Research Board: Highway Capacity Manual 1965, 2000*

2 *Control delay is defined as the component of delay that results when a control signal causes a lane group to reduce speed or to stop; it is measured by comparison with the uncontrolled condition.*

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TRANSPORTATION SOLUTIONS LIMITED

Ontario Street Bridge, Grand Bend
Traffic Operations Study

Paradigm Transportation Solutions Limited

February 2016

Project Summary

Project Number
150860

February 2016

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Ontario Street Bridge, Grand Bend Traffic Operations Study

List of Revisions

Version	Date	Author	Description
1	December 2015	J Mallett	Draft for Comment
2	January 2016	J Mallett	Revised Draft
3	February 2016	J Mallett	For Submission

Signatures and Seals

Signature

Engineer's Seal

Disclaimer

Executive Summary

Content

Paradigm Transportation Solutions Limited (Paradigm) was retained to conduct this Traffic Operations Study for Ontario Street in Grand Bend. The intersection of Ontario Street (Highway 21) and Main Street, as well as the bridge that crosses Parkhill Creek on the south leg of the intersection, will be of particular interest and will be the subject of analysis of potential improvements.

Traffic along Ontario Street (Highway 21) in Grand Bend has become an issue, particularly for the northbound traffic during the peak summer periods. With the northbound direction only having one through lane (as opposed to two through lanes for southbound traffic), the signal at the intersection of Ontario Street and Main Street appears to be the limiting factor for traffic movement, as there are no signalized intersections north of this intersection.

The current width of the bridge on the south leg of the intersection on Ontario Street limits the number of lanes to four; two southbound through lanes, a northbound left-turn lane and a northbound share through/right-turn lane.

Improvement Options

There are a number of potential geometric improvements that can be made to Ontario Street to improvement traffic operations. Most of these options involve widening the bridge on the south leg of the intersection of Ontario Street and Main Street to a five lane cross section. These potential improvements are analysed in this chapter. They include:

- Separate northbound through and right-turn lanes, or
- An additional northbound through lane.

Conclusions

Based on the investigations carried out, it is concluded that:

- The current configuration of the signalized intersection at Ontario Street and Main Street is insufficient to handle the traffic volumes seen by summer weekend volumes, particularly on holiday weekends, with the 95th percentile queue being calculated as being over capacity for the northbound through movement.
- The widening of the bridge on Ontario Street across Parkhill Creek, on the south leg of the intersection with Main Street to provide an additional travel lane is forecast to provide the capacity to allow the northbound movement to operate under capacity.

- Option 1 (separate northbound through and right-turn lanes) provides less benefit to the through movement than Option 2, but still brings the 95th percentile volume under capacity and also does not require an additional receiving lane on the north side of the intersection. However, if the additional lane is not made long enough, queues may be longer than forecast as right-turning vehicles would not be able to remove themselves from the through lane.
- Option 2 (adding an additional northbound through lane) provides the most benefit of the options presented, but has the implications of needing an additional receiving lane on the north side of the intersection.

Recommendations

Based on the findings of the analyses, it is recommended that Option 2 be considered for further design and study, as it provides the most benefit of the options presented. Option 1 could be considered a backup design, if the addition of a second northbound receiving lane is not possible.

Contents

Appendices

Figures

Tables

1 Introduction

1.1 Study Scope

Paradigm Transportation Solutions Limited (Paradigm) was retained to conduct this Traffic Operations Study for Ontario Street in Grand Bend. The intersection of Ontario Street (Highway 21) and Main Street, as well as the bridge that crosses Parkhill Creek on the south leg of the intersection, will be of particular interest and will be the subject of analysis of potential improvements. The location of the study area is shown in **Figure 1.1**.

The scope of the study includes a determination and assessment of the current traffic and conditions in the study area, the potential traffic growth a near future horizon, identification of problem areas, development of potential improvements to the intersection (focussing on the potential widening of the Ontario Street bridge) and analyses of these improvements.

This Traffic Operations Study (TIS) includes an analysis of existing traffic conditions, traffic forecasts for a five-year horizon (2020), and analysis of potential remedial measures.

1.2 Background

Traffic along Ontario Street (Highway 21) in Grand Bend has become an issue, particularly for the northbound traffic during the peak summer periods. Anecdotally, weekend traffic would see northbound traffic operating in a stop and go manner starting at the south end of the town. With the northbound direction only having one through lane (as opposed to two through lanes for southbound traffic), the signal at the intersection of Ontario Street and Main Street appears to be the limiting factor for traffic movement, as there are no signalized intersections north of this intersection.

The current width of the bridge on the south leg of the intersection on Ontario Street limits the number of lanes to four; two southbound through lanes, a northbound left-turn lane and a northbound share through/right-turn lane.

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Study Area

2 Existing Conditions

This section documents current traffic conditions, operational deficiencies, and constraints experienced by the public travelling at the intersections within the study area. The operational deficiencies and constraints identified at this stage will be fundamental to the process of defining the required remedial measures.

2.1 Road Network

Ontario Street is Provincial Highway 21, which runs north-south along the east coast of Lake Huron. Within Grand Bend, Ontario Street has a four lane cross section: two southbound lanes, one northbound lane and one centre two-way-left-turn (TWLT) lane. The speed limit on Ontario Street within Grand Bend is 50 km/h.

The intersection of Ontario Street and Main Street is currently signalized. The lane configuration is shown in **Figure 2.1**. The intersection currently has separate left-turn lanes and protected left-turn phase arrows on the signal heads for each direction. The westbound movement has a separate right-turn lane. In addition to the separate left-turn lanes, the southbound direction has a through lane and a shared through/right-turn lane, while the northbound direction only has a single shared through/right-turn lane.

2.2 Traffic Volumes

Turning movement volumes for the intersection of Ontario Street and Main Street were counted by Paradigm in June 2015 for the weekday conditions, as well as the Saturday of the long weekends on May (Victoria Day) and July (Canada Day).

Figure 2.2 summarizes the 2015 peak hour traffic volumes.

2.3 Traffic Operations

Intersection level of service (LOS) is a recognized method of quantifying the average delay experienced by drivers at intersections. It is based on the delay experienced by individual vehicles executing the various movements. The delay is related to the number of vehicles desiring to make a particular movement, compared to the estimated capacity for that movement. The capacity is based on a number of criteria related to the opposing traffic flows and intersection geometry.

The highest possible rating is LOS A, under which the average total delay is equal or less than 10.0 seconds per vehicle. When the average delay exceeds 80 seconds for signalized intersections, 50 seconds for unsignalized intersections or when the volume to capacity ratio is greater than 1.0, the movement is classed as LOS F and remedial measures are usually implemented, if they are feasible. LOS E is usually used as a

guideline for the determination of road improvement needs on through lanes, while LOS F may be acceptable for left-turn movements at peak times, depending on delays.

The operations of intersections in the study area were evaluated with the existing turning movement volumes using Synchro 9. The current signal timings were not available at the time of the report, however it was the intent to optimize the signal timings for this analysis regardless of what the current timings were. Therefore, for this analysis, cycle lengths and phase timings were optimized within Synchro, including the left-turn phases that currently operate at the intersection. Actual signal timings that will be used in the field if improvements are made should be designed by the road authority responsible for the signal at this intersection.

The intersection analysis considered two separate measures of performance:

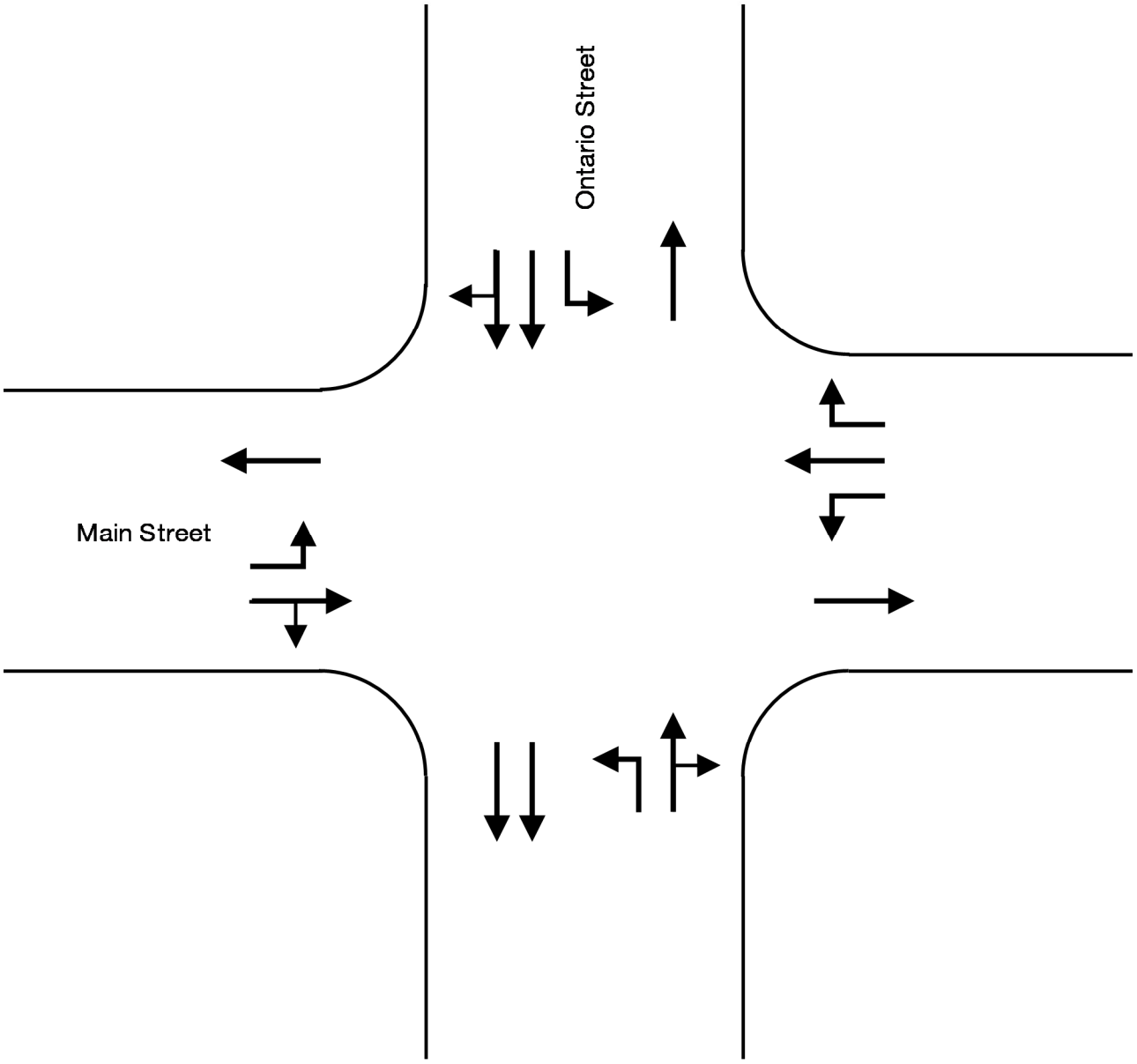
- The volume to capacity ratio for each intersection;
- The LOS for each turning movement. LOS is based on the average control delay per vehicle; and
- The 95th percentile queue.

The existing intersection operations are summarized in **Table 2.1**. The analysis confirms that there are operational issues at this intersection, particularly on the long weekend Saturdays. The 95th percentile queue lengths for the northbound movements were calculated to be 156 metres on the long weekend Saturday in May, which is greater than the distance to the next downstream intersection (River Road). Additionally, the Synchro results produce a note that the 95th percentile volume exceeds the capacity, so the queue may be longer. Anecdotally, queues are perceived to grow to much greater lengths, however these would not necessarily show up in the queue calculation for the intersection itself, as what is observed may be related to stop and go traffic that may be initiated by the signal at Ontario Street and Main Street. Also, the analysis indicates that much of the green time must be allocated to the north-south directions that the westbound left-turn movement is calculated to operate at LOS E and F at peak hours. Detailed Synchro reports are provided in **Appendix A**.

TABLE 2.1: 2015 PEAK HOUR TRAFFIC OPERATIONS

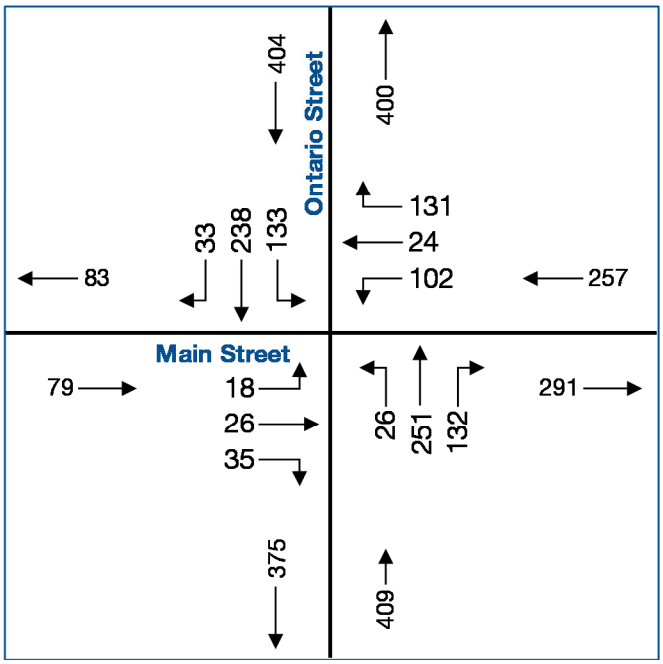
Intersection	Analysis Period	Control Type	MOE	Direction / Movement / Approach																
				Eastbound				Westbound				Northbound				Southbound				Overall
				Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	
Ontario Street & Main Street	Weekday AM Peak Hour	TSC	LOS	C	C	C	**C**	D	D	A	**C**	A	D	D	**C**	A	A	A	**A**	**C**
			Delay	23	20	20	**21**	29	30	8	**19**	9	25	25	**24**	7	9	9	**8**	**17**
			V/C	0.07	0.27	0.27		0.41	0.09	0.37		0.04	0.53	0.53		0.26	0.16	0.16		
			Q	8	15	15		27	11	14		4	95	95		15	20	20		
	Weekday PM Peak Hour	TSC	LOS	C	C	C	**C**	E	D	A	**D**	A	C	C	**C**	A	A	A	**A**	**C**
			Delay	24	20	20	**21**	38	31	10	**26**	6	17	17	**16**	7	10	10	**10**	**17**
			V/C	0.21	0.43	0.43		0.64	0.16	0.43		0.07	0.52	0.52		0.26	0.20	0.20		
			Q	15	22	22		39	16	16		6	74	74		16	26	26		
	Long Weekend Saturday Peak Hour (May)	TSC	LOS	C	C	C	**C**	E	D	A	**D**	A	E	E	**D**	B	C	C	**C**	**C**
			Delay	23	21	21	**21**	49	32	10	**29**	9	37	37	**32**	13	15	15	**15**	**24**
			V/C	0.25	0.58	0.58		0.79	0.27	0.56		0.27	0.87	0.87		0.52	0.41	0.41		
			Q	19	30	30		49	24	19		18	156	156		27	49	49		
	Long Weekend Saturday Peak Hour (July)	TSC	LOS	C	C	C	**C**	F	D	B	**D**	B	D	D	**D**	B	C	C	**C**	**D**
			Delay	24	23	23	**24**	64	34	11	**33**	13	30	30	**26**	13	19	19	**18**	**25**
			V/C	0.42	0.65	0.65		0.91	0.50	0.66		0.43	0.78	0.78		0.41	0.62	0.62		
			Q	24	36	36		54	43	22		23	136	136		22	73	73		

MOE - Measure of Effectiveness
LOS - Level of Service
Delay - Average Delay per Vehicle in Seconds
Q - 95th Percentile Queue Length
TCS - Traffic Control Signal
TWSC - Two-Way Stop Control
AWSC - All-Way Stop Control
RBT - Roundabout

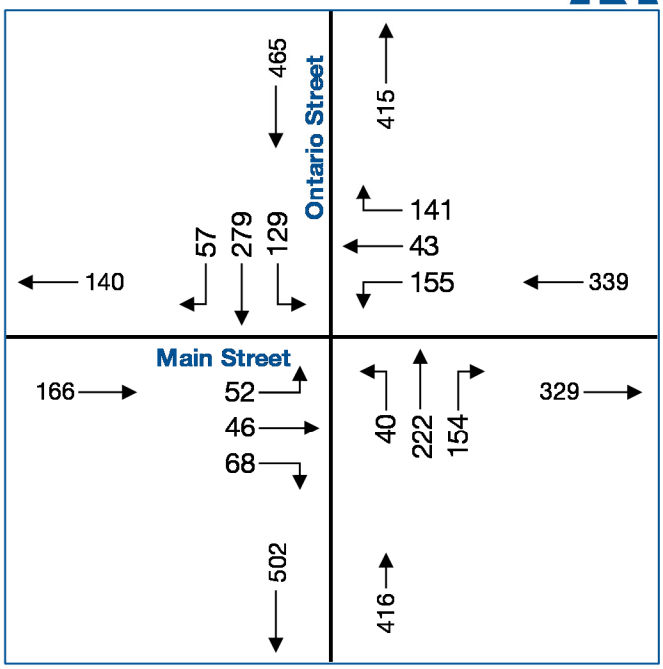


Ontario Street and Main Street Lane Configuration

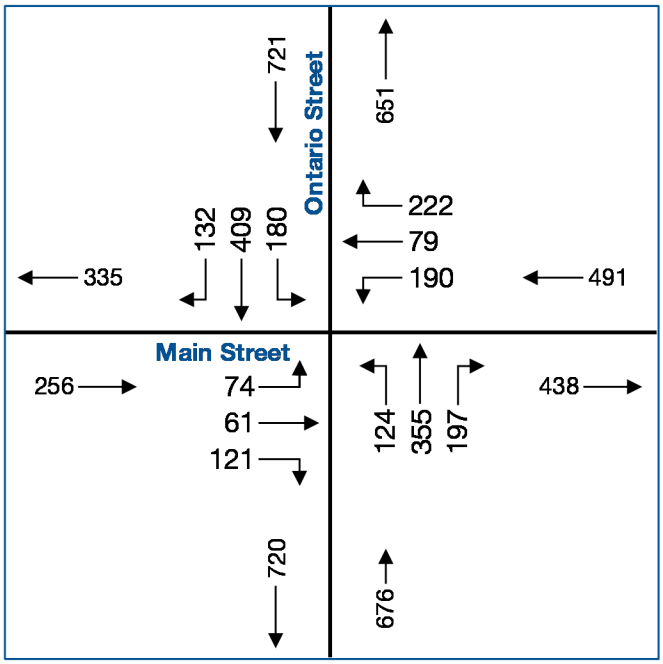
June Weekday
AM Peak Hour



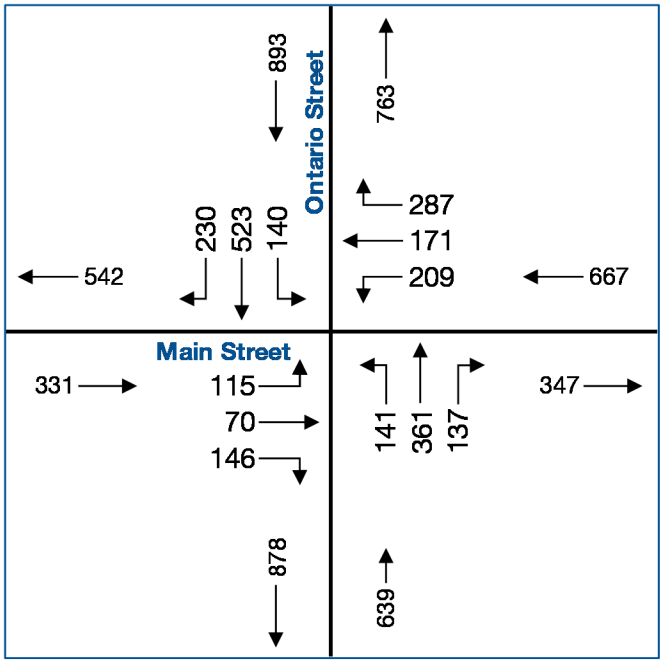
June Weekday
PM Peak Hour



May Long Weekend
Saturday Peak Hour



July Long Weekend
Saturday Peak Hour



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2015 Peak Hour Volumes

3 Future Traffic Forecasts

To assess the future needs of Ontario Street, traffic growth was forecast to a five-year horizon (2020). The historical volumes on Highway 21 in Grand Bend were used to calculate the growth rate.

