Transportation Impact Assessment – Step 4: Analysis

2095 Dilworth Road

Prepared for Dilworth Developments
by IBI Group
July 21, 2021

IBI GROUP
400–333 Preston Street
Ottawa ON K1S 5N4 Canada
tel 613 225 1311 fax 613 225 9868
ibigroup.com

Transmittal

To/Attention	Company/Address	Telephone No
Mike Giampa, P.Eng.	Project Manager, Senior Engineer Infrastructure Applications City of Ottawa 110 Laurier Avenue West (4th Floor) Ottawa, ON K2P-2H9	613-580-2424 x23567
Cc Gary Collauti	Dilworth Developments	

Delivery	E-mail/Electronic Submission
From	David Hook, P.Eng.
Sent By	David Hook
Date	July 21, 2021
Project No	134297
Subject	2095 Dilworth TIA - Step 4

Please find enclosed the TIA – Step 4 in support of a Zoning Bylaw Amendment application relating to a proposed gas station with convenience store, mini-warehouse self-storage facility, cross-dock warehouse and a commercial cardlock fuelling station to be located at 2095 Dilworth Road on behalf of Dilworth Developments. All comments and responses associated with this study have been documented and provided in Appendix A. Attached are the Synchro analysis files associated with this report.

If you require anything else, please don't hesitate to contact me at 613-225-1311 x64029 or by email at dhook@ibigroup.com.

Best Regards,

D Hook

David Hook, P.Eng.

TIA Plan Reports - Certification

On 14 June 2017, the Council of the City of Ottawa adopted new Transportation Impact Assessment (TIA) Guidelines. In adopting the guidelines, Council established a requirement for those preparing and delivering transportation impact assessments and reports to sign a letter of certification.

Individuals submitting TIA reports will be responsible for all aspects of development-related transportation assessment and reporting, and undertaking such work, in accordance and compliance with the City of Ottawa's Official Plan, the Transportation Master Plan and the Transportation Impact Assessment (2017) Guidelines.

By submitting the attached TIA report (and any associate documents) and signing this document, the individual acknowledges that s/he meets the four criteria listed below:

CERTIFICATION

1. I have reviewed and have a sound understanding of the objectives, needs and requirements of the City of Ottawa's Official Plan, Transportation Master Plan and the Transportation Impact Assessment (2017) Guidelines;

2. I have a sound knowledge of industry standard practice with respect to the preparation of transportation impact assessment reports, including multi modal level of service review;

3. I have substantial experience (more than 5 years) in undertaking and delivering transportation impact studies (analysis, reporting and geometric design) with strong background knowledge in transportation planning, engineering or traffic operations; and

4. I am either a licensed[1] or registered[1] professional in good standing, whose field of expertise [check √ appropriate field(s)] is either transportation engineering □ or transportation planning □.

[1] License or registration body that oversees the profession is required to have a code of conduct and ethics guidelines that will ensure appropriate conduct and representation for transportation planning and/or transportation engineering works.

Dated at Ottawa this 21st Day of July,2021.
(City)

Name: David Hook, P.Eng.

Professional Title: Associate - Manager, Transportation Engineering

Signature of Individual certifier that she/he meets the above four criteria

Office Contact Information (Please Print)
Address: 400-333 Preston Street
City / Postal Code: K1S 5N4
Telephone / Extension: 613-225-1311 ext. 64029
E-Mail Address: dhook@ibigroup.com

Stamp

Document Control Page

CLIENT:	Dilworth Developments
PROJECT NAME:	2095 Dilworth Road
REPORT TITLE:	Transportation Impact Assessment
IBI REFERENCE:	134297
VERSION:	Final
DIGITAL MASTER:	134297 2095 Dilworth Road - Internal Documents\6.0_Technical\ 6.23_Traffic\03_Tech-Reports
ORIGINATOR:	Ben Pascolo-Neveu/Ryan Ward
REVIEWER:	David Hook
AUTHORIZATION:	David Hook
CIRCULATION LIST:	Mike Giampa - City of Ottawa Transportation Project Manager
HISTORY:	1.0 – TIA Step 1 & 2 Submitted for City Review – May 10, 2021 2.0 – TIA Step 3 Submitted for City Review – June 14, 2021 3.0 – TIA Step 4 Submitted for City Review – July 21, 2021

Table of Contents

Table of Contents

Table of Contents

Table of Contents

List of Tables

Table of Contents

List of Figures

List of Exhibits

Table of Contents

List of Appendices

Executive Summary

IBI Group (IBI) was retained by Dilworth Developments to undertake a Transportation Impact Assessment (TIA) in support of a Zoning By-law Amendment application for a proposed gas station with convenience store, a mini-warehouse self-storage facility, a cross-dock warehouse and a commercial cardlock fuelling station to be located at 2095 Dilworth Road, Ottawa. The site will be accessed via two full-movement private approaches with direct connections to Dilworth Road.

In terms of build-out timing, the subject development is anticipated to be constructed and fully occupied in a single phase by 2022. In accordance with the TIA Guidelines, a 2027 horizon year was therefore applied, representing 5 years beyond the expected full build-out of the site.

There were no known developments of significance identified in the vicinity of the subject site that are either in the development application approval process, are in pre-construction or are in varying stages of construction. Given the proximity of the subject site to the Highway 416 Dilworth interchange, background growth was instead accounted for through the application of a 3% growth rate to remain consistency with historical trends on the segment of Veteran's Memorial Highway within the vicinity of the site.

Based on the trip generation undertaken for this study, the proposed development is expected to generate up to 157 and 148 new two-way vehicular trips during the weekday morning and afternoon peak hours, respectively. Pass-by traffic generated by the gas station use was also considered in the analysis, with up to 68 and 70 trips expected to occur during the weekday morning and afternoon peak hours, respectively. The mode share targets applied in this study were based on the Rural Southwest Traffic Assessment Zone (TAZ) and further refined to reflect the auto-oriented nature of the proposed development.