The Ministry of Transportation has made available historical volume data for the years 1988 to 2010. The data used for this study are the AADT (Annual Average Daily Traffic) and SADT (Sumer Average Daily Traffic). As the counts were performed in the summer months (treating the May long weekend as a summer-like condition), the SADT was used to calculate a growth rate to apply to the 2015 volumes.

Figure 3.1 shows the SADT values for the years 1988 to 2010. To determine the growth trend for these volumes, a trend line was added using Microsoft Excel. The trend line produced had a slope of 27.52, which represents the average expected growth of vehicles. When compared to the actual SADT volumes between 1998 and 2010, this represents a growth rate of between 0.25% and 0.30% per year. Based on this, the growth rate used in this study is 0.3%. **Figure 3.2** shows the 2020 volumes after the application of the growth rate to the 2015 volumes.

3.1 2020 Traffic Operations

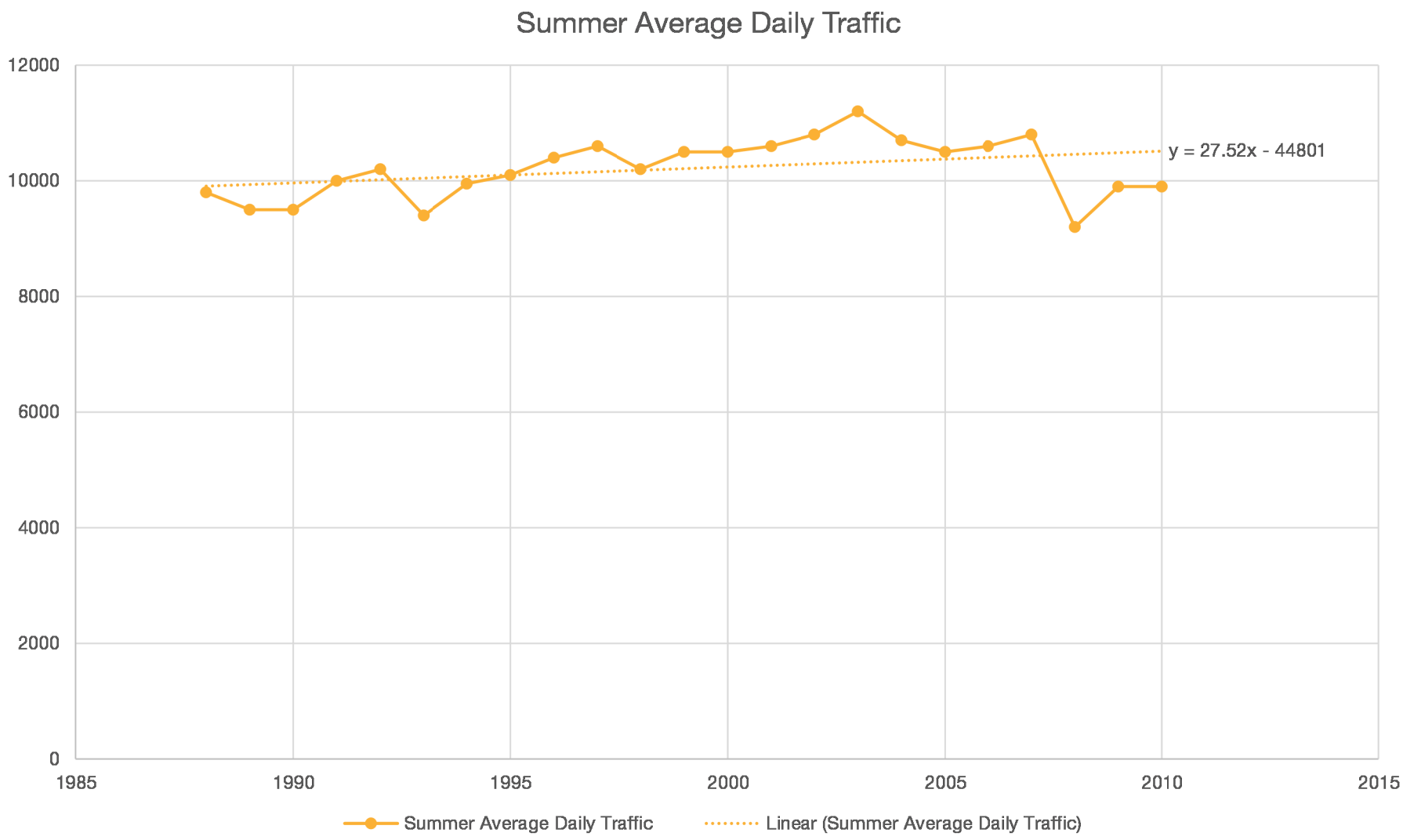
Based on the 2020 traffic volume forecasts, operations analyses have been conducted using Synchro 9 to determine the peak hour conditions for the intersections within the study area. No changes to the signal timings used for the existing conditions were made in this analysis.

As the 2020 volume forecasts were only slightly larger than the 2015 volumes, the results were mostly the same. **Table 3.1** summarizes the operation analyses and the detailed Synchro reports can be found in **Appendix B**.

TABLE 3.1: 2020 OPERATIONS SUMMARY

Intersection	Analysis Period	Control Type	MOE	Direction / Movement / Approach																
				Eastbound				Westbound				Northbound				Southbound				Overall
				Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	
Ontario Street & Main Street	Weekday AM Peak Hour	TSC	LOS	C	C	C	**C**	D	D	A	**C**	A	C	C	**C**	A	A	A	**A**	**C**
			Delay	23	20	20	**20**	29	30	9	**19**	6	18	18	**17**	7	9	9	**8**	**15**
			V/C	0.07	0.27	0.27		0.41	0.09	0.37		0.04	0.54	0.54		0.26	0.16	0.16		
			Q	7	15	15		27	11	15		4	77	77		19	21	21		
	Weekday PM Peak Hour	TSC	LOS	C	C	C	**C**	E	D	A	**D**	A	C	C	**C**	A	A	A	**A**	**C**
			Delay	24	20	20	**21**	39	32	10	**26**	6	17	17	**16**	7	10	10	**10**	**17**
			V/C	0.21	0.44	0.44		0.66	0.16	0.44		0.07	0.53	0.53		0.26	0.21	0.21		
			Q	16	23	23		41	16	16		7	76	76		16	26	26		
	Long Weekend Saturday Peak Hour (May)	TSC	LOS	C	C	C	**C**	F	D	A	**D**	A	E	E	**E**	C	C	C	**C**	**D**
			Delay	23	21	21	**22**	50	32	10	**29**	9	41	41	**35**	15	15	15	**15**	**25**
			V/C	0.26	0.59	0.59		0.81	0.27	0.56		0.28	0.90	0.90		0.54	0.42	0.42		
			Q	19	30	30		51	24	19		18	159	159		33	50	50		
	Long Weekend Saturday Peak Hour (July)	TSC	LOS	D	C	C	**C**	F	D	B	**D**	B	D	D	**D**	B	C	C	**C**	**D**
			Delay	25	24	24	**24**	67	34	11	**34**	13	31	31	**27**	13	20	20	**19**	**26**
			V/C	0.42	0.66	0.66		0.92	0.51	0.66		0.45	0.79	0.79		0.43	0.63	0.63		
			Q	25	37	37		55	44	22		23	139	139		23	74	74		

MOE - Measure of Effectiveness
LOS - Level of Service
Delay - Average Delay per Vehicle in Seconds
Q - 95th Percentile Queue Length
TCS - Traffic Control Signal
TWSC - Two-Way Stop Control
AWSC - All-Way Stop Control
RBT - Roundabout



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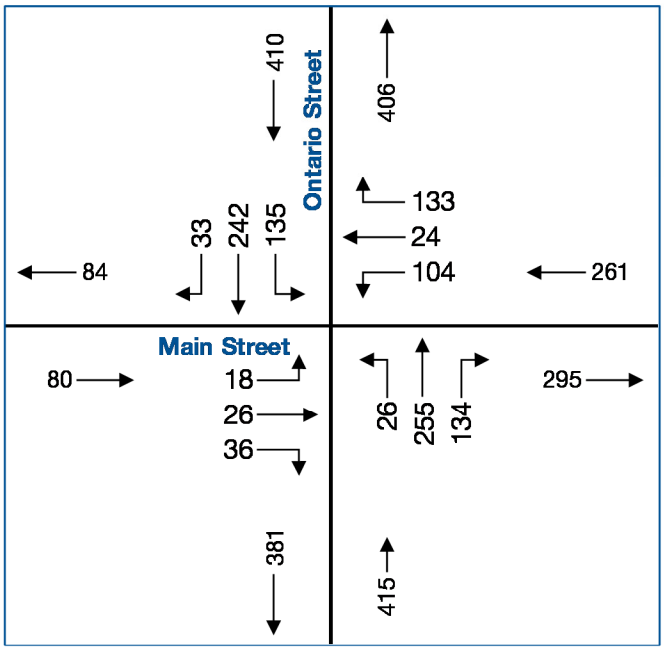
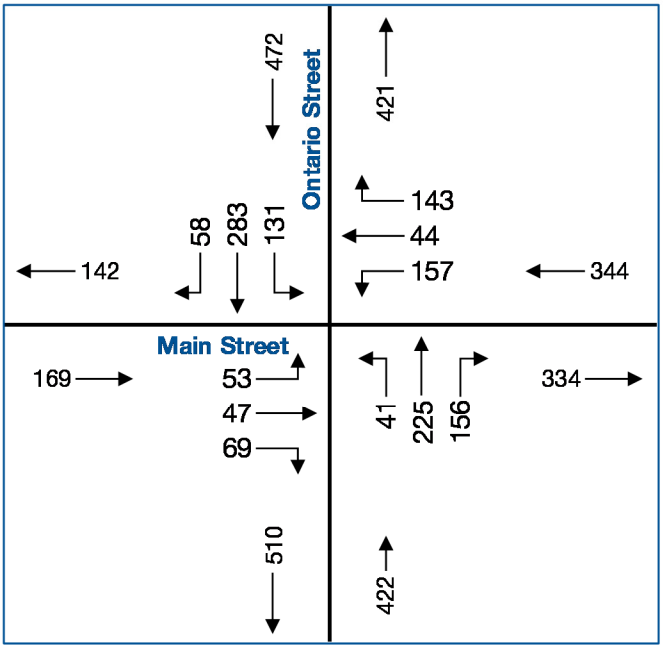
Highway 21, Grand Bend

SADT 1988 - 2010

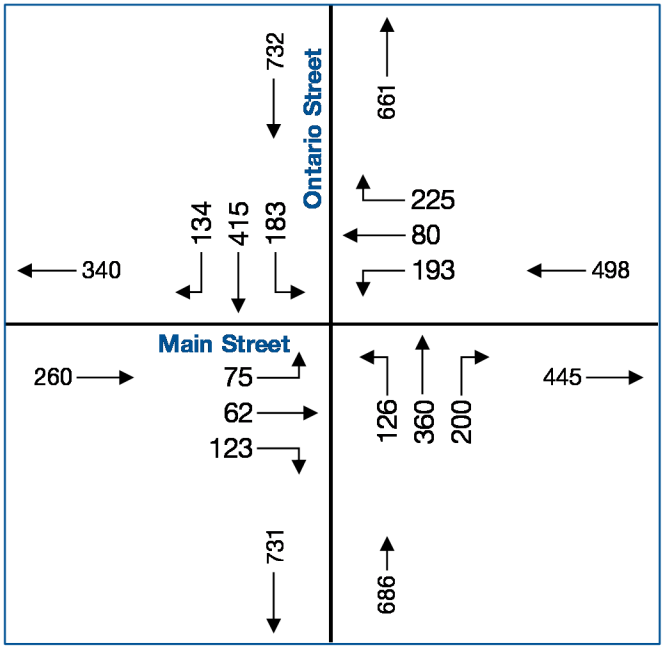
June Weekday
AM Peak Hour



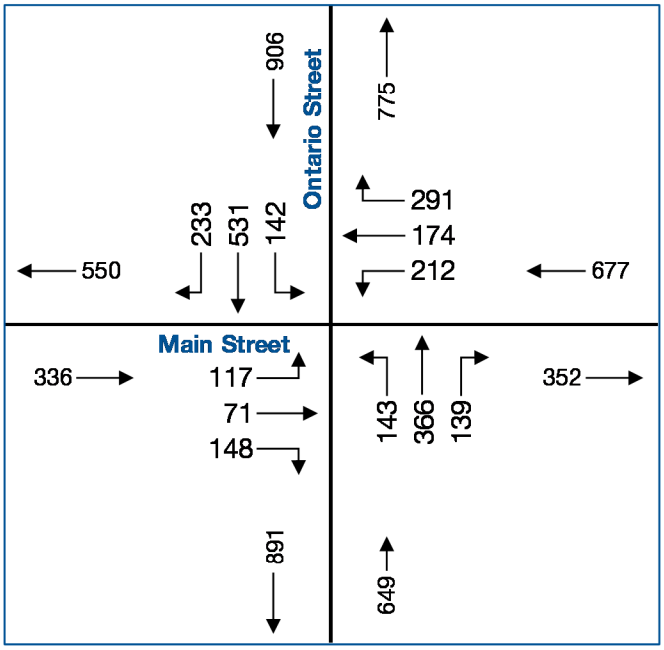
June Weekday
PM Peak Hour



May Long Weekend
Saturday Peak Hour



July Long Weekend
Saturday Peak Hour



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2020 Peak Hour Volume Forecasts

4 Potential Improvements

There are a number of potential geometric improvements that can be made to Ontario Street to improvement traffic operations. Most of these options involve widening the bridge on the south leg of the intersection of Ontario Street and Main Street to a five lane cross section. These potential improvements are analysed in this chapter. They include:

- Separate northbound through and right-turn lanes, or
- An additional northbound through lane.

4.1 Option 1: Exclusive Northbound Right-Turn Lane

The option is to widen the bridge on Ontario Street to be a five lane cross-section and have the additional lane be used to separate the through and right-turn movements. The lane configuration for the northbound direction would then be a left-turn lane, a through lane, and a right-turn lane, shown in **Figure 4.1**. The benefit of having the separate right-turn lane would be that the right-turn movements would remove themselves from the queue of the through movement and be able to make use of right-turn-on-red movements, thus increasing capacity for both movements. The other benefit, compared to Option 2 discussed below, is that there would not be a need for an additional receiving lane on the north side of the intersection.

An analysis of this option was done with Synchro 9 to compare the relative effectiveness. The summary of the operations for Option 1 is shown in **Table 4.1**.

The separation of the northbound through and right-turn movements improves the operations for all movements. With less vehicles in the northbound through lane, the volume to capacity ratio is decreased, allowing less green time to be needed to adequately service the movement, therefore more green time is able to be given to other movements. In particular, the westbound left-turn movement, which was forecast to operate at LOS F during the long weekend Saturday conditions, is forecast to operate at LOS D with the addition of the northbound through lane, which is within acceptable levels for a signalized left-turn movement.

The 95th percentile queue for the northbound movement is forecast to be 85 metres during the May and July long weekend Saturday peak hours, which is improved from the current configuration. As well, there is no note given by the Synchro results that “the 95th percentile volume exceeds the capacity, so the queue may be longer”, which there was under the current lane configurations.