A segment-based multi-modal analysis identified deficiencies for sustainable modes on Dilworth Road adjacent to the site. It should be noted, however, that due to the rural context of the site and auto dependency of uses proposed, no improvements are required to safely accommodate the transportation demands of the subject development.

Based on the intersection capacity analyses conducted for this study, all four study area intersections are expected to operate at an acceptable level of service beyond the 2027 horizon year.

Queuing analysis conducted under Future (2027) Total Traffic conditions provided further indication that traffic operational issues are not expected to be a concern at any of the study area intersections within the timeframe of this study. Auxiliary left- or right-turn lanes at both existing ramp terminal intersections are expected to sufficiently accommodate future travel demands within the timeframe of this study. Further, the analysis did not identify the need for any auxiliary lanes to support site-generated traffic volumes on Dilworth Road at either proposed site access driveway.

As all study area intersections were shown to operate well within the capacity constraints of the adjacent transportation network, an RMA will not be required. Further, a post-development Monitoring Plan is also not a requirement of this study.

Based on the findings of this study, it is the overall opinion of IBI Group that the proposed development will integrate well with and can be safely accommodated by the adjacent transportation network.

1 Introduction

IBI Group (IBI) was retained by Dilworth Developments to undertake a Transportation Impact Assessment (TIA) in support of a Zoning By-law Amendment application for a proposed gas station with convenience store, a mini-warehouse self-storage facility, a cross-dock warehouse and a commercial cardlock fuelling station to be located at 2095 Dilworth Road, Ottawa.

In accordance with the City of Ottawa's Transportation Impact Assessment Guidelines, published in June 2017, the following report is divided into four major components:

- **Screening** – Prior to the commencement of a TIA, an initial assessment of the proposed development is undertaken to establish the need for a comprehensive review of the site based on three triggers: Trip Generation, Location and Safety.
- **Scoping** – This component of the TIA report describes both the existing and planned conditions in the vicinity of the development and defines study parameters such as the study area, analysis periods and analysis years of the development. It also provides an opportunity to identify any scope exemptions that would eliminate elements of scope described in the TIA Guidelines but not relevant to the development proposal, based on consultation with City staff.
- **Forecasting** – The Forecasting component of the TIA is intended to review both the development-generated travel demand and the background network travel demand. It also provides an opportunity to rationalize this demand to ensure projections are within the capacity constraints of the transportation network.
- **Analysis** – This component documents the results of any analyses undertaken to ensure that the transportation related features of the proposed development are in conformance with prescribed technical standards and that its impacts on the transportation network are both sustainable and effectively managed. It also identifies a development strategy to ensure that what is being proposed is aligned with the City of Ottawa's policies and city-building objectives.

Throughout the development of a TIA report, each of the four study components above are submitted in draft form to the City of Ottawa and undergo a review by a designated Transportation Project Manager. Any comments received are addressed to the satisfaction of the City's Transportation Project Manager before proceeding with subsequent components of the study. All technical comments and responses throughout this process are included in **Appendix A**.

Dependent on the findings of this report, the complete submission of this Transportation Impact Assessment may also require Functional Design Drawings of recommended roadway improvements to support a Roadway Modification Application (RMA). The submission may require a post-development Monitoring Plan to track performance of the planned TIA Strategy, however the need for a Monitoring Plan will be confirmed through the analysis undertaken in this report.

Due to the proximity of the proposed development to Highway 416, the Ontario Ministry of Transportation (MTO) will review the final TIA report, however the study will be carried out using the Ottawa TIA Guidelines, as described above.

2 TIA Screening

An initial screening was completed to confirm the need for a Transportation Impact Assessment by reviewing the following three triggers:

- **Trip Generation**: Preliminary trip generation estimates were developed based on the Institute of Transportation Engineers (ITE) Trip Generation Manual (10^{th} Edition). A 1.28 person-trip conversion factor was applied to the base trip generation data to obtain person-trip generation. The 60 person-trip threshold prescribed by the TIA Guidelines is exceeded during the weekday morning and afternoon peak hours, therefore the Trip Generation trigger is satisfied.
- **Location**: The proposed development will not be accessed from a boundary street that is designated as part of the City's Transit Priority, Rapid Transit network or Spine Bicycle Networks nor is the subject site within a Design Priority Area or Transit-Oriented Development zone, therefore, the Location trigger is not satisfied.
- **Safety**: Boundary street conditions were reviewed to determine if there is an elevated potential for safety concerns adjacent to the site. Given that Dilworth Road has a posted speed limit of 80km/h and that its vertical alignment may limit visibility and the proposed site access location, the Safety trigger is satisfied.

As the proposed development meets the Trip Generation trigger, the need to undertake a Transportation Impact Assessment is confirmed.

A copy of the Screening Form is provided in **Appendix B**.

3 Project Scoping

3.1 Description of Proposed Development

3.1.1 Site Location

The proposed development is located in rural south Ottawa and is bound by Dilworth Road to the south and Highway 416 to the west. A full interchange exists on Highway 416 adjacent to the site.

The site location and its surrounding context is illustrated in **Exhibit 1**.

IBI

2095 Dilworth Road
Transportation Impact Assessment

Exhibit 1:
Site Location

PROJECT No. 134297

SCALE: 0m 125m 250m

3.1.2 Land Use Details

The concept plan for this site includes a gas station with convenience store, a 20 cross-dock warehouse, a mini-warehouse self-storage facility and a commercial cardlock fuelling station. The primary function of this site is to serve as a commercial refuelling station for goods movements in and out of the Ottawa region.

Table 1 below summarizes the proposed land uses statistics for this development.

Table 1 - Land Use Statistics

LAND USE	SIZE
Gas Station with Convenience Market	8 fuelling positions
Warehousing	~8,361 m^2 (90,000 ft^2)
Mini-Warehouse Self-Storage Facility	~1,394 m^2 (15,000 ft^2)
Commercial Cardlock Fuelling Station	8 fuelling positions

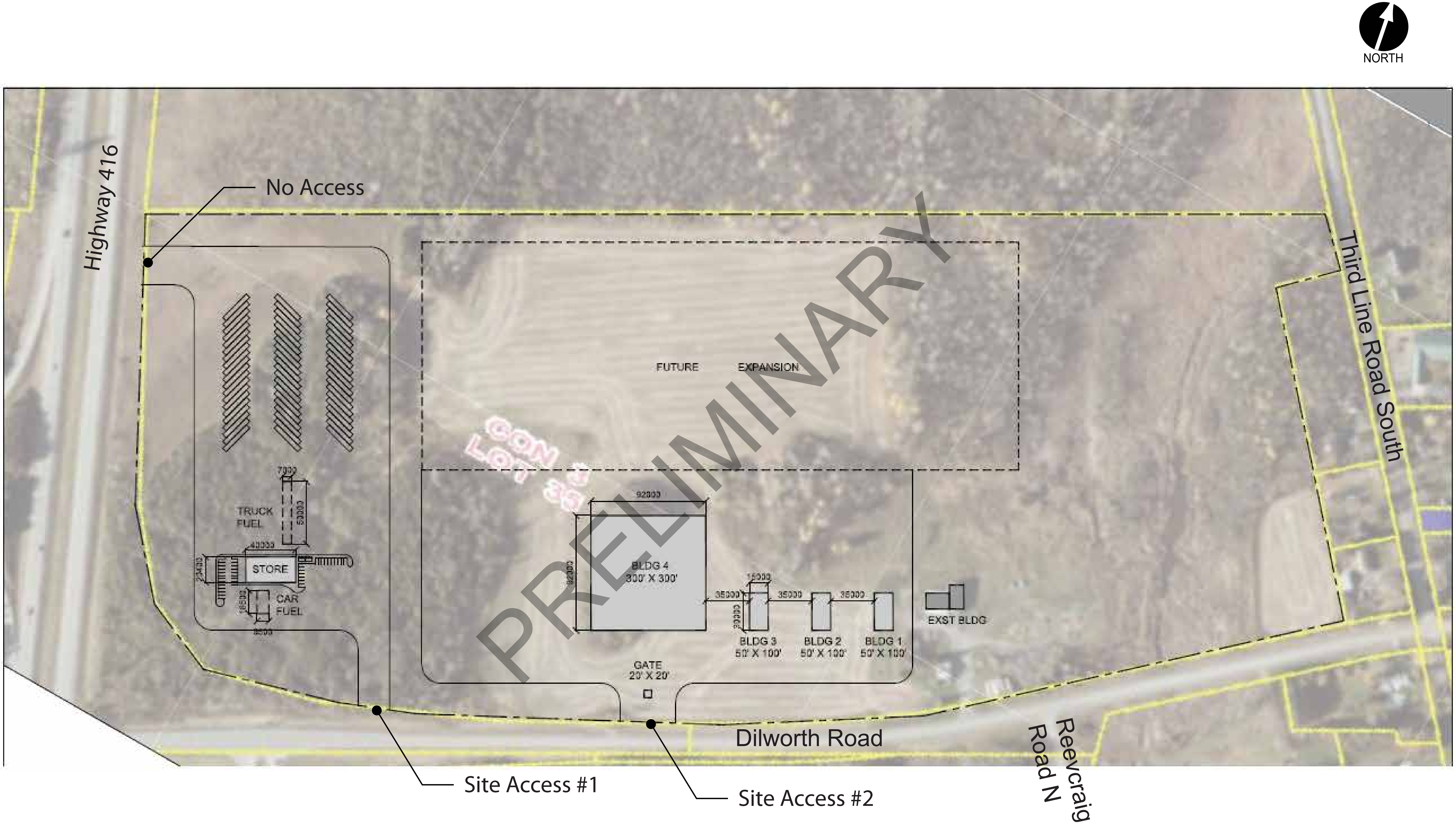
The proposed development is illustrated in **Exhibit 2** below.

The site will be accessed via two full-movement private approaches with direct connections to Dilworth Road.

3.1.3 Development Phasing & Date of Occupancy

The proposed development will be constructed in a single phase. It is anticipated that the development will be constructed and fully occupied by 2022.

It should be noted that the conceptual site plan identifies a ‘Future Expansion Phase’ which is not included in the scope of this TIA.



2095 Dilworth Road
Transportation Impact Assessment

Exhibit 2: Proposed Development

PROJECT No. 134297

SCALE: 0m 40m 80m

3.2 Existing Conditions

3.2.1 Existing Road Network

3.2.1.1 Roadways

The proposed development is bound by the following street(s):

- **Highway 416** is a four-lane, divided highway under the jurisdiction of the Ontario Ministry of Transportation with a right-of-way protection of approximately 30 metres and a posted speed limit of 100 km/h.
- **Dilworth Road** is a two-lane rural arterial road extending east-west from McCordick Road to Rideau Valley Drive. Within the context area, the road has an approximate 30m right-of-way, a posted speed limit of 80 km/h and is identified as a Truck Route in the TMP.

Other streets within the vicinity of the proposed development are as follows:

- **Third Line Road South** is a two-lane rural road extending from Phelan Road in the north to approximately 280 metres south of Dilworth Road. Within the context area, this road is classified as a 'collector' north of Dilworth Road and a local road further south. Third Line Road South has an approximate 26-metre right-of-way and an unposted speed limit of 50 km/h.
- **Reevecraig Road North** is a two-lane, local road with an unposted speed limit of 50 km/h within the vicinity of the subject lands and a right-of-way protection of approximately 20 metres.
- **Fourth Line Road** is two lane arterial road with a posted speed limit of 80km/h and a right-of-way protection of approximately 30 metres.