With the 95th percentile queues forecast to reach as high as 85 metres, the additional right-turn lane should be designed to be long enough that the queue from the through lane does not block right-turning vehicles from entering the right-turn lane. The effect of this would be that the queue in the

through lane would be greater than what is forecast, as the right-turning vehicles would add additional length to the queue as they wait for the opportunity to get into the right-turn lane.

TABLE 4.1: 2020 OPTION 1 OPERATIONS SUMMARY

Intersection	Analysis Period	Control Type	MOE	Direction / Movement / Approach																Overall
				Eastbound				Westbound				Northbound				Southbound				
				Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	
Ontario Street & Main Street	Weekday AM Peak Hour	TCS	LOS	C	C	C	**C**	C	C	A	**B**	A	B	A	**B**	A	A	A	**A**	**B**
			Delay	22	20	20	**21**	28	29	9	**18**	6	16	3	**11**	9	9	9	**9**	**13**
			V/C	0.07	0.28	0.28		0.40	0.08	0.36		0.05	0.35	0.19		0.22	0.17	0.17		
			Q	7	15	15		27	11	15		4	50	10		16	21	21		
	Weekday PM Peak Hour	TCS	LOS	C	C	C	**C**	C	C	A	**C**	A	B	A	**B**	A	B	B	**A**	**B**
			Delay	23	20	20	**21**	35	30	9	**24**	7	16	4	**11**	7	11	11	**10**	**15**
			V/C	0.21	0.44	0.44		0.61	0.15	0.42		0.07	0.31	0.22		0.21	0.21	0.21		
			Q	15	23	23		37	16	16		7	47	12		17	27	27		
	Long Weekend Saturday Peak Hour (May)	TCS	LOS	C	C	C	**C**	D	C	A	**C**	A	C	A	**B**	A	B	B	**B**	**B**
			Delay	23	21	21	**21**	42	31	9	**25**	9	23	4	**15**	10	16	16	**15**	**18**
			V/C	0.26	0.59	0.59		0.75	0.26	0.55		0.29	0.56	0.30		0.38	0.43	0.43		
			Q	19	30	30		46	24	19		18	85	14		26	52	52		
	Long Weekend Saturday Peak Hour (July)	TCS	LOS	C	C	C	**C**	D	C	A	**C**	B	C	A	**B**	B	C	C	**C**	**C**
			Delay	24	25	25	**25**	45	32	10	**26**	14	24	4	**18**	12	22	22	**20**	**22**
			V/C	0.43	0.68	0.68		0.82	0.47	0.64		0.46	0.59	0.23		0.33	0.67	0.67		
			Q	24	37	37		48	43	21		24	85	12		24	78	78		

MOE - Measure of Effectiveness
LOS - Level of Service
Delay - Average Delay per Vehicle in Seconds
Q - 95th Percentile Queue Length
TCS - Traffic Control Signal
TWSC - Two-Way Stop Control
AWSC - All-Way Stop Control
RBT - Roundabout

4.2 Option 2: Additional Northbound Through Lane

The second option is to widen the bridge on Ontario Street to be a five lane cross-section and have the additional lane be an exclusive northbound through lane. The lane configuration for the northbound direction would then be a left-turn lane, a through lane, and a shared through/right-turn lane. As the through movement is the highest of the northbound movements, designating the additional lane as a through lane would provide the most benefit in terms of additional capacity.

However, as two lanes would then be able to make a northbound through movement, an additional northbound receiving lane would have to be constructed on the north leg of the intersection. Currently, there is only a single receiving lane. This would require the north leg of the intersection to be widened, which would likely have to happen on the east side of the road (in front of the Bank of Montreal building), as the west side does not appear to have much additional room for expansion (parking for the Colonial Hotel is directly adjacent to the sidewalk). A preliminary field check suggests that there is enough land on the east side of Ontario Street for widening until Municipal Drive, which is approximately 55 metres north of Main Street. At Municipal Street, there are two options to bring the width of Ontario Street back to the existing width: the additional through lane would need to be dropped, or the centre two-way-left-turn lane would need to be converted to a northbound through lane. The former option would need to have an appropriate taper to merge the two through lanes into one. **Figure 4.2** shows a concept plan that shows this. The latter option would need to convert the centre northbound lane back to a two-way-left-turn lane at some point. A reasonable opportunity to do this would be to make the inside northbound through lane become an exclusive left-turn lane at the next available intersection, which is Oak Street.

An analysis of this option was done with Synchro 9 to compare the relative effectiveness. The summary of the operations for Option 2 is shown in **Table 4.1**.

The addition of a northbound through lane improves the operations for all movements, as in Option 1. The northbound through movement is improved by the additional capacity to the movement, but as capacity for that movement is increased, less green time is needed to adequately service the movement, therefore more green time is able to be given to other movements. Like in Option 1, the westbound left-turn movement, which was forecast to operate at LOS F during the long weekend Saturday conditions, is forecast to operate at LOS C and D with the addition of the northbound through lane, which is within acceptable levels for a signalized left-turn movement.

The 95th percentile queue for the northbound movement is forecast to be 55 metres during the May long weekend Saturday peak hour and 50 metres during the July long weekend Saturday peak hour, which is much improved from the current configuration. As well, there is no note given by the

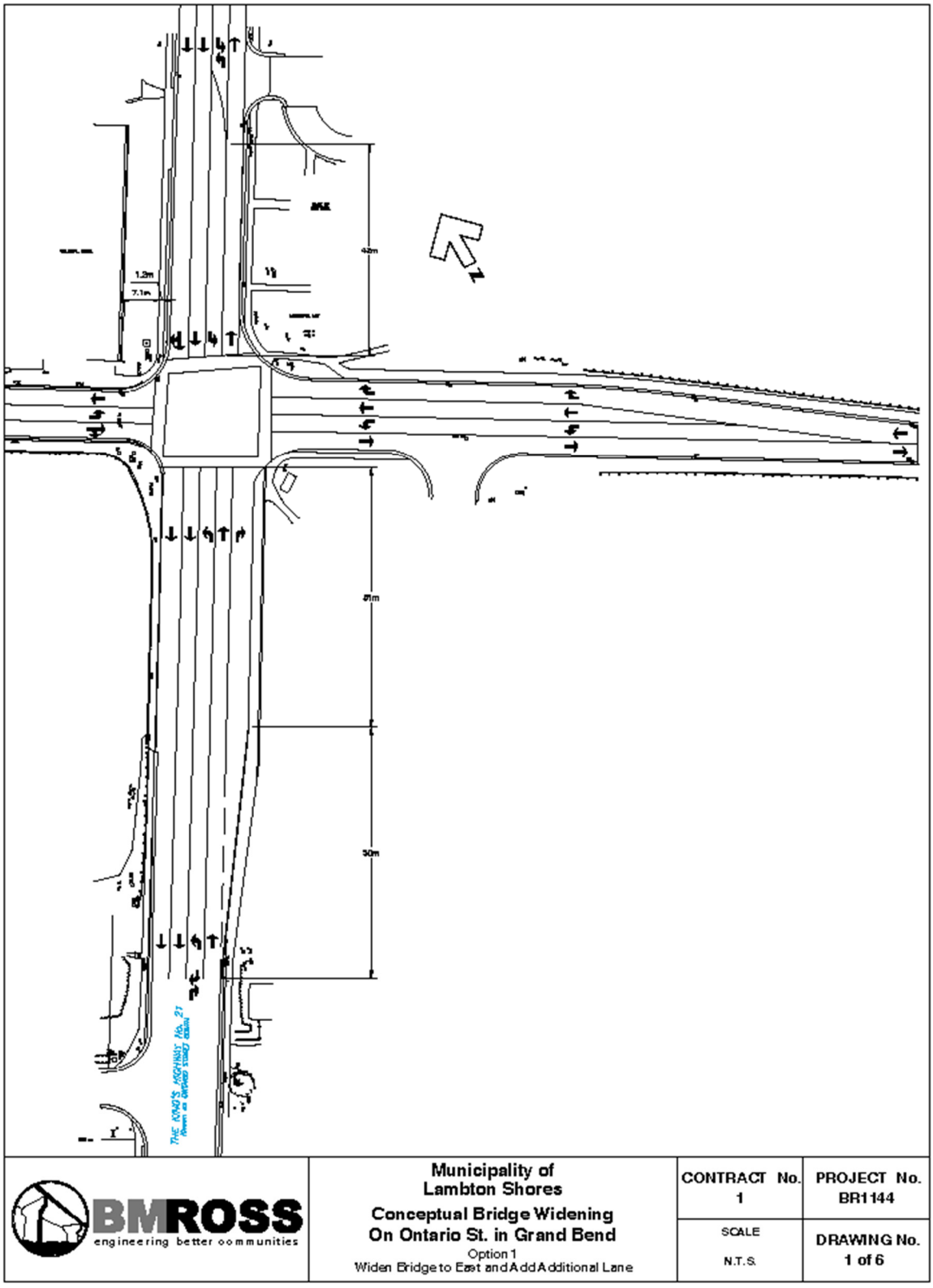
Synchro results that "the 95th percentile volume exceeds the capacity, so the queue may be longer", which there was under the current lane configurations.

The Synchro analysis assumes a balanced use of both through lanes in its calculations (ie. the usage of each lane is balanced so that the same volume uses both lanes). In practice, there is potential for an unbalanced use of the through lanes as regular drivers will have knowledge of the lane additional northbound through lane being dropped just north of the intersection. Instead of having to merge, some drivers may opt to always use the inside lane. This imbalance would have the effect of the queues being longer than what is reported in the Synchro analysis. The worst case scenario for this unbalanced lane utilization would resemble the lane configuration for Option 1. This worst case scenario would only occur if all northbound through vehicles decided to use the left-hand side lane to make their movement, which is not likely. Therefore, the actual operation of this scenario would fall somewhere in between the balanced lane utilization of Option 1 and the operations of Option 2.

TABLE 4.2: 2020 OPTION 2 OPERATIONS SUMMARY

Intersection	Analysis Period	Control Type	MOE	Direction / Movement / Approach																
				Eastbound				Westbound				Northbound				Southbound				Overall
				Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	Left	Through	Right	Approach	
Ontario Street & Main Street	Weekday AM Peak Hour	TCS	LOS	C	C	C	**C**	C	C	A	**B**	A	A	A	**A**	A	A	A	**A**	**B**
			Delay	22	20	20	**21**	28	29	8	**18**	6	10	10	**10**	7	9	9	**9**	**12**
			V/C	0.07	0.28	0.28		0.40	0.08	0.36		0.05	0.28	0.28		0.23	0.17	0.17		
			Q	7	15	15		27	11	15		4	26	26		16	21	21		
	Weekday PM Peak Hour	TCS	LOS	C	C	C	**C**	C	C	A	**C**	A	A	A	**A**	A	B	B	**B**	**B**
			Delay	23	20	20	**21**	31	29	9	**22**	7	10	10	**9**	8	12	12	**11**	**14**
			V/C	0.21	0.44	0.44		0.56	0.15	0.40		0.08	0.28	0.28		0.24	0.21	0.21		
			Q	15	23	23		37	16	15		7	25	25		18	28	28		
	Long Weekend Saturday Peak Hour (May)	TCS	LOS	C	C	C	**C**	C	C	A	**B**	B	B	B	**B**	B	B	B	**B**	**B**
			Delay	21	21	21	**21**	28	28	7	**19**	11	18	18	**17**	12	19	19	**17**	**18**
			V/C	0.25	0.59	0.59		0.61	0.21	0.49		0.31	0.51	0.51		0.43	0.47	0.47		
			Q	17	30	30		39	23	17		20	55	55		29	56	56		
	Long Weekend Saturday Peak Hour (July)	TCS	LOS	C	C	C	**C**	D	C	A	**C**	B	B	B	**B**	B	C	C	**C**	**C**
			Delay	24	25	25	**25**	45	32	10	**26**	15	18	18	**17**	11	22	22	**20**	**22**
			V/C	0.43	0.68	0.68		0.82	0.47	0.64		0.46	0.44	0.44		0.33	0.67	0.67		
			Q	24	37	37		48	43	21		24	50	50		24	78	78		

MOE - Measure of Effectiveness
LOS - Level of Service
Delay - Average Delay per Vehicle in Seconds
Q - 95th Percentile Queue Length
TCS - Traffic Control Signal
TWSC - Two-Way Stop Control
AWSC - All-Way Stop Control
RBT - Roundabout



BMROSS engineering better communities	Municipality of Lambton Shores Conceptual Bridge Widening On Ontario St. in Grand Bend Option 1 Widen Bridge to East and Add Additional Lane	CONTRACT No. 1	PROJECT No. BR1144
		SCALE N.T.S.	DRAWING No. 1 of 6

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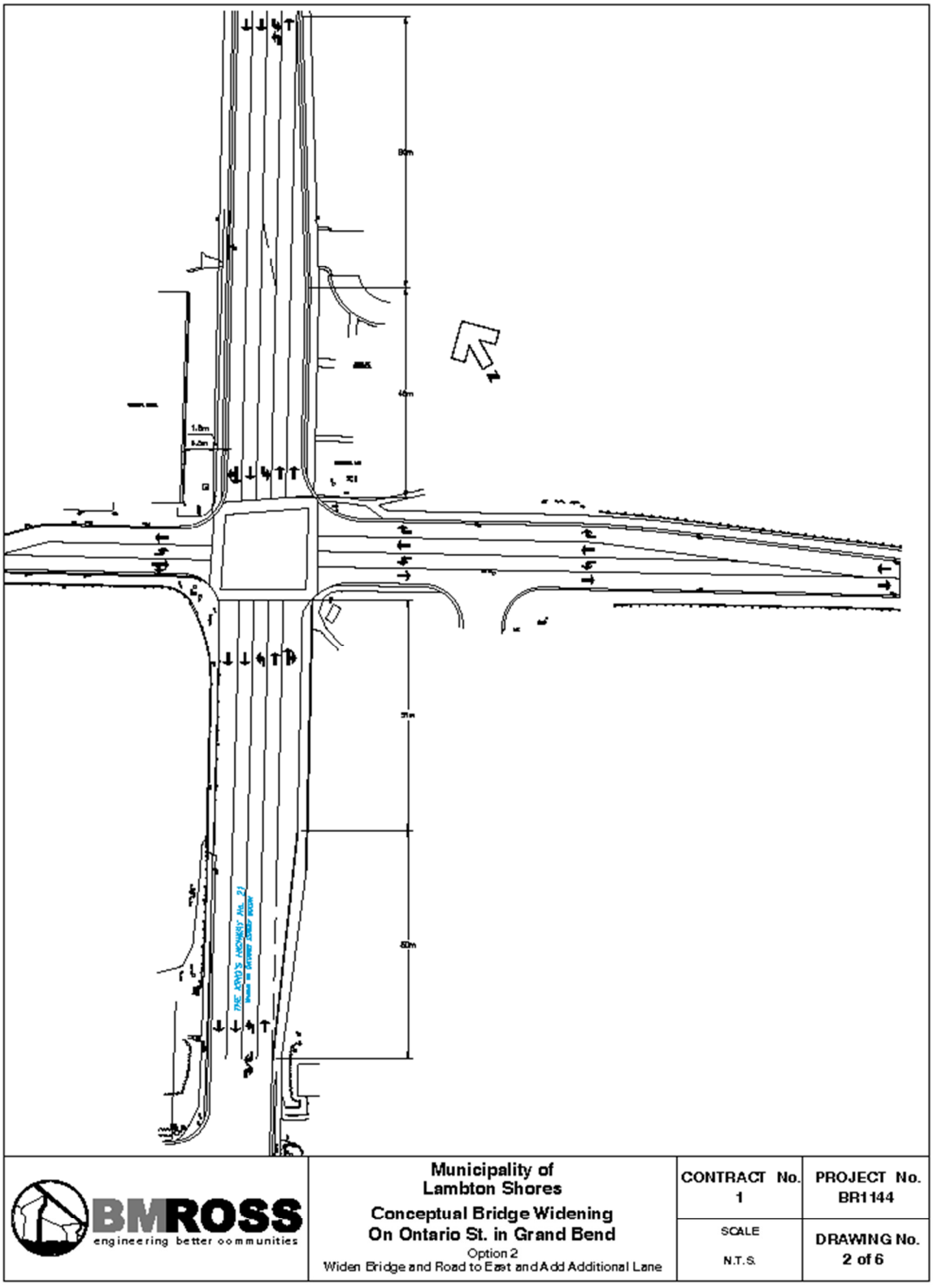
Option 1 Lane Configuration Concept



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Option 2 Lane Configuration Concept

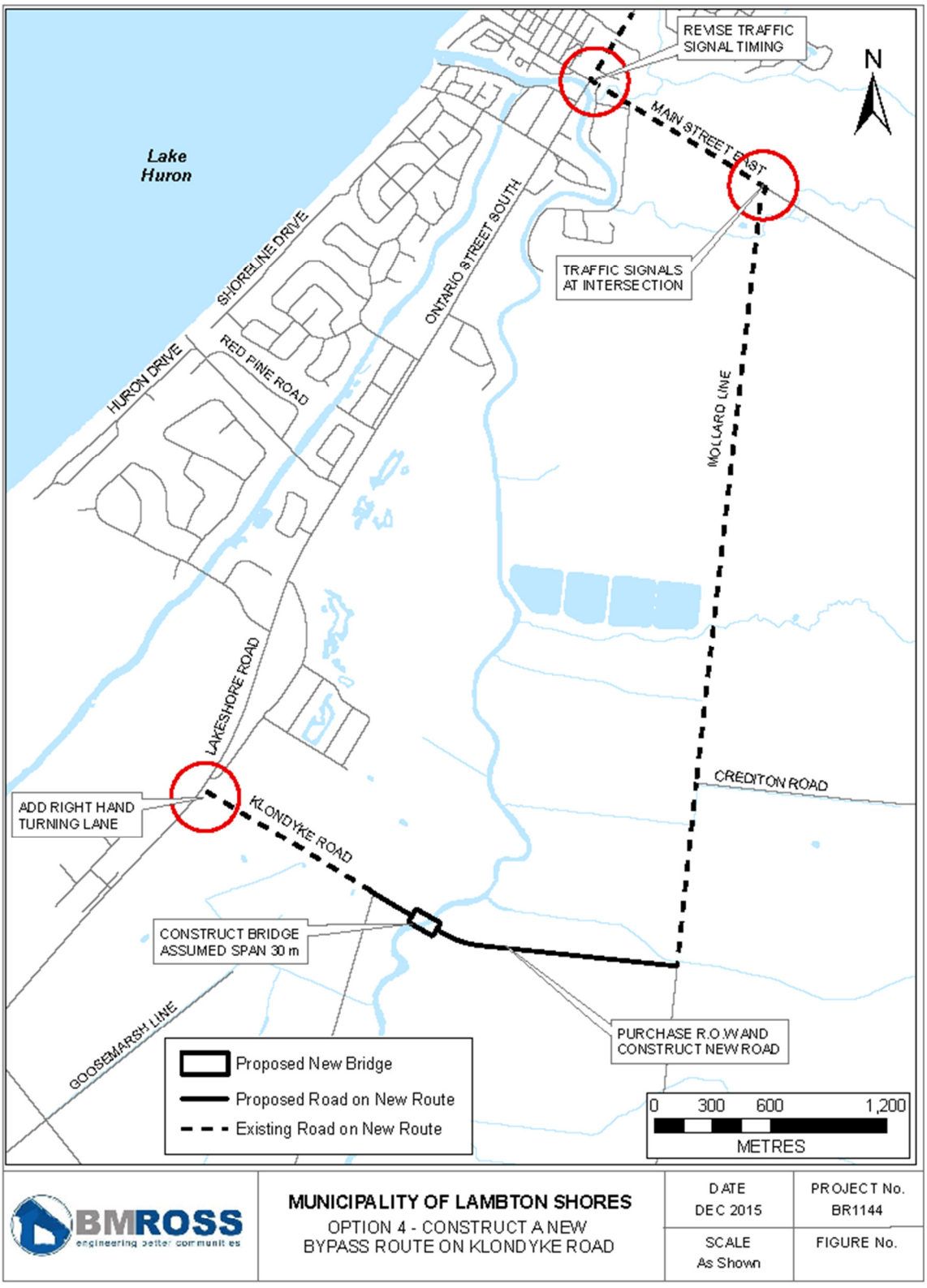
5 Highway 21 Grand Bend Bypass

As part of a higher level planning proposal, a concept for a bypass route around Grand Bend has been created. **Figure 5.1** shows the preliminary route for this bypass. The bypass would use Klondyke Road, cross a new bridge to Mollard Line, then back to Ontario Street by way of Main Street.