3.2.1.2 Intersections

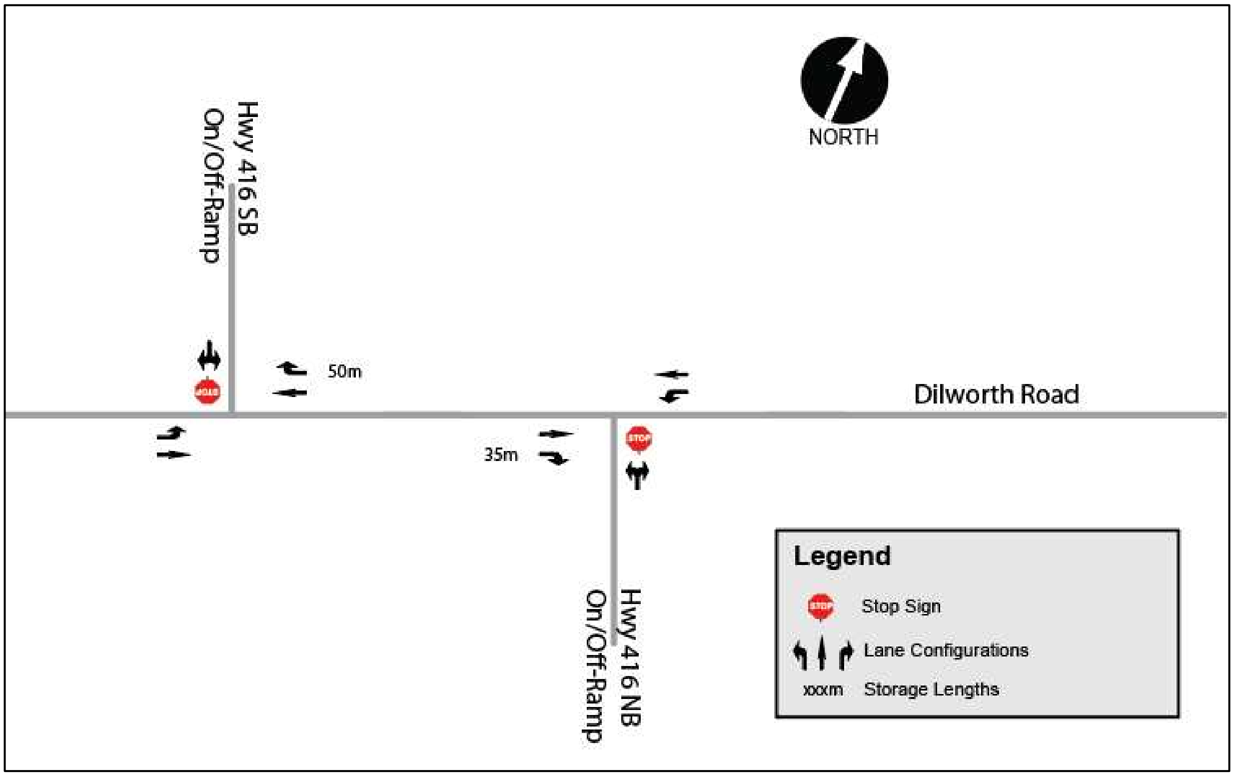
The following existing intersections have been identified as having the greatest potential to be impacted by the proposed development:

Highway 416 Northbound On/Off-Ramp & Dilworth Road is a three-legged, unsignalized intersection with stop-control on the ramp terminal approach. On Dilworth Road, a right-turn auxiliary lane exists on the eastbound approach, while a slip-around lane exists on the westbound direction to segregate through and left-turning vehicles.

Highway 416 Southbound On/Off-Ramp & Dilworth Road is a three-legged, unsignalized intersection with stop-control on the ramp terminal approach. On Dilworth Road, a right-turn auxiliary lane exists on the westbound approach, while a slip-around lane exists on the eastbound approach to segregate through and left-turning vehicles.

The existing lane configurations and intersection control are illustrated in **Figure 1** below.

Figure 1 – Existing Lane Configurations & Intersection Control



3.2.1.3 Traffic Management Measures

There are currently no traffic management or traffic calming measures on the boundary streets within the vicinity of the proposed development.

3.2.1.4 Nearby Driveways

There are currently no driveways within 200 metres of the proposed site access driveway. The adjacent snowmobile dealer to the east has a private approach, with the nearest being approximately 240 metres from the proposed site access.

3.2.1.5 Existing Traffic Volumes

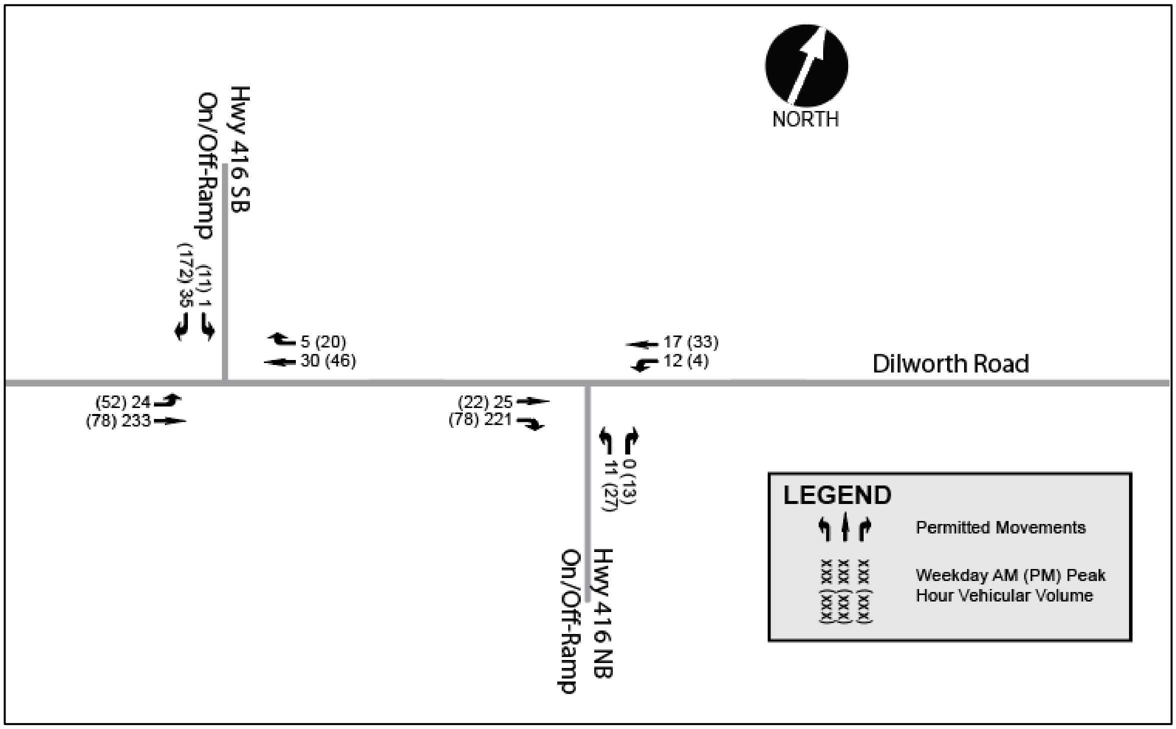
Weekday morning and afternoon peak hour turning movement counts from the Ministry of Transportation were obtained at the following ramp terminal locations:

- Highway 416 NB On/Off Ramp & Dilworth Road (MTO – May 24, 2018)
- Highway 416 SB On/Off Ramp & Dilworth Road (MTO – September 12, 2013)

It is recognized that the above noted counts were conducted more than 3 years ago, however as a result of the ongoing COVID-19 pandemic and Stay-At-Home order restrictions, it was not possible to collect data representative of typical conditions at either location. As such, the application of growth rate was used applied to these counts to approximate existing (2021) traffic volumes. Justification of background traffic volumes is discussed further in the Forecasting section of this report.

Peak hour vehicle volumes representative of existing (2021) conditions are shown in **Figure 2**. Traffic count data is provided in **Appendix C**.

Figure 2 – Existing (2021) Traffic



3.2.2 Existing Bicycle and Pedestrian Facilities

A desktop review of the context area indicates that no formal bicycle or pedestrian facilities presently exist within the vicinity of the proposed development.

3.2.3 Existing Transit Facilities and Service

There are no transit facilities in the vicinity of the proposed development.

3.2.4 Collision History

A review of historical collision data is typically conducted for the road network surrounding the proposed development for the most recent 5 years of collision data available. The TIA Guidelines require a safety review if at least six collisions for any one movement or of a discernible pattern, over a five-year period have occurred.

Through correspondence with City of Ottawa technical staff, it was determined that there have been no reported collisions within the context area of the proposed development. Further, it was confirmed by MTO that a collision/ safety analysis would not be required as part of this study.

3.3 Planned Conditions

3.3.1 Transportation Network

3.3.1.1 Future Road Network Projects

The 2013 Transportation Master Plan (TMP) '2031 Affordable Network' or '2031 Network Concept Plan' does not identify any planned road network modifications within the context area.

3.3.1.2 Future Transit Facilities and Services

Due to its rural context, the TMP does not identify any planned Rapid Transit or Transit Priority (RTTP) projects within the vicinity of the proposed development as part of the '2031 Affordable Network' or '2031 Network Concept'.

3.3.1.3 Future Cycling and Pedestrian Facilities

It is not anticipated that any additional pedestrian and cycling facilities will be implemented within the vicinity of the proposed development.

3.3.2 Future Adjacent Developments

The City of Ottawa Transportation Impact Assessment (TIA) Guidelines specify that all significant developments proposed within the surrounding area which are likely to occur within the study's horizon year must be identified and taken into consideration in the development of future background traffic projections. A review of the City's development application data, DevApps, indicates that there are presently no adjacent developments within the context area.

3.3.3 Network Concept Screenline

Not Applicable: Network screenline analysis is not expected to be necessary for this development, as it does not trigger the threshold prescribed in the TIA Guidelines of 200 person-trips beyond what is otherwise permitted by the current zoning. Detailed trip generation calculations will be provided in the Forecasting section of the report.

3.4 Study Area

The information presented thus far provides a base level of information for the development's context. Based on preliminary trip generation estimates, the proposed development is expected to generate approximately 241 person-trips during the weekday morning and afternoon peak hours. Given the site's proximity to the Highway 416 Dilworth interchange, the vast majority of site-generated trips will access the site via this interchange. As such, minimal downstream impacts east of the proposed site access at the intersections of Dilworth with Reevecraig or Third Line are anticipated.

A condensed study area is therefore proposed for this TIA, consisting of the following intersections:

- Dilworth Road & Highway 416 Northbound On/Off-Ramp
- Dilworth Road & Highway 416 Southbound On/Off-Ramp
- Dilworth Road & Site Access #1 (proposed)
- Dilworth Road & Site Access #2 (proposed)

The remainder of this TIA will focus on site-specific impacts, integration with its boundary streets, including a functional review of the site access geometry and intersection control, on-site drive aisle requirements to accommodate proposed design vehicles and a review of the site's parking and loading requirements.

An intersection Multi-Modal Level of Service (MMLOS) analysis is only required for signalized intersections and based on the relatively low impact expected for the proposed development, it is not anticipated that the need for traffic signals will be triggered at either of the study area intersections. This will be verified through intersection capacity analysis in the Analysis component of the study. Segment-based MMLOS analysis will be limited to Dilworth Road along the subject site's frontage.

3.5 Time Periods

Based on a preliminary review of trip generation rates associated with the proposed land uses, traffic generated during the weekday morning and afternoon peak hour is expected to result in the most significant impact to traffic operations on the adjacent road network. These two time periods will constitute the critical analysis periods for this study.

3.6 Study Horizon Year

Traffic analyses associated with TIA's typically involve a review of existing conditions, as well as the anticipated future conditions, both with- and without the proposed development, at the year of full-occupancy as well as five years beyond. Phased developments will often require interim analyses to provide a timeline for any necessary transportation infrastructure improvements.

For the purpose of this study, it is expected that the proposed development will be constructed and fully occupied in a single phase in 2022. The horizon year for this study is therefore 2027.

3.7 Exemptions Review

The TIA Guidelines provide exemption considerations for elements of the Design Review and Network Impact components. **Table 2** summarizes the TIA modules that are not applicable to this study.

Table 2 - Exemptions Review

TIA MODULE	ELEMENT	EXEMPTION CONISDERATIONS	REQUIRED
DESIGN REVIEW COMPONENT			
4.1 Development Design	4.1.2 Circulation and Access	• Only required for site plans	✓
	4.1.3 New Street Networks	• Only required for plans of subdivision	✗
4.2 Parking	4.2.1 Parking Supply	• Only required for site plans	✓
	4.2.2 Spillover Parking	• Only required for site plans where parking supply is 15% below unconstrained demand	✗
NETWORK IMPACT COMPONENT			
4.5 Transportation Demand Management	All Elements	• Not required for site plans expected to have fewer than 60 employees and/or students on location at any given time	✗
4.6 Neighbourhood Traffic Management	4.6.1 Adjacent Neighbourhoods	• Only required when the development relies on local or collector streets for access and total volumes exceed ATM capacity thresholds	✗
4.8 Network Concept	n/a	• Only required when proposed development generates more than 200 person-trips during the peak hour in excess of the equivalent volume permitted by established zoning	✗

4 Forecasting

4.1 Development Generated Traffic

4.1.1 Trip Generation Methodology

Peak hour site-generated traffic volumes were developed using the Institute of Transportation Engineers' (ITE) Trip Generation Manual (10th Edition). The TIA Guidelines indicate that vehicle-trip generation rates from the ITE Trip Generation Manual should be converted to person-trips through the application of a 1.28 vehicle-to-person-trip conversion factor.

Following the application of the vehicle-to-person-trip conversion factor, the person-trips were then subdivided based on representative mode share percentages applicable to the study area to determine the number of auto driver, auto passenger, transit, pedestrian, cycling and 'other' trip types.

Mode share targets were developed based on a review of local mode share distributions from the 2011 Origin-Destination Survey and further refined to reflect the site context.

4.1.2 Trip Generation Results

4.1.2.1 Base Vehicle Trip Generation

Peak hour vehicular traffic volumes associated with the proposed development were determined using appropriate peak hour trip generation rates from the ITE Trip Generation Manual.

The baseline vehicular trip generation for the Commercial Cardlock Gas Station is not represented by any ITE land uses, therefore trip generation rates derived for a similar facility analysed as part of the Northwest Paradise Park Road development Portland, Oregon was applied.[1]

In accordance with the TIA Guidelines, a heavy vehicle factor of 1.7 was applied to the Commercial Cardlock Fuelling Station and Warehousing trip generation to convert truck trips to Passenger Car Equivalent (PCE) vehicles.

The vehicular trip generation results have been summarized in **Table 3** below.

Relevant extracts relating to trip generation data are provided in **Appendix D**.

[1] Source (page 2): https://www.ci.lacenter.wa.us/city_departments/pdfs/K%20Trip%20Generation%20Letter.pdf

Table 3 - Base Vehicular Trip Generation Results

LAND USE	SIZE	PERIOD	GENERATED TRIPS (VPH)		
			IN	OUT	TOTAL
Gasoline/Service Station w/ Convenience Market (ITE Code 945)	8 fuelling positions	AM	51	49	100
		PM	57	55	112
Warehousing (ITE Code 150) [1]	~8,361 m^2 (90,000 ft^2)	AM	13	13	26
		PM	15	15	30
Mini-Warehouse (ITE Code 151)	~1,394 m^2 (15,000 ft^2)	AM	1	1	2
		PM	1	2	3
Commercial Cardlock Fuelling Station [1,2]	8 fuelling positions	AM	30	30	60
		PM	20	20	40

Notes: vph = vehicles per hour

[1] Trip generation rates were increased by heavy vehicle factor of 1.7 in accordance with the TIA Guidelines

[2] Source (page 2): *https://www.ci.lacenter.wa.us/city_departments/pdfs/K%20Trip%20Generation%20Letter.pdf*

4.1.2.2 Person Trip Generation

As mentioned previously, the TIA Guidelines indicate that a 1.28 vehicle-to-person-trip conversion rate should be applied to convert the base ITE vehicular trip generation results into person trips. For consistency, the same conversion factor was also applied to the Commercial Cardlock Fuelling Station baseline trip generation data.

The resulting number of site-generated person-trips is summarized in **Table 3** below.

Table 3 - Person-Trip Generation

LAND USE	PERSON-TRIP CONVERSION FACTOR	PERIOD	PERSON TRIPS (PPH)		
			IN	OUT	TOTAL
Gasoline/Service Station with Convenience Market	1.28	AM	65	63	128
	1.28	PM	73	70	143
Warehousing	1.28	AM	17	17	34
	1.28	PM	19	19	38
Mini-Warehouse Self-Storage Facility	1.28	AM	1	1	2
	1.28	PM	1	2	3
Commercial Cardlock Fuelling Station	1.28	AM	39	39	78
	1.28	PM	26	26	52
		AM Total	**122**	**119**	**241**
		PM Total	**118**	**117**	**235**

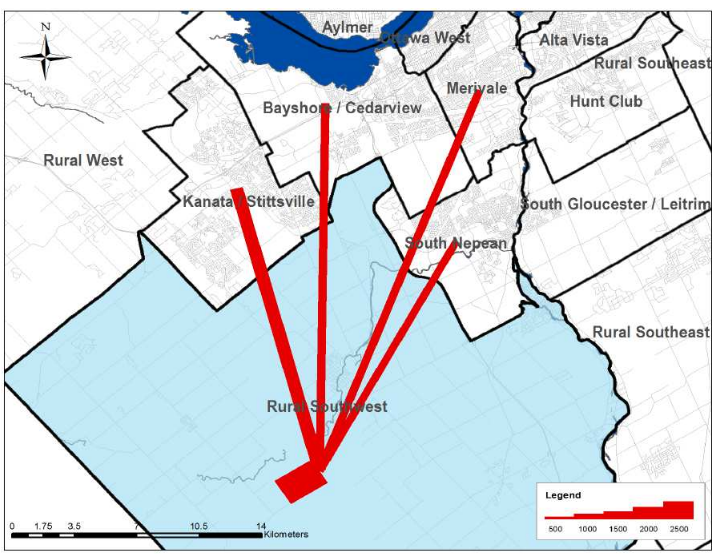
Notes: pph = persons per hour

4.1.2.3 Mode Share Proportions

The 2011 TRANS Origin-Destination (O-D) Survey provides approximations of the existing modal share within the Rural Southwest Traffic Assessment Zone (TAZ). The extents of the Rural Southwest TAZ are illustrated in **Figure 3** below.