If all traffic travelling through Grand Bend (ie. not destined for or originating from Grand Bend) were to use this bypass, the impact would be to reducing the through traffic on Ontario Street itself, and therefore reducing the impact of the stop and go nature of the traffic, particularly for the northbound direction in the south end of the town. However, there are two major drawbacks to this bypass concept:

- The bypass itself would measure approximately 7.8 km, compared to approximate 4.2 km between Klondyke Road and Main Street along Ontario Street. Even with the current delays, a trip that is nearly double the length might not have any travel time savings. As well, unless Highway 21 was reconfigured at Klondyke Road to make the bypass the “through” movement and Ontario Street the “left-turn” movement, it might be difficult to get drivers to willingly turn onto Klondyke Road to use the bypass, particularly if there is no indication of traffic delays that far south.
- The traffic will still have to pass through the signalized intersection of Ontario Street and Main Street and therefore will not reduce the total number of vehicles using the intersection. The impact of the vehicles using the bypass now being westbound right-turns would be that more green time would be required for the east-west movements, therefore reducing the green time (and capacity) for the north-south movements.

With the issues noted above, along with the assumed additional cost of construction, it is not expected that this bypass as proposed would provide adequate benefit to be recommended.



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Highway 21 Bypass Concept

6 Conclusions and Recommendations

6.1 Conclusions

Based on the investigations carried out, it is concluded that:

- The current configuration of the signalized intersection at Ontario Street and Main Street is insufficient to handle the traffic volumes seen by summer weekend volumes, particularly on holiday weekends, with the 95th percentile queue being calculated as being over capacity for the northbound through movement.
- The widening of the bridge on Ontario Street across Parkhill Creek, on the south leg of the intersection with Main Street to provide an additional travel lane is forecast to provide the capacity to allow the northbound movement to operate under capacity.
- Option 1 (separate northbound through and right-turn lanes) provides less benefit to the through movement than Option 2, but still brings the 95th percentile volume under capacity and also does not require an additional receiving lane on the north side of the intersection. However, if the additional lane is not made long enough, queues may be longer than forecast as right-turning vehicles would not be able to remove themselves from the through lane.
- Option 2 (adding an additional northbound through lane) provides the most benefit of the options presented, but has the implications of needing an additional receiving lane on the north side of the intersection.

6.2 Recommendations

Based on the findings of the analyses, it is recommended that Option 2 be considered for further design and study, as it provides the most benefit of the options presented. Option 1 could be considered a backup design, if the addition of a second northbound receiving lane is not possible.

Appendix A

2015 Traffic Operations Reports

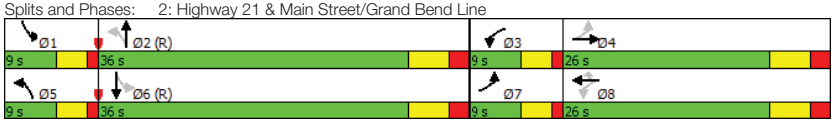
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	18	26	35	102	24	131	26	251	132	133	238	33
Future Volume (vph)	18	26	35	102	24	131	26	251	132	133	238	33
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.99					0.97	0.99	0.99		0.99	1.00	
Frt		0.914				0.850		0.948			0.982	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1626	1679	0	1656	1759	1583	1671	1595	0	1770	3093	0
Flt Permitted	0.740			0.610			0.572			0.375		
Satd. Flow (perm)	1252	1679	0	1063	1759	1533	993	1595	0	694	3093	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		38				150		38			22	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	6					6	10		7	7		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	11%	4%	3%	9%	8%	2%	8%	14%	7%	2%	16%	0%
Adj. Flow (vph)	20	28	38	111	26	142	28	273	143	145	259	36
Shared Lane Traffic (%)												
Lane Group Flow (vph)	20	66	0	111	26	142	28	416	0	145	295	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	36.0		9.0	36.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	11.3%	45.0%		11.3%	45.0%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	16.1	10.1		17.7	13.7	13.7	46.6	38.6		52.3	46.4	
Actuated g/C Ratio	0.20	0.13		0.22	0.17	0.17	0.58	0.48		0.65	0.58	
v/c Ratio	0.07	0.27		0.41	0.09	0.37	0.04	0.53		0.26	0.16	
Control Delay	22.7	20.0		29.2	30.1	8.4	6.0	17.1		6.9	9.0	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	22.7	20.0		29.2	30.1	8.4	6.0	17.1		6.9	9.0	
LOS	C	C		C	C	A	A	B		A	A	
Approach Delay		20.7			18.7			16.4			8.3	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			B			B			A	
Queue Length 50th (m)	2.4	4.1		14.0	3.2	0.0	1.5	41.4		8.1	8.7	
Queue Length 95th (m)	7.5	15.3		27.1	10.9	14.4	4.3	74.4		15.3	20.4	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	276	448		272	439	495	629	789		561	1804	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.07	0.15		0.41	0.06	0.29	0.04	0.53		0.26	0.16	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.53
Intersection Signal Delay: 14.4 Intersection LOS: B
Intersection Capacity Utilization 55.3% ICU Level of Service B
Analysis Period (min) 15

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



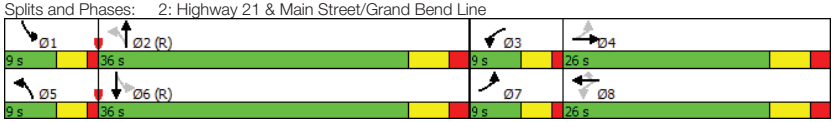
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	52	46	68	155	43	141	40	222	154	129	279	57
Future Volume (vph)	52	46	68	155	43	141	40	222	154	129	279	57
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.97	0.98		0.99		0.95	0.99	0.99		0.99	0.99	
Frt		0.910				0.850		0.939			0.975	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1702	0	1752	1900	1524	1719	1599	0	1719	3253	0
Flt Permitted	0.726			0.490			0.534			0.385		
Satd. Flow (perm)	1317	1702	0	893	1900	1451	955	1599	0	693	3253	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		74				153		50			34	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	14		12	12		14	10		6	6		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	57	50	74	168	47	153	43	241	167	140	303	62
Shared Lane Traffic (%)												
Lane Group Flow (vph)	57	124	0	168	47	153	43	408	0	140	365	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	36.0		9.0	36.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	11.3%	45.0%		11.3%	45.0%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	15.1	10.5		17.3	12.3	12.3	46.5	38.2		51.4	43.9	
Actuated g/C Ratio	0.19	0.13		0.22	0.15	0.15	0.58	0.48		0.64	0.55	
v/c Ratio	0.21	0.43		0.64	0.16	0.43	0.07	0.52		0.26	0.20	
Control Delay	23.9	19.8		38.3	31.4	10.0	6.4	16.8		7.2	10.3	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	23.9	19.8		38.3	31.4	10.0	6.4	16.8		7.2	10.3	
LOS	C	B		D	C	A	A	B		A	B	
Approach Delay		21.1			25.7			15.8			9.5	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			B			A	
Queue Length 50th (m)	6.9	7.3		21.9	6.9	0.0	2.3	38.7		7.8	15.2	
Queue Length 95th (m)	15.3	22.2		#39.4	16.1	15.8	6.3	74.0		16.0	25.8	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	276	481		261	475	477	615	789		549	1799	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.21	0.26		0.64	0.10	0.32	0.07	0.52		0.26	0.20	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.64
Intersection Signal Delay: 16.7 Intersection LOS: B
Intersection Capacity Utilization 57.6% ICU Level of Service B
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



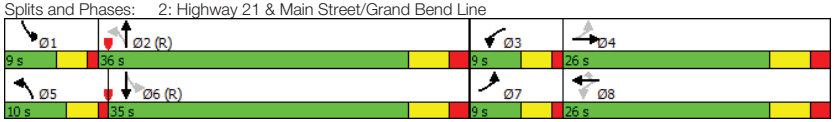
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	74	61	121	190	79	222	124	355	197	180	409	132
Future Volume (vph)	74	61	121	190	79	222	124	355	197	180	409	132
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.93	0.97		0.98		0.91	0.97	0.99		1.00	0.98	
Frt		0.900				0.850		0.946			0.964	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1659	0	1752	1900	1524	1719	1604	0	1719	3182	0
Flt Permitted	0.701			0.473			0.415			0.183		
Satd. Flow (perm)	1218	1659	0	851	1900	1380	729	1604	0	330	3182	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		120				241		40			61	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	38		28	28		38	39		10	10		39
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	80	66	132	207	86	241	135	386	214	196	445	143
Shared Lane Traffic (%)												
Lane Group Flow (vph)	80	198	0	207	86	241	135	600	0	196	588	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	10.0	36.0		9.0	35.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	12.5%	45.0%		11.3%	43.8%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	18.5	11.5		19.3	13.3	13.3	43.3	33.1		47.8	35.4	
Actuated g/C Ratio	0.23	0.14		0.24	0.17	0.17	0.54	0.41		0.60	0.44	
v/c Ratio	0.25	0.58		0.79	0.27	0.56	0.27	0.87		0.52	0.41	
Control Delay	23.3	20.6		48.6	32.3	9.9	8.6	37.3		13.4	15.3	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	23.3	20.6		48.6	32.3	9.9	8.6	37.3		13.4	15.3	
LOS	C	C		D	C	A	A	D		B	B	
Approach Delay		21.4			28.5			32.0			14.8	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			C			B	
Queue Length 50th (m)	9.9	11.6		27.7	12.8	0.0	7.5	79.2		11.3	27.4	
Queue Length 95th (m)	18.7	29.9		#49.2	24.1	18.8	17.5	#155.7		26.9	48.6	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	316	504		261	475	525	494	687		377	1442	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.25	0.39		0.79	0.18	0.46	0.27	0.87		0.52	0.41	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 80
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.87
Intersection Signal Delay: 24.1 Intersection LOS: C
Intersection Capacity Utilization 83.2% ICU Level of Service E
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



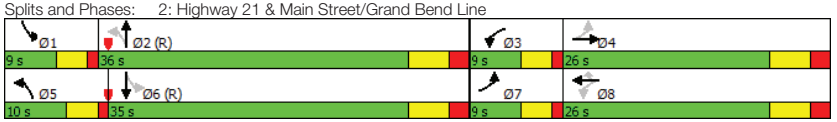
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	115	70	146	209	171	287	141	361	137	140	523	230
Future Volume (vph)	115	70	146	209	171	287	141	361	137	140	523	230
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.83	0.86		0.87		0.76	0.95	0.98		0.98	0.93	
Frt		0.899				0.850		0.959			0.954	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1475	0	1752	1900	1524	1719	1608	0	1719	3006	0
Flt Permitted	0.622			0.423			0.248			0.266		
Satd. Flow (perm)	959	1475	0	680	1900	1153	426	1608	0	474	3006	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		126				312		27			97	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	114		161	161		114	111		26	26		111
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	125	76	159	227	186	312	153	392	149	152	568	250
Shared Lane Traffic (%)												
Lane Group Flow (vph)	125	235	0	227	186	312	153	541	0	152	818	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	10.0	36.0		9.0	35.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	12.5%	45.0%		11.3%	43.8%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	20.8	13.8		21.6	15.6	15.6	43.4	33.7		42.9	33.5	
Actuated g/C Ratio	0.26	0.17		0.27	0.20	0.20	0.54	0.42		0.54	0.42	
v/c Ratio	0.42	0.65		0.91	0.50	0.66	0.43	0.78		0.41	0.62	
Control Delay	24.4	23.0		63.9	33.8	10.7	12.7	30.1		12.8	19.3	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	24.4	23.0		63.9	33.8	10.7	12.7	30.1		12.8	19.3	
LOS	C	C		E	C	B	B	C		B	B	
Approach Delay		23.5			33.3			26.3			18.3	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			C			B	
Queue Length 50th (m)	14.9	15.9		28.9	27.9	0.0	9.7	69.5		9.7	46.2	
Queue Length 95th (m)	24.4	35.8		#53.6	43.1	21.8	22.5	#135.8		22.4	72.6	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	300	463		250	475	522	354	693		369	1315	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.42	0.51		0.91	0.39	0.60	0.43	0.78		0.41	0.62	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 80
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.91
Intersection Signal Delay: 24.9 Intersection LOS: C
Intersection Capacity Utilization 81.6% ICU Level of Service D
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



Appendix B

2020 Traffic Operations Reports

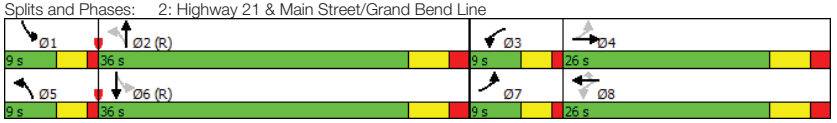
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	18	26	36	104	24	133	26	255	134	135	242	33
Future Volume (vph)	18	26	36	104	24	133	26	255	134	135	242	33
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.99					0.97	0.99	0.99		0.99	1.00	
Frt		0.913				0.850		0.948			0.982	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1626	1677	0	1656	1759	1583	1671	1595	0	1770	3093	0
Flt Permitted	0.740			0.610			0.569			0.368		
Satd. Flow (perm)	1252	1677	0	1063	1759	1533	988	1595	0	681	3093	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		39				150		38			21	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	6					6	10		7	7		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	11%	4%	3%	9%	8%	2%	8%	14%	7%	2%	16%	0%
Adj. Flow (vph)	20	28	39	113	26	145	28	277	146	147	263	36
Shared Lane Traffic (%)												
Lane Group Flow (vph)	20	67	0	113	26	145	28	423	0	147	299	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	36.0		9.0	36.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	11.3%	45.0%		11.3%	45.0%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	16.2	10.2		17.8	13.8	13.8	46.5	38.5		52.2	46.4	
Actuated g/C Ratio	0.20	0.13		0.22	0.17	0.17	0.58	0.48		0.65	0.58	
v/c Ratio	0.07	0.27		0.41	0.09	0.37	0.04	0.54		0.26	0.17	
Control Delay	22.6	19.8		29.3	29.9	8.7	6.0	17.5		7.0	9.1	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	22.6	19.8		29.3	29.9	8.7	6.0	17.5		7.0	9.1	
LOS	C	B		C	C	A	A	B		A	A	
Approach Delay		20.4			18.8			16.8			8.4	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			B			B			A	
Queue Length 50th (m)	2.4	4.1		14.3	3.2	0.0	1.5	42.6		8.2	8.9	
Queue Length 95th (m)	7.4	15.2		27.4	10.8	15.0	4.3	76.9		15.8	20.8	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	277	448		273	439	495	625	786		555	1802	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.07	0.15		0.41	0.06	0.29	0.04	0.54		0.26	0.17	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.54
Intersection Signal Delay: 14.6 Intersection LOS: B
Intersection Capacity Utilization 55.9% ICU Level of Service B
Analysis Period (min) 15

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



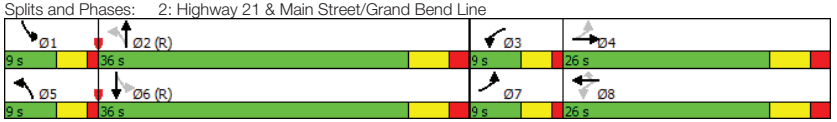
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	53	47	69	157	44	143	41	225	156	131	283	58
Future Volume (vph)	53	47	69	157	44	143	41	225	156	131	283	58
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.97	0.98		0.99		0.95	0.99	0.99		0.99	0.99	
Frt		0.911				0.850		0.939			0.975	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1704	0	1752	1900	1524	1719	1599	0	1719	3253	0
Flt Permitted	0.726			0.489			0.531			0.378		
Satd. Flow (perm)	1317	1704	0	891	1900	1451	949	1599	0	680	3253	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		75				155		50			34	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	14		12	12		14	10		6	6		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	58	51	75	171	48	155	45	245	170	142	308	63
Shared Lane Traffic (%)												
Lane Group Flow (vph)	58	126	0	171	48	155	45	415	0	142	371	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	36.0		9.0	36.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	11.3%	45.0%		11.3%	45.0%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	15.1	10.5		17.3	12.3	12.3	46.5	38.1		51.4	43.8	
Actuated g/C Ratio	0.19	0.13		0.22	0.15	0.15	0.58	0.48		0.64	0.55	
v/c Ratio	0.21	0.44		0.66	0.16	0.44	0.07	0.53		0.26	0.21	
Control Delay	23.9	19.9		39.0	31.5	10.0	6.4	17.1		7.3	10.4	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	23.9	19.9		39.0	31.5	10.0	6.4	17.1		7.3	10.4	
LOS	C	B		D	C	A	A	B		A	B	
Approach Delay		21.2			26.0			16.1			9.6	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			B			A	
Queue Length 50th (m)	7.1	7.5		22.3	7.0	0.0	2.4	39.7		7.9	15.5	
Queue Length 95th (m)	15.5	22.5		#40.6	16.3	16.0	6.5	76.1		16.3	26.4	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	277	482		261	475	479	612	788		542	1797	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.21	0.26		0.66	0.10	0.32	0.07	0.53		0.26	0.21	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.66
Intersection Signal Delay: 16.9 Intersection LOS: B
Intersection Capacity Utilization 58.1% ICU Level of Service B
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



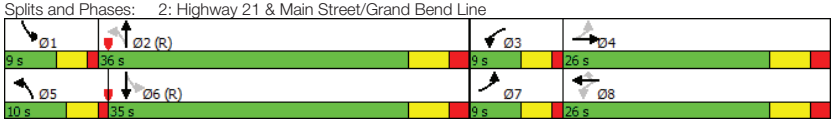
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	75	62	123	193	80	225	126	360	200	183	415	134
Future Volume (vph)	75	62	123	193	80	225	126	360	200	183	415	134
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.93	0.97		0.98		0.91	0.97	0.99		1.00	0.98	
Frt		0.900				0.850		0.946			0.963	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1659	0	1752	1900	1524	1719	1604	0	1719	3178	0
Flt Permitted	0.701			0.467			0.415			0.165		
Satd. Flow (perm)	1218	1659	0	840	1900	1380	729	1604	0	297	3178	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		120				245		40			61	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	38		28	28		38	39		10	10		39
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	82	67	134	210	87	245	137	391	217	199	451	146
Shared Lane Traffic (%)												
Lane Group Flow (vph)	82	201	0	210	87	245	137	608	0	199	597	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	10.0	36.0		9.0	35.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	12.5%	45.0%		11.3%	43.8%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	18.6	11.6		19.4	13.4	13.4	42.7	32.6		48.2	35.3	
Actuated g/C Ratio	0.23	0.14		0.24	0.17	0.17	0.53	0.41		0.60	0.44	
v/c Ratio	0.26	0.59		0.81	0.27	0.56	0.28	0.90		0.54	0.42	
Control Delay	23.3	20.9		50.3	32.2	9.8	8.7	40.8		14.9	15.4	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	23.3	20.9		50.3	32.2	9.8	8.7	40.8		14.9	15.4	
LOS	C	C		D	C	A	A	D		B	B	
Approach Delay		21.6			29.1			34.9			15.3	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			C			B	
Queue Length 50th (m)	10.1	12.1		28.1	13.0	0.0	7.6	81.7		11.5	28.0	
Queue Length 95th (m)	19.0	30.3		#50.6	24.4	18.9	17.7	#158.8		#32.5	49.5	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	317	504		260	475	528	489	676		371	1436	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.26	0.40		0.81	0.18	0.46	0.28	0.90		0.54	0.42	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 90
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.90
Intersection Signal Delay: 25.4 Intersection LOS: C
Intersection Capacity Utilization 84.0% ICU Level of Service E
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



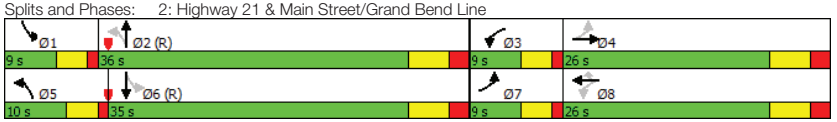
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	117	71	148	212	174	291	143	366	139	142	531	233
Future Volume (vph)	117	71	148	212	174	291	143	366	139	142	531	233
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.83	0.86		0.87		0.76	0.95	0.98		0.98	0.93	
Frt		0.899				0.850		0.959			0.954	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1475	0	1752	1900	1524	1719	1608	0	1719	3006	0
Flt Permitted	0.615			0.418			0.241			0.257		
Satd. Flow (perm)	950	1475	0	672	1900	1153	414	1608	0	458	3006	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		125				316		27			97	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	114		161	161		114	111		26	26		111
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	127	77	161	230	189	316	155	398	151	154	577	253
Shared Lane Traffic (%)												
Lane Group Flow (vph)	127	238	0	230	189	316	155	549	0	154	830	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		9.0	26.0	26.0	10.0	36.0		9.0	35.0	
Total Split (%)	11.3%	32.5%		11.3%	32.5%	32.5%	12.5%	45.0%		11.3%	43.8%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	20.9	13.9		21.7	15.7	15.7	43.3	33.6		42.9	33.4	
Actuated g/C Ratio	0.26	0.17		0.27	0.20	0.20	0.54	0.42		0.54	0.42	
v/c Ratio	0.42	0.66		0.92	0.51	0.66	0.45	0.79		0.43	0.63	
Control Delay	24.5	23.6		67.0	33.8	10.7	13.1	31.2		13.2	19.6	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	24.5	23.6		67.0	33.8	10.7	13.1	31.2		13.2	19.6	
LOS	C	C		E	C	B	B	C		B	B	
Approach Delay		23.9			34.3			27.2			18.6	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			C			B	
Queue Length 50th (m)	15.2	16.6		29.3	28.3	0.0	9.9	71.3		9.9	47.3	
Queue Length 95th (m)	24.8	36.5		#55.1	44.0	22.0	22.8	#138.9		22.7	74.0	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	299	462		249	475	525	348	691		362	1312	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.42	0.52		0.92	0.40	0.60	0.45	0.79		0.43	0.63	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 80
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.92
Intersection Signal Delay: 25.6 Intersection LOS: C
Intersection Capacity Utilization 82.4% ICU Level of Service E
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



Appendix C

2020 Option 1 Traffic Operations Reports

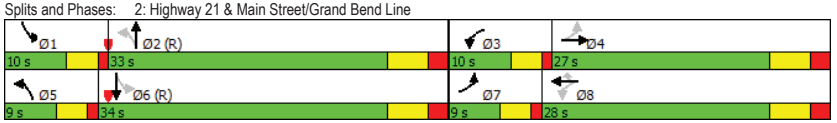
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	18	26	36	104	24	133	26	255	134	135	242	33
Future Volume (vph)	18	26	36	104	24	133	26	255	134	135	242	33
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95	1.00	0.95	0.95
Ped Bike Factor	0.99					0.98	0.99	0.99		0.99	1.00	
Frt		0.913				0.850		0.948			0.982	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1626	1677	0	1656	1759	1583	1671	3031	0	1770	3093	0
Flt Permitted	0.740			0.575			0.569			0.450		
Satd. Flow (perm)	1258	1677	0	1002	1759	1553	988	3031	0	832	3093	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		39				150		129			21	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			114.0	
Travel Time (s)		20.4			149.1			30.0			8.2	
Confl. Peds. (#/hr)	6					6	10		7	7		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	11%	4%	3%	9%	8%	2%	8%	14%	7%	2%	16%	0%
Adj. Flow (vph)	20	28	39	113	26	145	28	277	146	147	263	36
Shared Lane Traffic (%)												
Lane Group Flow (vph)	20	67	0	113	26	145	28	423	0	147	299	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	27.0		10.0	28.0	28.0	9.0	33.0		10.0	34.0	
Total Split (%)	11.3%	33.8%		12.5%	35.0%	35.0%	11.3%	41.3%		12.5%	42.5%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	16.0	10.0		18.8	14.4	14.4	45.8	37.8		51.7	45.8	
Actuated g/C Ratio	0.20	0.12		0.24	0.18	0.18	0.57	0.47		0.65	0.57	
v/c Ratio	0.07	0.28		0.40	0.08	0.36	0.05	0.28		0.23	0.17	
Control Delay	22.2	20.1		27.8	29.4	8.4	6.3	10.2		7.0	9.4	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	22.2	20.1		27.8	29.4	8.4	6.3	10.2		7.0	9.4	
LOS	C	C		C	C	A	A	B		A	A	
Approach Delay		20.6			18.1			9.9			8.6	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			B			A			A	
Queue Length 50th (m)	2.3	4.1		14.0	3.2	0.0	1.5	14.5		8.6	9.2	
Queue Length 95th (m)	7.4	15.4		27.3	10.8	15.0	4.3	25.6		15.8	20.9	
Internal Link Dist (m)		260.0			2046.2			392.1			90.0	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	274	468		284	483	535	617	1500		633	1778	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.07	0.14		0.40	0.05	0.27	0.05	0.28		0.23	0.17	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.40
Intersection Signal Delay: 12.0 Intersection LOS: B
Intersection Capacity Utilization 50.7% ICU Level of Service A
Analysis Period (min) 15

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line

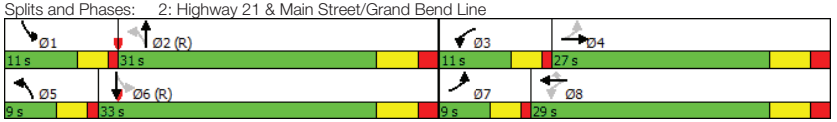


Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	53	47	69	157	44	143	41	225	156	131	283	58
Future Volume (vph)	53	47	69	157	44	143	41	225	156	131	283	58
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95	1.00	0.95	0.95
Ped Bike Factor	0.98	0.98		0.99		0.97	0.99	0.99		0.99	0.99	
Frt		0.911				0.850		0.939			0.975	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1704	0	1752	1900	1524	1719	3038	0	1719	3253	0
Flt Permitted	0.726			0.467			0.531			0.453		
Satd. Flow (perm)	1331	1704	0	852	1900	1480	949	3038	0	815	3253	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		75				155		170			32	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			105.1	
Travel Time (s)		20.4			149.1			30.0			7.6	
Confl. Peds. (#/hr)	14		12	12		14	10		6	6		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	58	51	75	171	48	155	45	245	170	142	308	63
Shared Lane Traffic (%)												
Lane Group Flow (vph)	58	126	0	171	48	155	45	415	0	142	371	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	27.0		11.0	29.0	29.0	9.0	31.0		11.0	33.0	
Total Split (%)	11.3%	33.8%		13.8%	36.3%	36.3%	11.3%	38.8%		13.8%	41.3%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	15.1	10.5		20.1	13.9	13.9	44.7	36.4		49.9	42.2	
Actuated g/C Ratio	0.19	0.13		0.25	0.17	0.17	0.56	0.46		0.62	0.53	
v/c Ratio	0.21	0.44		0.56	0.15	0.40	0.08	0.28		0.24	0.21	
Control Delay	22.5	19.9		30.9	29.4	8.8	7.3	9.5		7.8	11.5	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	22.5	19.9		30.9	29.4	8.8	7.3	9.5		7.8	11.5	
LOS	C	B		C	C	A	A	A		A	B	
Approach Delay		20.7			21.6			9.3			10.5	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			A			B	
Queue Length 50th (m)	6.8	7.5		21.5	6.8	0.0	2.6	12.2		8.6	16.5	
Queue Length 95th (m)	14.9	22.5		36.6	15.7	15.4	7.0	24.6		17.5	28.1	
Internal Link Dist (m)		260.0			2046.2			392.1			81.1	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	278	502		304	546	535	591	1473		604	1730	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.21	0.25		0.56	0.09	0.29	0.08	0.28		0.24	0.21	

Intersection Summary		
Area Type:	Other	
Cycle Length: 80		
Actuated Cycle Length: 80		
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green		
Natural Cycle: 70		
Control Type: Actuated-Coordinated		
Maximum v/c Ratio: 0.56		
Intersection Signal Delay: 14.1	Intersection LOS: B	
Intersection Capacity Utilization 53.1%	ICU Level of Service A	
Analysis Period (min) 15		

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line

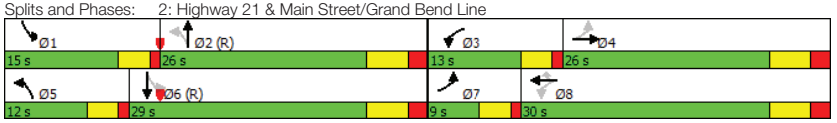


Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	75	62	123	193	80	225	126	360	200	183	415	134
Future Volume (vph)	75	62	123	193	80	225	126	360	200	183	415	134
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95	1.00	0.95	0.95
Ped Bike Factor	0.96	0.97		0.98		0.94	0.97	0.99		0.99	0.98	
Frt		0.900				0.850		0.946			0.963	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1659	0	1752	1900	1524	1719	3047	0	1719	3178	0
Flt Permitted	0.701			0.379			0.383			0.328		
Satd. Flow (perm)	1253	1659	0	684	1900	1437	671	3047	0	590	3178	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		120				245		126			55	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			106.5	
Travel Time (s)		20.4			149.1			30.0			7.7	
Confl. Peds. (#/hr)	38		28	28		38	39		10	10		39
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	82	67	134	210	87	245	137	391	217	199	451	146
Shared Lane Traffic (%)												
Lane Group Flow (vph)	82	201	0	210	87	245	137	608	0	199	597	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		13.0	30.0	30.0	12.0	26.0		15.0	29.0	
Total Split (%)	11.3%	32.5%		16.3%	37.5%	37.5%	15.0%	32.5%		18.8%	36.3%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	18.6	11.6		26.4	17.2	17.2	40.0	29.4		43.2	31.0	
Actuated g/C Ratio	0.23	0.14		0.33	0.22	0.22	0.50	0.37		0.54	0.39	
v/c Ratio	0.25	0.59		0.61	0.21	0.49	0.31	0.51		0.43	0.47	
Control Delay	20.5	20.9		28.3	27.8	7.4	10.9	18.3		12.1	19.1	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	20.5	20.9		28.3	27.8	7.4	10.9	18.3		12.1	19.1	
LOS	C	C		C	C	A	B	B		B	B	
Approach Delay		20.8			18.8			16.9			17.3	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			B			B			B	
Queue Length 50th (m)	9.4	12.1		26.0	12.2	0.0	8.9	29.0		13.5	31.7	
Queue Length 95th (m)	17.4	30.3		39.6	22.7	17.4	20.2	54.6		28.6	56.1	
Internal Link Dist (m)		260.0			2046.2			392.1			82.5	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	322	504		345	570	602	458	1198		494	1264	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.25	0.40		0.61	0.15	0.41	0.30	0.51		0.40	0.47	

Intersection Summary		
Area Type:	Other	
Cycle Length: 80		
Actuated Cycle Length: 80		
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green		
Natural Cycle: 70		
Control Type: Actuated-Coordinated		
Maximum v/c Ratio: 0.61		
Intersection Signal Delay: 17.9	Intersection LOS: B	
Intersection Capacity Utilization 69.3%	ICU Level of Service C	
Analysis Period (min) 15		

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



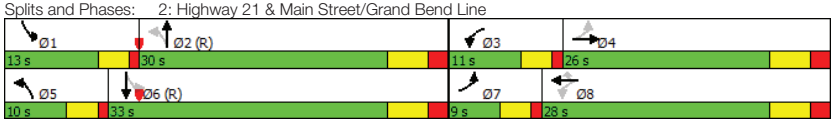
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	117	71	148	212	174	291	143	366	139	142	531	233
Future Volume (vph)	117	71	148	212	174	291	143	366	139	142	531	233
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		0	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95	1.00	0.95	0.95
Ped Bike Factor	0.82	0.86		0.88		0.76	0.94	0.98		0.98	0.93	
Frt		0.899				0.850		0.959			0.954	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1475	0	1752	1900	1524	1719	3056	0	1719	3006	0
Flt Permitted	0.639			0.359			0.237			0.390		
Satd. Flow (perm)	982	1475	0	582	1900	1153	405	3056	0	692	3006	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		125				316		70			93	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			92.6	
Travel Time (s)		20.4			149.1			30.0			6.7	
Confl. Peds. (#/hr)	114		161	161		114	111		26	26		111
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	127	77	161	230	189	316	155	398	151	154	577	253
Shared Lane Traffic (%)												
Lane Group Flow (vph)	127	238	0	230	189	316	155	549	0	154	830	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA		pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2			6		
Detector Phase	7	4		3	8	8	5	2		1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0		5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0		9.0	26.0	
Total Split (s)	9.0	26.0		11.0	28.0	28.0	10.0	30.0		13.0	33.0	
Total Split (%)	11.3%	32.5%		13.8%	35.0%	35.0%	12.5%	37.5%		16.3%	41.3%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0		3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0		1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0		4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag		Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes		Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max		None	C-Max	
Act Effct Green (s)	20.3	13.3		24.7	17.1	17.1	41.2	31.2		42.2	31.7	
Actuated g/C Ratio	0.25	0.17		0.31	0.21	0.21	0.52	0.39		0.53	0.40	
v/c Ratio	0.43	0.68		0.82	0.47	0.64	0.46	0.44		0.33	0.67	
Control Delay	23.8	24.9		45.1	31.5	9.8	14.5	18.1		11.2	21.5	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0		0.0	0.0	
Total Delay	23.8	24.9		45.1	31.5	9.8	14.5	18.1		11.2	21.5	
LOS	C	C		D	C	A	B	B		B	C	
Approach Delay		24.5			26.4			17.3			19.9	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			B			B	
Queue Length 50th (m)	14.9	16.9		28.9	28.0	0.0	10.2	27.9		10.2	48.8	
Queue Length 95th (m)	23.9	36.8		#47.8	42.7	21.3	23.8	49.7		23.7	77.7	
Internal Link Dist (m)		260.0			2046.2			392.1			68.6	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	298	462		282	522	546	338	1235		492	1248	
Starvation Cap Reductn	0	0		0	0	0	0	0		0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0		0	0	
Storage Cap Reductn	0	0		0	0	0	0	0		0	0	
Reduced v/c Ratio	0.43	0.52		0.82	0.36	0.58	0.46	0.44		0.31	0.67	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.82
Intersection Signal Delay: 21.6 Intersection LOS: C
Intersection Capacity Utilization 78.1% ICU Level of Service D
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