Relevant extracts from the 2011 O-D Survey are provided in **Appendix D**.

Figure 3 – Rural Southwest TAZ



Source: 2011 O-D Survey

A blended mode share for the proposed development was derived based on weighted averages of mode share distributions from the weekday morning and afternoon peak periods of the Rural Southwest TAZ and further refined to better represent realistic mode share targets for the proposed development. Given the rural context of the site and the nature of the uses proposed within the subject development, any sustainable or 'other' mode share allocations were redistributed proportionally to 'auto driver'.

Table 4 below summarizes the existing 2011 O-D Survey mode share distribution and proposed mode share targets.

Table 4 - 2011 O-D Survey Mode Share Distributions and Proposed Mode Share Targets

MODE	EXISTING MODE SHARE WITHIN TAZ				BLENDED MODE SHARE [1]	MODE SHARE TARGETS [2]
	AM TO	AM WITHIN	PM FROM	PM WITHIN		
Auto Driver	61%	38%	73%	49%	56%	87%
Auto Passenger	9%	10%	17%	15%	13%	13%
Transit	7%	0%	6%	1%	4%	0%
Cycling	1%	1%	1%	2%	1%	0%
Walking	1%	4%	0%	10%	3%	0%
Other	21%	47%	4%	23%	23%	0%

Notes:

[1] – Weighted average of the 'AM To', 'AM Within', 'PM From' and 'PM Within' mode share distributions.

[2] – The sustainable and 'other' mode share percentages derived from the weighted average were distributed proportionally to the 'auto driver' and 'auto passenger' modes.

4.1.2.4 Trip Reduction Factors

Deduction of Existing Development Trips

Not Applicable: The proposed development lands are currently undeveloped, and do not generate any traffic volumes.

Pass-by Traffic

Based on survey data collected for the *ITE Trip Generation Handbook (3rd edition),* the gas station with convenience market land use was shown to generate an average of 62% and 56% pass-by trips during the weekday morning and afternoon peak periods, respectively. As such, these pass-by rates were applied in the development of site-generated traffic volumes.

It is assumed that the self-storage facility will not generate pass-by traffic.

Synergy/ Internalization

Synergy or internalization is typically applied to developments with two or more land uses to prevent double-counting of trips with multiple intermediate destinations within the same site. With respect to this site, the interaction between the self-storage and gas station uses as the primary trip purpose is not expected to be significant. As such, no internalization has been considered in the analysis.

4.1.2.5 Trip Generation by Mode

The mode share targets from **Table 4** were applied to the number of development-generated person-trips to establish the expected number of trips per travel mode, as summarized in **Table 5** below. Any mode share targets with a 0% allocation were excluded. It should be noted that commercial trucks typically do not have passengers, therefore for the 'Warehousing' and 'Commercial Cardlock Fuelling Station' land uses 100% of the mode share is allocated to auto driver.

Table 5 - Peak Hour Person Trips by Mode

MODE	AM			PM		
	IN	OUT	TOTAL	IN	OUT	TOTAL
GAS STATION W/ CONVENIENCE MARKET						
Persons Trips	65	63	128	73	70	143
Auto Driver (87%)	57	54	111	64	61	125
Auto Passenger (13%)	8	8	17	9	9	19
Pass-by Trips[1]	34	34	68	35	35	70
New Auto Trips	**23**	**20**	**43**	**29**	**26**	**55**
WAREHOUSING						
Persons Trips	17	17	34	19	19	38
Auto Driver (100%)	17	17	34	19	19	38
New Auto Trips	**17**	**17**	**34**	**19**	**19**	**38**
MINI-WAREHOUSE SELF-STORAGE FACILITY						
Persons Trips	1	1	2	1	2	3
Auto Driver (87%)	1	1	2	1	2	3
Auto Passenger (13%)	0	0	0	0	0	0
New Auto Trips	**1**	**1**	**2**	**1**	**2**	**3**
COMMERCIAL CARDLOCK FUELLING STATION						
Persons Trips	39	39	78	26	26	52
Auto Driver (100%)	39	39	78	26	26	52
New Auto Trips	**39**	**39**	**78**	**26**	**26**	**52**
TOTAL NEW AUTO TRIPS	**80**	**77**	**157**	**75**	**73**	**148**

Notes: [1] AM Pass-by rate is 62%; PM Pass-by rate is 56%

Based on the results provided in **Table 5** above, it is anticipated that the proposed development will generate up to 157 and 148 new two-way vehicular trips during the weekday morning and afternoon peak hours, respectively.

4.1.3 Trip Distribution and Assignment

As the proposed development is expected to primarily generate traffic from Highway 416 via the Dilworth interchange, new site-generated auto trips have been distributed to the adjacent road network based on a comparison of ramp volume data reviewed provided for this study.

Distribution for New Auto Trips

- 85% to/from the North
    - 100% on Highway 416
- 15% to/from the South
    - 100% on Highway 416

Alternative distributions were derived to reflect the expected travel patterns of pass-by trips associated with the proposed gas station and convenience market land use, as shown in **Table 6** below.