Appendix D

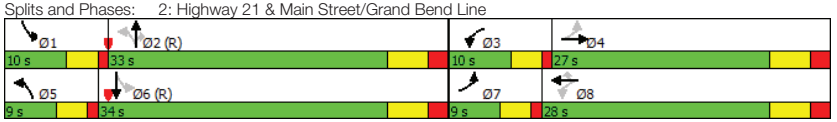
2020 Option 2 Traffic Operations Reports

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	18	26	36	104	24	133	26	255	134	135	242	33
Future Volume (vph)	18	26	36	104	24	133	26	255	134	135	242	33
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		55.0	40.0		0.0
Storage Lanes	1		0	1		1	1		1	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.99					0.97	0.99		0.97	0.99	1.00	
Frt		0.913				0.850			0.850		0.982	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1626	1677	0	1656	1759	1583	1671	1667	1509	1770	3093	0
Flt Permitted	0.740			0.575			0.569			0.498		
Satd. Flow (perm)	1252	1677	0	1002	1759	1533	988	1667	1458	920	3093	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		39				150			150		21	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	6					6	10		7	7		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	11%	4%	3%	9%	8%	2%	8%	14%	7%	2%	16%	0%
Adj. Flow (vph)	20	28	39	113	26	145	28	277	146	147	263	36
Shared Lane Traffic (%)												
Lane Group Flow (vph)	20	67	0	113	26	145	28	277	146	147	299	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA	Perm	pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2		2	6		
Detector Phase	7	4		3	8	8	5	2	2	1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0	20.0	5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0	26.0	9.0	26.0	
Total Split (s)	9.0	27.0		10.0	28.0	28.0	9.0	33.0	33.0	10.0	34.0	
Total Split (%)	11.3%	33.8%		12.5%	35.0%	35.0%	11.3%	41.3%	41.3%	12.5%	42.5%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0	4.0	3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0	2.0	1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0	6.0	4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag	Lag	Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max	C-Max	None	C-Max	
Act Effct Green (s)	16.0	10.0		18.8	14.4	14.4	45.8	37.8	37.8	51.7	45.8	
Actuated g/C Ratio	0.20	0.12		0.24	0.18	0.18	0.57	0.47	0.47	0.65	0.57	
v/c Ratio	0.07	0.28		0.40	0.08	0.36	0.05	0.35	0.19	0.22	0.17	
Control Delay	22.2	20.1		27.8	29.4	8.5	6.3	16.3	3.3	6.8	9.4	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Delay	22.2	20.1		27.8	29.4	8.5	6.3	16.3	3.3	6.8	9.4	
LOS	C	C		C	C	A	A	B	A	A	A	
Approach Delay		20.6			18.1			11.4			8.6	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			B			B			A	
Queue Length 50th (m)	2.2	3.8		13.2	3.0	0.0	1.4	26.5	0.0	8.1	8.6	
Queue Length 95th (m)	7.0	14.5		25.6	10.1	14.0	4.0	46.7	9.2	14.9	19.6	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0		55.0	40.0		
Base Capacity (vph)	274	468		284	483	530	617	787	768	681	1778	
Starvation Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Storage Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Reduced v/c Ratio	0.07	0.14		0.40	0.05	0.27	0.05	0.35	0.19	0.22	0.17	

Intersection Summary		
Area Type:	Other	
Cycle Length: 80		
Actuated Cycle Length: 80		
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green		
Natural Cycle: 70		
Control Type: Actuated-Coordinated		
Maximum v/c Ratio: 0.40		
Intersection Signal Delay: 12.6	Intersection LOS: B	
Intersection Capacity Utilization 50.7%	ICU Level of Service A	
Analysis Period (min) 15		

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line

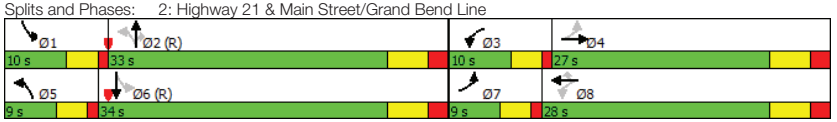


Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	53	47	69	157	44	143	41	225	156	131	283	58
Future Volume (vph)	53	47	69	157	44	143	41	225	156	131	283	58
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		55.0	40.0		0.0
Storage Lanes	1		0	1		1	1		1	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.97	0.98		0.99		0.95	0.99		0.97	0.99	0.99	
Frt		0.911				0.850			0.850		0.975	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1704	0	1752	1900	1524	1719	1681	1524	1719	3253	0
Flt Permitted	0.726			0.467			0.531			0.531		
Satd. Flow (perm)	1317	1704	0	851	1900	1451	949	1681	1475	953	3253	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		75				155			170		33	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	14		12	12		14	10		6	6		10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	58	51	75	171	48	155	45	245	170	142	308	63
Shared Lane Traffic (%)												
Lane Group Flow (vph)	58	126	0	171	48	155	45	245	170	142	371	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA	Perm	pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2		2	6		
Detector Phase	7	4		3	8	8	5	2	2	1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0	20.0	5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0	26.0	9.0	26.0	
Total Split (s)	9.0	27.0		10.0	28.0	28.0	9.0	33.0	33.0	10.0	34.0	
Total Split (%)	11.3%	33.8%		12.5%	35.0%	35.0%	11.3%	41.3%	41.3%	12.5%	42.5%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0	4.0	3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0	2.0	1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0	6.0	4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag	Lag	Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max	C-Max	None	C-Max	
Act Effct Green (s)	15.1	10.5		18.7	13.1	13.1	45.6	37.3	37.3	50.6	43.0	
Actuated g/C Ratio	0.19	0.13		0.23	0.16	0.16	0.57	0.47	0.47	0.63	0.54	
v/c Ratio	0.21	0.44		0.61	0.15	0.42	0.07	0.31	0.22	0.21	0.21	
Control Delay	23.2	19.9		34.5	30.4	9.4	6.9	16.4	3.5	7.2	11.0	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Delay	23.2	19.9		34.5	30.4	9.4	6.9	16.4	3.5	7.2	11.0	
LOS	C	B		C	C	A	A	B	A	A	B	
Approach Delay		21.0			23.6			10.7			9.9	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			B			A	
Queue Length 50th (m)	6.5	7.0		20.5	6.5	0.0	2.3	22.8	0.0	7.8	15.0	
Queue Length 95th (m)	14.2	21.1		35.0	15.0	14.7	6.3	43.8	11.0	15.9	25.5	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0		55.0	40.0		
Base Capacity (vph)	277	502		279	522	511	602	783	777	681	1763	
Starvation Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Storage Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Reduced v/c Ratio	0.21	0.25		0.61	0.09	0.30	0.07	0.31	0.22	0.21	0.21	

Intersection Summary		
Area Type:	Other	
Cycle Length: 80		
Actuated Cycle Length: 80		
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green		
Natural Cycle: 70		
Control Type: Actuated-Coordinated		
Maximum v/c Ratio: 0.61		
Intersection Signal Delay: 14.8	Intersection LOS: B	
Intersection Capacity Utilization 53.1%	ICU Level of Service A	
Analysis Period (min) 15		

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



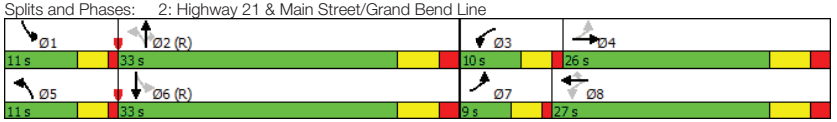
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	75	62	123	193	80	225	126	360	200	183	415	134
Future Volume (vph)	75	62	123	193	80	225	126	360	200	183	415	134
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		0.0	40.0		0.0
Storage Lanes	1		0	1		1	1		1	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.93	0.97		0.98		0.91	0.97		0.96	0.99	0.98	
Frt		0.900				0.850			0.850		0.963	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1659	0	1752	1900	1524	1719	1681	1524	1719	3178	0
Flt Permitted	0.701			0.434			0.390			0.398		
Satd. Flow (perm)	1218	1659	0	782	1900	1380	685	1681	1463	714	3178	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		120				245			217		59	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	38		28	28		38	39		10	10		39
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	82	67	134	210	87	245	137	391	217	199	451	146
Shared Lane Traffic (%)												
Lane Group Flow (vph)	82	201	0	210	87	245	137	391	217	199	597	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA	Perm	pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2		2	6		
Detector Phase	7	4		3	8	8	5	2	2	1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0	20.0	5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0	26.0	9.0	26.0	
Total Split (s)	9.0	26.0		10.0	27.0	27.0	11.0	33.0	33.0	11.0	33.0	
Total Split (%)	11.3%	32.5%		12.5%	33.8%	33.8%	13.8%	41.3%	41.3%	13.8%	41.3%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0	4.0	3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0	2.0	1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0	6.0	4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag	Lag	Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max	C-Max	None	C-Max	
Act Effct Green (s)	18.6	11.6		21.2	14.4	14.4	43.4	33.1	33.1	45.5	34.1	
Actuated g/C Ratio	0.23	0.14		0.26	0.18	0.18	0.54	0.41	0.41	0.57	0.43	
v/c Ratio	0.26	0.59		0.75	0.26	0.55	0.29	0.56	0.30	0.38	0.43	
Control Delay	22.6	20.9		42.0	31.0	9.2	9.1	23.0	4.1	9.9	16.4	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Delay	22.6	20.9		42.0	31.0	9.2	9.1	23.0	4.1	9.9	16.4	
LOS	C	C		D	C	A	A	C	A	A	B	
Approach Delay		21.4			25.4			15.0			14.8	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			B			B	
Queue Length 50th (m)	9.3	11.4		25.8	12.0	0.0	7.4	41.8	0.0	11.3	27.1	
Queue Length 95th (m)	17.4	28.4		#43.1	22.4	17.3	17.2	79.6	13.4	24.3	48.6	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0			40.0		
Base Capacity (vph)	317	504		279	498	542	483	695	732	523	1388	
Starvation Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Storage Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Reduced v/c Ratio	0.26	0.40		0.75	0.17	0.45	0.28	0.56	0.30	0.38	0.43	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.75
Intersection Signal Delay: 18.1 Intersection LOS: B
Intersection Capacity Utilization 71.5% 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: 2: Highway 21 & Main Street/Grand Bend Line



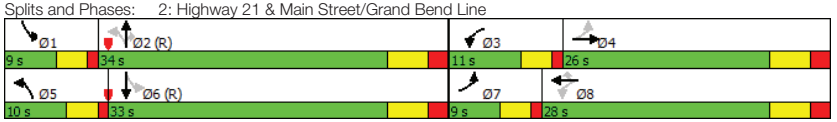
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	117	71	148	212	174	291	143	366	139	142	531	233
Future Volume (vph)	117	71	148	212	174	291	143	366	139	142	531	233
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	25.0		0.0	55.0		55.0	55.0		55.0	40.0		0.0
Storage Lanes	1		0	1		1	1		1	1		0
Taper Length (m)	7.5			7.5			7.5			7.5		
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.95
Ped Bike Factor	0.82	0.86		0.88		0.76	0.95		0.93	0.98	0.93	
Frt		0.899				0.850			0.850		0.954	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1770	1475	0	1752	1900	1524	1719	1681	1524	1719	3006	0
Flt Permitted	0.639			0.359			0.231			0.399		
Satd. Flow (perm)	982	1475	0	582	1900	1153	397	1681	1415	705	3006	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)		125				316			151		93	
Link Speed (k/h)		50			50			50			50	
Link Distance (m)		284.0			2070.2			416.1			338.9	
Travel Time (s)		20.4			149.1			30.0			24.4	
Confl. Peds. (#/hr)	114		161	161		114	111		26	26		111
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles (%)	2%	0%	0%	3%	0%	6%	5%	13%	6%	5%	9%	0%
Adj. Flow (vph)	127	77	161	230	189	316	155	398	151	154	577	253
Shared Lane Traffic (%)												
Lane Group Flow (vph)	127	238	0	230	189	316	155	398	151	154	830	0
Turn Type	pm+pt	NA		pm+pt	NA	Perm	pm+pt	NA	Perm	pm+pt	NA	
Protected Phases	7	4		3	8		5	2		1	6	
Permitted Phases	4			8		8	2		2	6		
Detector Phase	7	4		3	8	8	5	2	2	1	6	
Switch Phase												
Minimum Initial (s)	5.0	10.0		5.0	10.0	10.0	5.0	20.0	20.0	5.0	20.0	
Minimum Split (s)	9.0	26.0		9.0	26.0	26.0	9.0	26.0	26.0	9.0	26.0	
Total Split (s)	9.0	26.0		11.0	28.0	28.0	10.0	34.0	34.0	9.0	33.0	
Total Split (%)	11.3%	32.5%		13.8%	35.0%	35.0%	12.5%	42.5%	42.5%	11.3%	41.3%	
Yellow Time (s)	3.0	4.0		3.0	4.0	4.0	3.0	4.0	4.0	3.0	4.0	
All-Red Time (s)	1.0	2.0		1.0	2.0	2.0	1.0	2.0	2.0	1.0	2.0	
Lost Time Adjust (s)	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Lost Time (s)	4.0	6.0		4.0	6.0	6.0	4.0	6.0	6.0	4.0	6.0	
Lead/Lag	Lead	Lag		Lead	Lag	Lag	Lead	Lag	Lag	Lead	Lag	
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	
Recall Mode	None	None		None	None	None	None	C-Max	C-Max	None	C-Max	
Act Effct Green (s)	20.3	13.3		24.7	17.1	17.1	42.0	32.1	32.1	41.3	31.7	
Actuated g/C Ratio	0.25	0.17		0.31	0.21	0.21	0.52	0.40	0.40	0.52	0.40	
v/c Ratio	0.43	0.68		0.82	0.47	0.64	0.46	0.59	0.23	0.33	0.67	
Control Delay	23.8	24.9		45.1	31.5	9.8	14.0	24.4	4.4	11.8	21.5	
Queue Delay	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Total Delay	23.8	24.9		45.1	31.5	9.8	14.0	24.4	4.4	11.8	21.5	
LOS	C	C		D	C	A	B	C	A	B	C	
Approach Delay		24.5			26.4			17.8			20.0	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Approach LOS		C			C			B			B	
Queue Length 50th (m)	14.0	15.8		27.1	26.2	0.0	9.6	45.3	0.0	9.5	45.7	
Queue Length 95th (m)	22.4	34.5		#44.8	40.1	19.9	22.3	79.6	11.1	22.2	72.9	
Internal Link Dist (m)		260.0			2046.2			392.1			314.9	
Turn Bay Length (m)	25.0			55.0		55.0	55.0		55.0	40.0		
Base Capacity (vph)	298	462		282	522	546	339	674	657	460	1248	
Starvation Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Spillback Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Storage Cap Reductn	0	0		0	0	0	0	0	0	0	0	
Reduced v/c Ratio	0.43	0.52		0.82	0.36	0.58	0.46	0.59	0.23	0.33	0.67	

Intersection Summary

Area Type: Other
Cycle Length: 80
Actuated Cycle Length: 80
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green
Natural Cycle: 70
Control Type: Actuated-Coordinated
Maximum v/c Ratio: 0.82
Intersection Signal Delay: 21.7 Intersection LOS: C
Intersection Capacity Utilization 78.1% ICU Level of Service D
Analysis Period (min) 15
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 2: Highway 21 & Main Street/Grand Bend Line



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18 October 2017
Project: 161920

Ken Logtenberg, P. Eng.
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Engineers and Planners
62 North Street
Goderich ON N7A 2T4

Dear Mr. Logtenberg:

RE: ONTARIO STREET, GRAND BEND IMPROVEMENT OPPORTUNITIES AND PRIORITIES

The following letter outlines the traffic operations experienced on Ontario Street in Grand Bend and the potential improvement opportunities that exist.

Current Operations

Counts at the intersection of Ontario Street with Main Street and with Lake Road were conducted on the May (Victoria Day) and July (Canada Day) long weekends in 2015, and the count at Ontario Street and Main Street was counted again on the September (Labour Day) long weekend in 2016. **Figure 1** shows the traffic volumes observed at these intersections. Video recordings of queueing and pedestrian activity were also taken during the September 2016 counts from the No Frills plaza in the south and up to Oak Street in the north. Video recording locations are shown in **Figure 2**.

Queue Observations

Northbound queues at the Main Street intersection on Ontario Street observed during the Saturday of Labour Day weekend show that the northbound capacity at this intersection is not adequate for the demand. The back of queue was observed to reach at least Lake Street several times between 10:55am and 1:55pm. When the queue reaches Lake Street, it disrupts the operations of the signal at the intersection of Ontario Street and Lake Street, preventing northbound through movements from proceeding through the intersection. This disruption causes the northbound queue from Lake Street to reach back as far as the No Frills plaza. Due to the ripple effect nature of traffic flow, the queue at Lake Street was observed as taking upwards of nine minutes to clear even after the queue from Main Street was no longer reaching Lake Street.

Synchro Analysis

The volumes were analysed in Synchro. It was found that Synchro is under representing the queues experienced in the field. Attempts to have Synchro better quantify the queue lengths by lowering the

Ideal Saturation Flow Rate (SFR) from the default of 1900 vehicles per hour (vph) to 1600 vph (which is generally considered to be near the low end of the acceptable SFR scale), only had the 95th percentile queue be reported to be 165 metres. Video recordings from the same day as the September 2016 count show that queues experienced were considerably longer, at times reaching to Lake Street as noted above, which caused delays at Lake Street causing its queue to extend further south near the No Frills at times. A possible cause of this difference in the analysis and real-world conditions would be the density of driveways on Ontario Street, causing a number of mid-block turning movements. As Synchro does not model driveway volumes, this could explain some of the discrepancy. Therefore, results calculated by Synchro can not be relied upon solely to assess traffic operations, particularly queue lengths, but can be used to assess the relative changes by potential improvements assessed below.

Improvements at Main Street Intersection

A widening of the bridge across Parkhill Creek on Ontario Street, which is immediately south of the intersection of Ontario Street and Main Street, is currently being considered by the Ministry of Transportation and the Municipality of Lambton Shores. The bridge currently has width to accommodate 4 travel lanes, which are used by two southbound lanes, a northbound left-turn lane and a northbound through/right-turn lane. The shared northbound through/right-turn lane does not have sufficient capacity to accommodate the volumes experienced during the high season, and therefore a widening to accommodate a fifth lane on the bridge is under consideration. Two options were considered:

- separate the through and right-turn movements into their own lanes, or
- add a dedicated through lane, keeping a shared through/right-turn lane.

These two designs were analysed in a report dated January 2016[1] which concluded that the design that provided a through lane and a shared through/right-turn lane is the option that provides the most benefit to traffic operations. The design that provided a separate through land and separate right-turn lane could be considered a backup design, in the case that two northbound receiving lanes on the north leg of the intersection could not be accommodated, as improvement to the capacity of the intersection would still be provided over the current design.

Improvements North of Main Street

Currently, Ontario Street is a four-lane cross section north of Main Street. At the intersection of Main Street, the north leg has a southbound shared through / right-turn lane, a southbound through lane, a southbound left-turn lane, and a northbound receiving lane. The southbound left-turn lane has a storage length of approximately 40 metres. North of the storage length, the lane serves as a two-way left-turn lane until Ontario Street narrows to a two-lane cross section north of Oak Street.

If the improvements at Main Street include a second northbound through lane, then a second northbound receiving lane is required at this intersection. Widening is possible immediately north of the intersection between Main Street and Municipal Drive. Beyond that, widening Ontario Street is

[1] Ontario Street Bridge, Grand Bend Traffic Operations Study, Paradigm Transportation Solutions Limited, January 2016.

difficult due to the proximity of buildings and sidewalks to the existing curbs. Therefore, there are two options to accommodate the additional northbound lane:

- merge the two northbound receiving lanes at Municipal Drive, or
- convert the two-way left-turn lane to a northbound lane.

The positive implication of the first option is that the two-way left-turn lane is kept intact, which is useful for a street with the high density of driveways like Ontario Street. However, the negative implication is the short distance from the intersection to the point of merging of the second northbound lane from Main Street may result in a poor lane distribution of northbound through traffic as regular traffic may avoid using the lane that ends. This concept option is shown in **Figure 3**.

For the second option, the positive implication is that the continuation of the second northbound lane would promote a more even distribution of vehicles using the northbound through lanes at the Main Street intersection. The negative implication is that it would result in the loss of the two-way left-turn lane, which would mean left-turns would wait in the inside through lane for both northbound and southbound directions, effectively reducing the capacity of Ontario Street and creating additional lane changing situations, which may increase minor collisions. An additional consideration of this option would include where the northbound direction would merge back into a single lane, as Ontario Street narrows to a two-lane cross-section approximately 175 metres north of Oak Street. Two potential solutions to this would include:

- Having the inside northbound lane become a dedicated northbound left-turn lane at Oak Street, or
- Continuing the two northbound through lanes past Oak Street and merging into a single lane at the point where the southbound lanes widen to two lanes (at approximately 175 metres north of Oak Street).

These concepts are shown in **Figure 4** and **Figure 5**. To encourage even distribution of the two northbound through lanes at the intersection with Main Street, the second option (converting the two-way left-turn lane to an additional northbound lane) is preferable between these two.

If the improvement option at the intersection of Main Street and Ontario Street does not include the additional northbound through lane, then no improvements are necessary north of Main Street.

Ontario Street and Oak Street

It was requested that the intersection of Ontario Street and Oak Street be analysed to determine if a signal is warranted at this intersection, as it has been reported that vehicles experience long wait times to turn onto Ontario Street from Oak Street due to high volumes on Ontario Street. For a signal to be warranted at an intersection, a number of warrant calculations must be met over 8 hours of a typical day. These warrant calculations consider both volumes on the main street (Ontario Street) and the minor street (Oak Street). Turning movement counts were conducted at this intersection during the mid-day Saturday peak period of the Labour Day weekend. While Ontario Street does have high volumes, there were insufficient volumes on Oak Street during the peak period, let alone a sustained eight hours of volumes, to justify a signal at this intersection.

There are limited alternatives for vehicles trying to access Ontario Street from the west besides travelling south to turn left onto Main Street. As approximately 75% of the vehicles on Oak Street

turning onto Ontario Street are right-turns, a partial solution to relieving delays at Oak Street would be to widen the west leg of the intersection on Oak Street to allow right-turning vehicles to by-pass vehicles waiting to turn left.

Improvements at Lake Road

The operational issues at Lake Road are believed to be caused by the excess northbound queueing at the intersection of Ontario Street and Main Street, and that improvements to that intersection would likely solve any operational issues at Lake Road. Currently, if the northbound queue from Main Street reaches Lake Street, it prevents northbound traffic from entering the intersection and causing the queues to reach further back, sometimes as far as the No Frills Plaza, approximately 300 metres south of Lake Street. However, if it is shown that improvements at Main Street do not improve the operations at Lake Street, three options were analysed in Synchro 9 to determine the impact of altering the intersection:

- Existing Configuration (4 lane cross section with left-turn lanes)
- Widened to a 5-lane cross section with an additional northbound through lane, and
- Converting the left-turn lanes in to a northbound through lane (4 lane cross-section without left-turn lanes)

Synchro considers intersections independently when it does the analysis (ie. does not consider the impact of the queue from Main Street) and the results for the existing configuration indicate that the intersection should operate well. In certain cases, Synchro can underestimate queue lengths, therefore this analysis is not to determine the absolute operations of the intersection, but rather to compare the options to each other. **Table 1** summarizes the results of the Synchro analysis, in which an effort was made to keep the signal timings as similar as possible between scenarios. Counts from the 2015 long weekends were used in this analysis.

TABLE 1: LAKE ROAD OPERATIONAL COMPARISONS

Scenario	Northbound Delay	Southbound Delay	Overall Intersection Delay
Existing	11.1 s	6.9 s	13.3 s
5-lane w/ LT lanes	6.8 s	6.9 s	11.4 s
4-lane w/o LT lanes	7.4 s	7.3 s	11.9 s

In this case, the scenario with the addition northbound through lane proved to have the lowest delays. The scenario with no left-turn lanes showed to be better than the existing configuration. In most cases, having no left-turn lane on the main street at a signalized intersection is not recommended, as left-turns would hold up through movements behind them and therefore turning one of the through lanes into a de-facto dedicated left-turn lane. However, the number of northbound and southbound left-turns observed at this intersection are quite low which allows the intersection to operate well without left-turn lanes. Caution should be used, however, as weekday volumes may show higher left-turn demands.

However, this analysis shows that the difference between scenarios is minor, and if the improvements at Main Street solve the operational issues at Lake Road, then no improvements are recommended.

Ontario Street Cross Section South of Main Street

Similar to the cross-section north of Main Street, the section of Ontario Street south of Main Street may require to be reconfigured if the Lake Road intersection adds an additional northbound through lane. This would require a second northbound receiving lane north of Lake Street, and the same options as north of Main Street would be applicable in this section:

- merge the two northbound receiving lanes north of Lake Road, or
- convert the two way left-turn lane to a northbound lane.

Similar to the section north of Main Street, the option to convert the two-way left-turn lane to a northbound lane would best promote even distribution of the two northbound lanes. However, this would result in left-turns waiting in the inside through lane for both directions and, similar to the situation north of Main Street, would result in lower capacity and increased collision risks.

If the two-way left-turn lanes are removed both north of Main Street and north of Lake Road, then it follows that the two-way left-turn lane south of Lake Road could be converted to a northbound through lane to promote uniformity along Ontario Street within Grand Bend.

Left Turn Lane at No Frills Plaza

If the conversion of the centre two-way left-turn lane to a northbound lane occurs south of Lake Road, the need for maintaining a southbound left-turn lane into the No Frills Plaza at the south end of Grand Bend should be analysed. No turning movement data for this plaza was collected as part of this study, so therefore a recommendation cannot be made. However, given that most inbound traffic to this plaza is likely to come from the north (and therefore be a southbound left-turning movement), a southbound left-turn lane has the potential to be warranted. Maintaining a southbound left-turn lane at this location would either require widening Ontario Street to 5 lanes as this location, or offsetting the start of the second northbound lane to be north of the plaza. The determination preferred method of accommodating the lanes would require further study.

Bicycle Facilities

It is noted that dedicated bicycle facilities are planned for Ontario Street. The ultimate design of these facilities is not final at this time. The design could include on-street bike lanes, off-street multi-use trails, or other designs.

Pedestrian Crossings

Recently, two pedestrian crossovers were installed on Ontario Street; one approximately 130 metres north of Main Street (at The Beer Store location) and the other at the No Frills Plaza at the south end of Grand Bend. Video observation of these crossings was collected on Labour Day Weekend 2016 along with the queuing observations. Observations of the busiest hour at these crossovers include:

- the crossover signal at The Beer Store was activated 18 times, serving 40 pedestrians
- The crossover signal at No Frills was activated 15 times, serving 32 pedestrians

The Beer Store Crossover

Observations of the number of pedestrians crossing north of the Beer Store signal were also carried out to determine the crossings made without the use of a signal. **Figure 6** shows the area that was observed for these pedestrian crossings. During the busiest hour of crossing for the section north of the crossover signal at The Beer Store and the intersection of Ontario Street and Oak Street, 55 pedestrians were observed crossing, in 24 groups. It is noted that some of these pedestrian crossings occurred very near the pedestrian crossover signal, which suggests a certain portion of pedestrians will cross at the near point, regardless of the location of the signal. On the other hand, it was observed that 6 of the 40 pedestrians crossing at the signal came from a location north of the signal and headed back north after crossing, suggesting that some pedestrians will walk out of their way to use a crossover signal.

If the signal was located closer to Oak Street it would serve many of the pedestrians that were observed crossing north of the current location of the signal. However, it can not be guaranteed that those 55 observed pedestrians would use a relocated crossover signal instead of crossing at the nearest point, nor can it be assumed that the pedestrians using the current crossover signal would alter their walking path to use the relocated crossover signal or the traffic signal at the intersection of Ontario Street and Main Street.

Therefore, a relocation of the pedestrian crossover signal from The Beer Store closer to Oak Street has the potential to capture more pedestrian crossings, but an increase can not be guaranteed and usage may remain the same.

No Frills Crossover

In contrast to the crossover signal at The Beer Store, the crossover signal located near the No Frills plaza does not have as many nearby pedestrian generators. Therefore, this signal is considered to be in the optimal location.

Conclusions and Recommendation

In summary, the order in which improvements should be considered are:

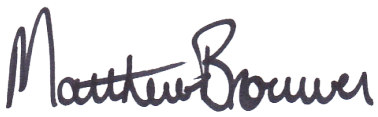
- Improvements to the intersection of Ontario Street and Main Street, including an additional northbound lane on the south leg of the intersection;
- Improvements to the cross-section of Ontario Street north of Main Street, including consideration for an additional northbound receiving lane on the north leg of the Ontario Street and Main Street intersection, and the option of converting the centre two-way left-turn lane to a northbound lane;
- Improvements to the intersection of Ontario Street and Lake Road;
- Improvements to the cross-section of Ontario Street south of Main Street, including consideration of the option of converting the centre two-way left-turn lane to a northbound lane.
- There is not enough evidence to suggest that relocating the pedestrian crossover signal from its current location near The Beer Store closer to Oak Street would increase usage.

It is recommended that improvement plans begin with the intersection of Ontario Street and Main Street, as it is likely that operational issues experienced on Ontario Street are caused by the northbound movement being over capacity. While plans should be made to continue improvements in the case that the improvements at Main Street do not solve all the operational issues, decisions to carry out those improvements shall occur after observation of the effectiveness of the improvements at Main Street.

Please contact us if you have any questions or comments.

Yours very truly,

PARADIGM TRANSPORTATION SOLUTIONS LIMITED



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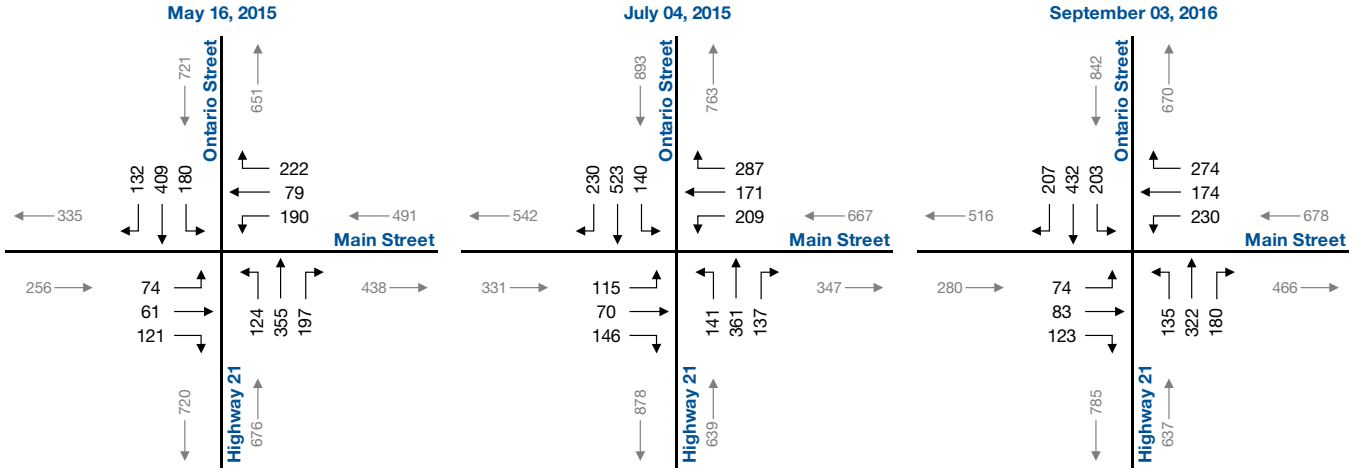
Ontario & Main Counts



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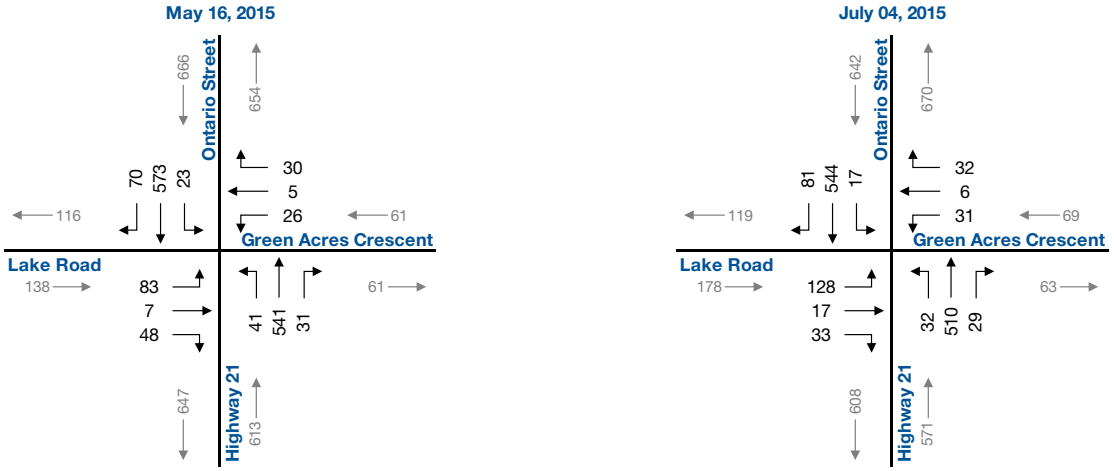

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Traffic Volumes

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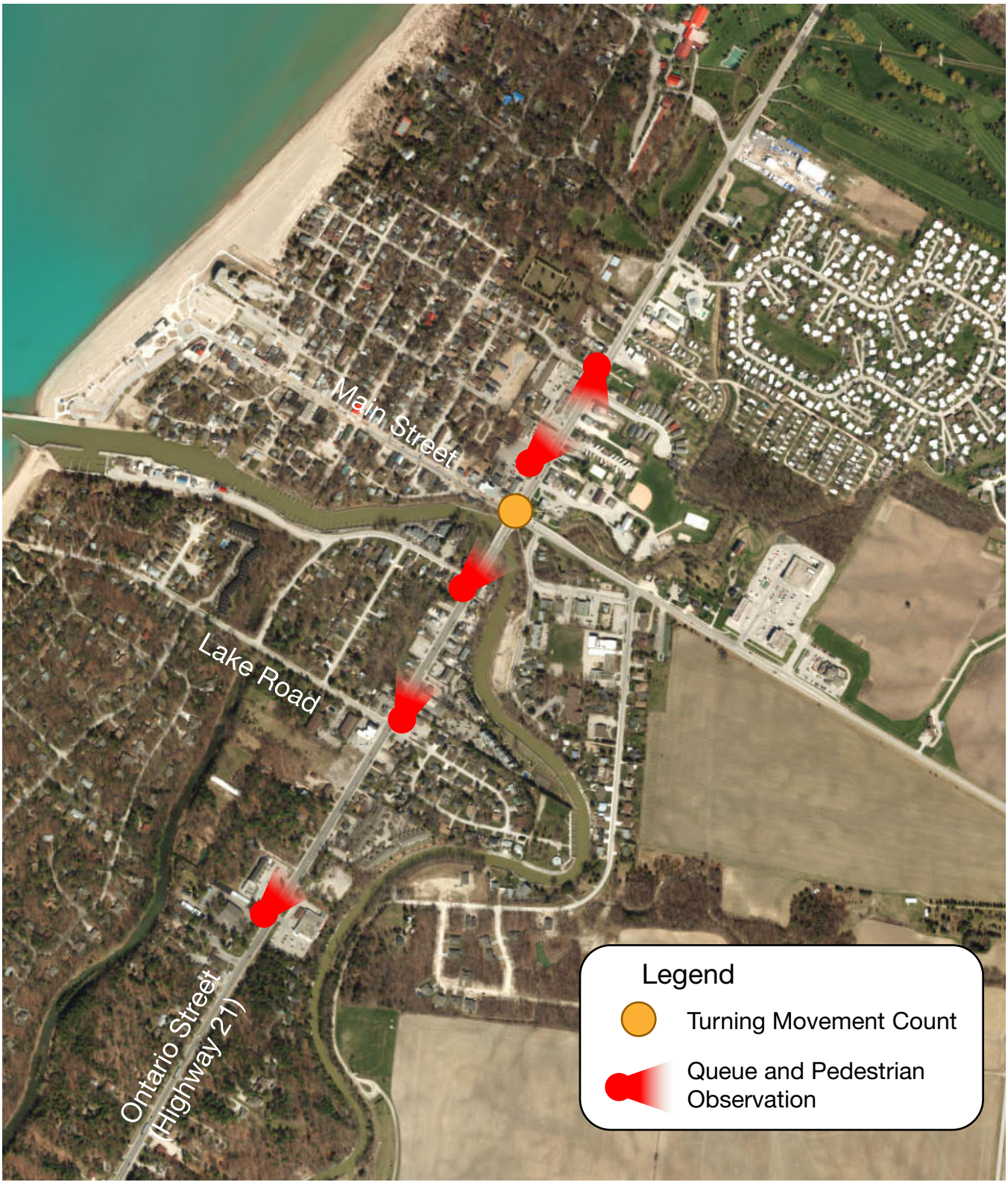
Figure 1



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Video Recording Locations

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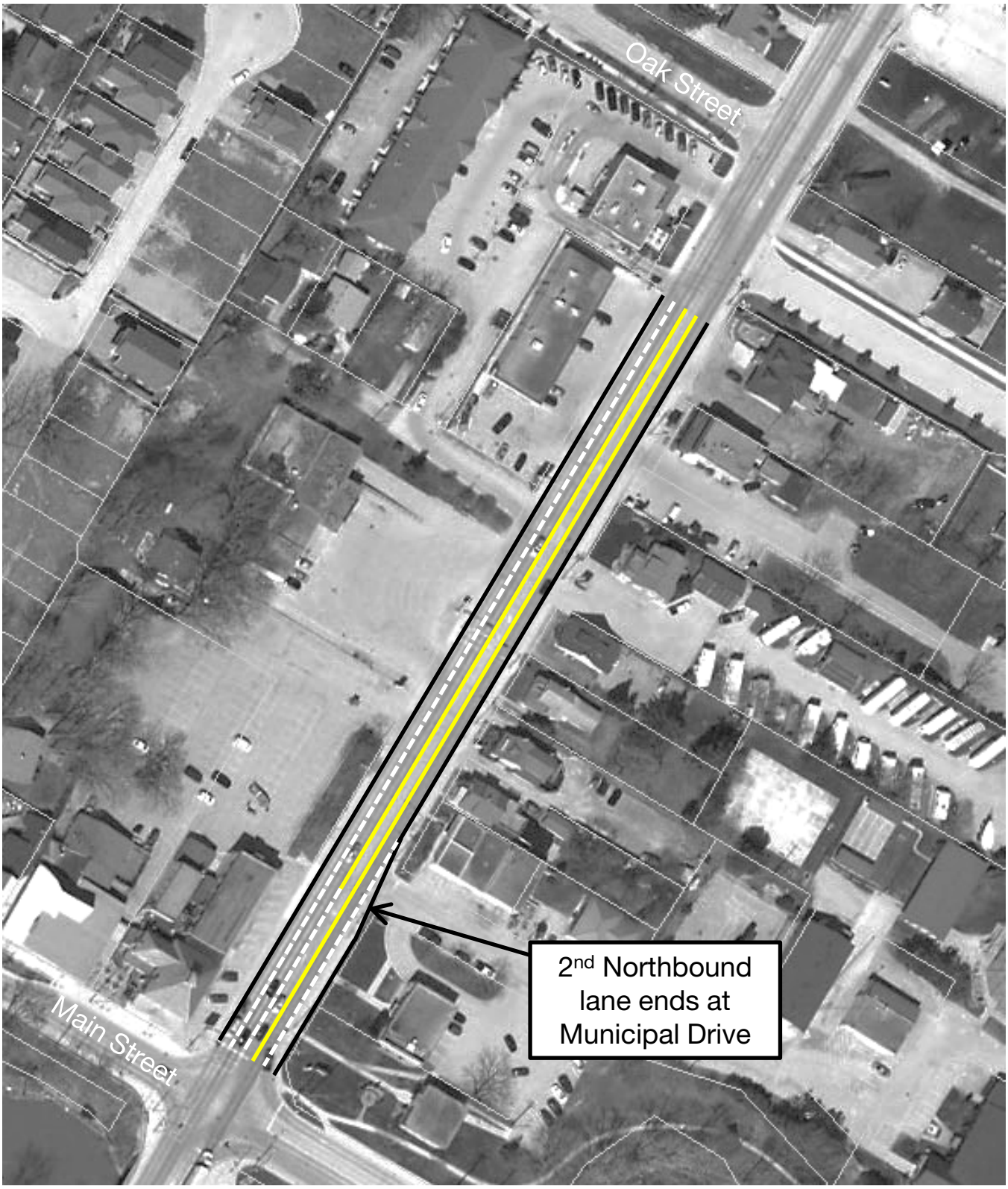
Figure 2



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Northbound Lane Concept 1

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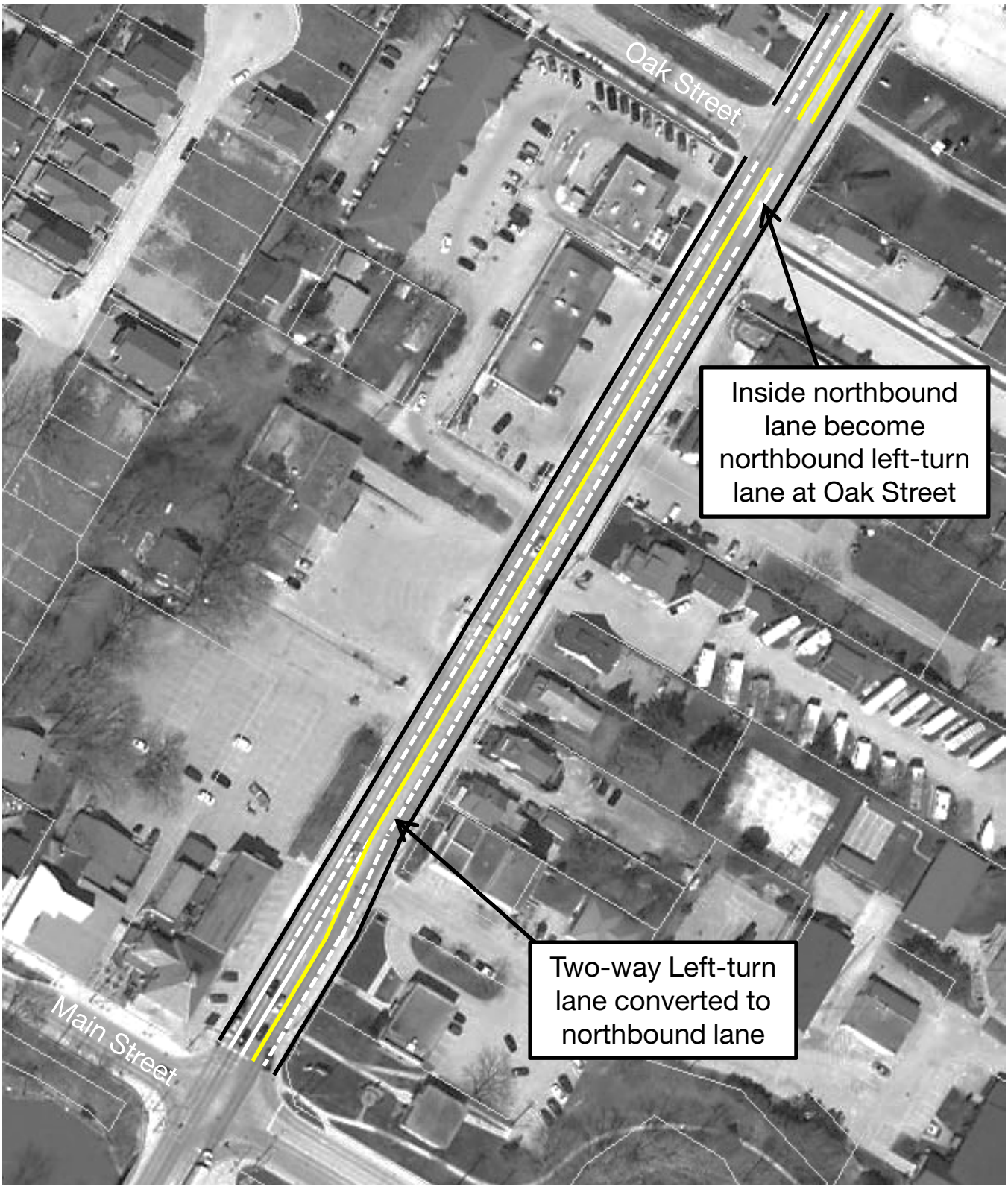
Figure 3



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Northbound Lane Concept 2a

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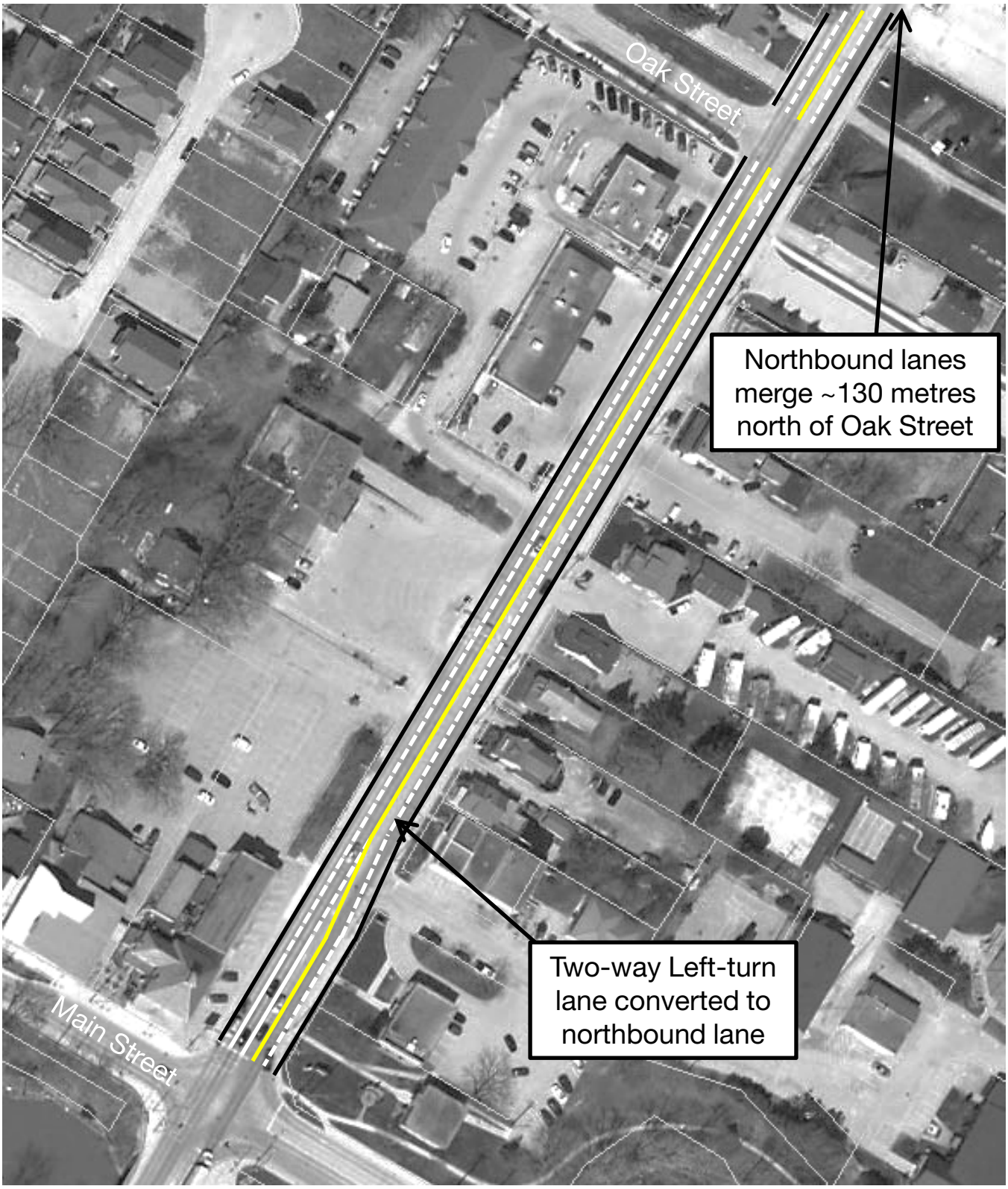
Figure 4



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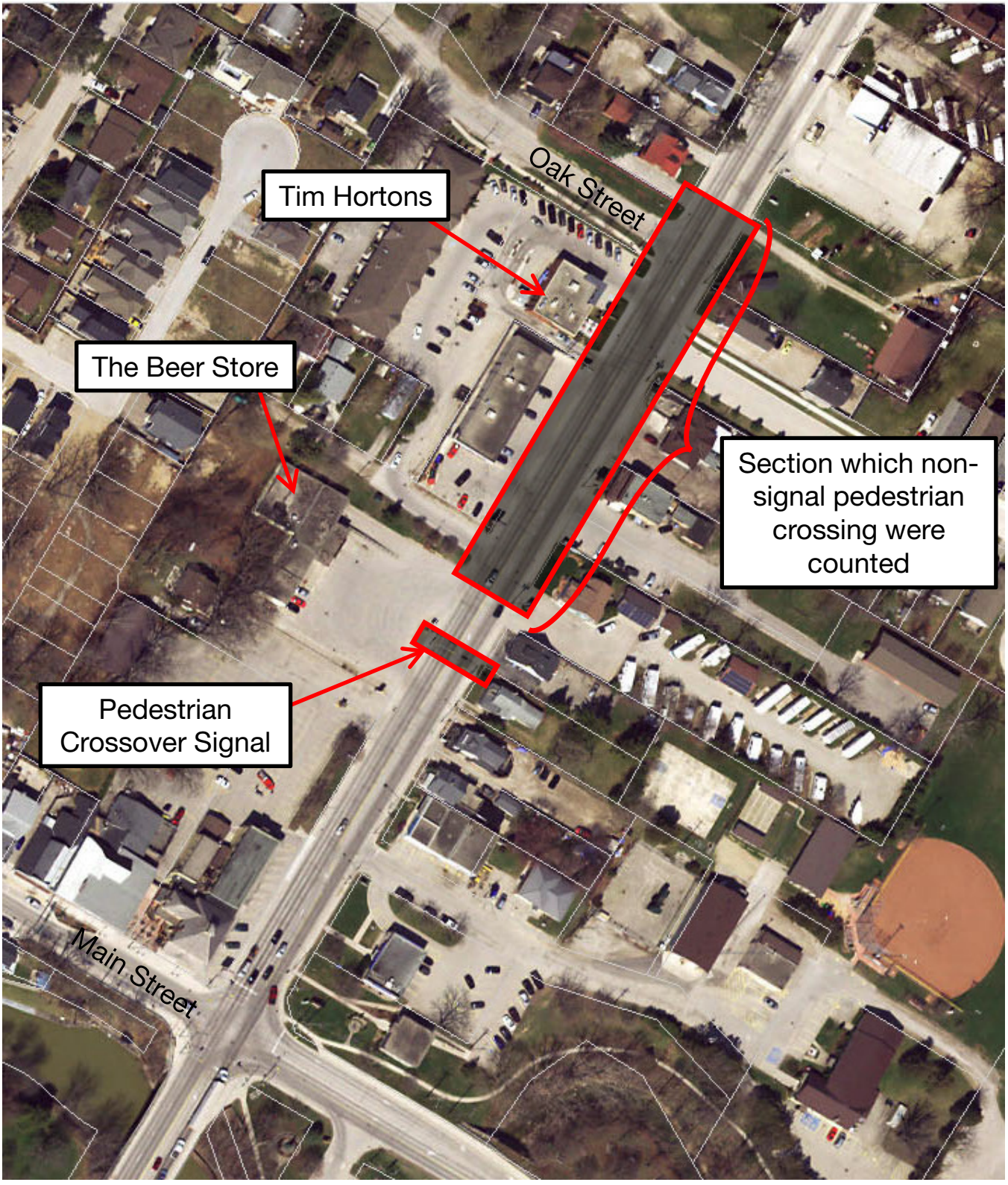
Northbound Lane Concept 2b



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Pedestrian Crossing Observation Locations