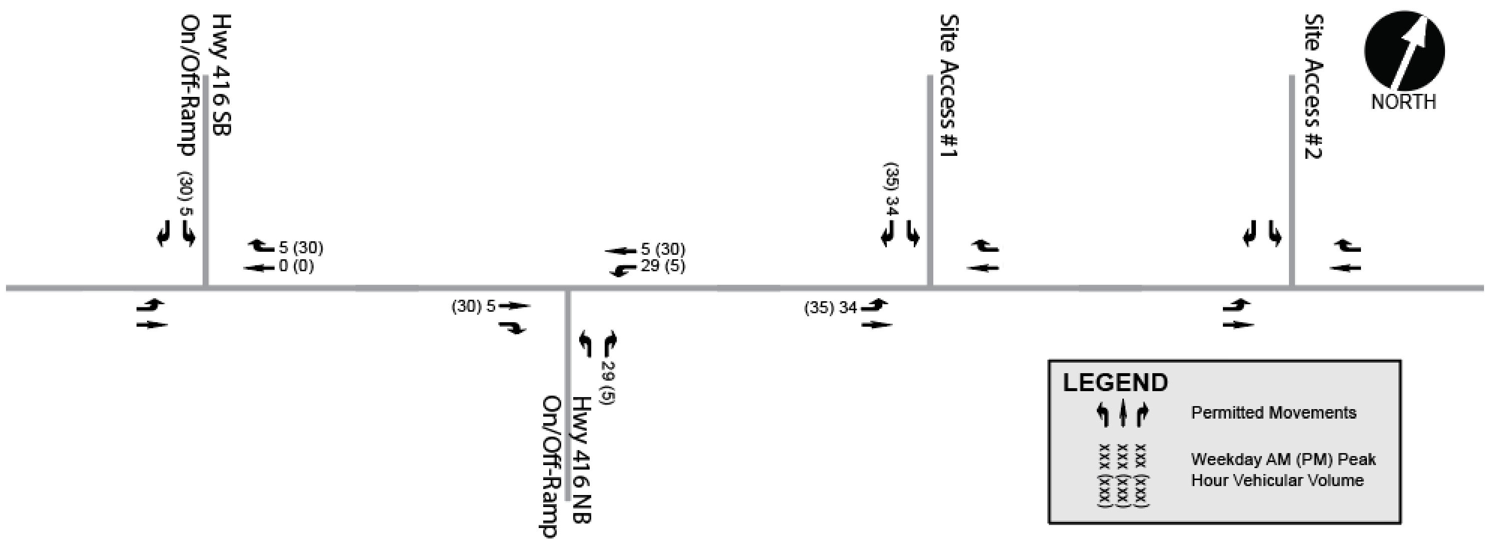
Table 6 - Distributions for Pass-by Trips

CARDINAL DIRECTION	AM	PM
Northbound	85% on Highway 416	15% on Highway 416
Southbound	15% on Highway 416	85% on Highway 416

Utilizing the estimated number of new auto trips and pass-by trips and applying the corresponding distributions at each study area intersection, the resulting traffic volumes are illustrated in **Figure 4** and **Figure 5**, respectively.

Figure 4 - Site Generated Traffic (New Auto Trips)

Figure 5 - Site Generated Traffic (Pass-by Trips)



4.2 Background Network Traffic

4.2.1 Changes to the Background Transportation Network

To properly assess future traffic conditions, planned modifications to the transportation network that may impact travel patterns or demand within the study area must be considered. The Scoping section of this TIA reviewed the anticipated changes to the study area transportation network based on the Transportation Master Plan (TMP), the Ottawa Cycling Plan, the Ottawa Pedestrian Plan, as well as the 2019 City-Wide Development Charges Background Study and determined that there are no network modifications planned within the timeframe of this study.

4.2.2 General Background Growth Rates

The background growth rate is intended to represent regional growth from outside the study area that will travel along the adjacent road network.

A review of historical Average Annual Daily Traffic (AADT) collected by the Ontario Ministry of Transportation (MTO) on Highway 416 near the Dilworth Road interchange indicates that this segment of the Veteran's Memorial Highway has experienced overall growth in the order of 3% per annum. As such, this growth rate has been applied to all movements at the Dilworth Road ramp terminal intersections, while its application has been limited to through movements at the proposed site access driveways.

Relevant extracts from MTO historical traffic data are provided in **Appendix C**.

4.2.3 Other Area Development

As discussed previously in the Scoping component of this study, there are presently no development applications of significance within the context area of the subject development.

4.3 Demand Rationalization

The purpose of this section is to rationalize future travel demands within the study area to account for potential capacity limitations in the transportation network and its ability to effectively accommodate the additional demand generated by a new development.

4.3.1 Description of Capacity Issues

It is generally accepted that the capacity of an arterial road is 1,000 vehicles per hour per lane (vphpl). Traffic count data collected by the City indicates that peak hour volumes on Dilworth Road are presently in the order of 220 to 260 vehicles per hour in the peak direction which is well within the capacity limitations for a two-lane arterial roadway. Based on this preliminary capacity review, it expected that any additional traffic resulting from development-generated and background network demands will not result in the exceedance of the arterial threshold. The Analysis section of this TIA will confirm any traffic operational issues at the study area intersections under both background and total traffic conditions and suggest mitigation measures where applicable.

4.3.2 Adjustment to Development Generated Demands

Recognizing the lack of documented capacity issues at any of the study area intersections, no adjustments have been made to future background traffic volumes.

4.3.3 Adjustment to Background Network Demands

As prescribed in the TIA Guidelines, the effects of peak-hour spreading have been considered in in future analysis years of this study. It is anticipated that as traffic volumes continue to gradually increase, vehicular trips will have a natural tendency to be more evenly distributed across the peak hour (PHF = 1.0) and eventually increase demands in the shoulders of the peak as well. The impacts of peak hour spreading are accounted for in the Synchro modelling, completed as part of the Analysis component of this study.

As no specific capacity issues have been identified through previous studies, no further adjustments to background network demands are necessary.

4.4 Traffic Volume Summary

4.4.1 Future Background Traffic Volumes

Future background traffic volumes have been established through the application of a growth rate to the Existing (2021) Traffic, as discussed previously.

Figure 6 and **Figure 7** present the future background traffic volumes anticipated for the 2022 build-out year, as well as the 2027 study horizon, respectively.

4.4.2 Future Total Traffic Volumes

Future total volumes have been derived by superimposing the new site-generated auto trips from **Figure 4** and the pass-by trips from **Figure 5** onto the future background volumes presented in **Figure 6** and **Figure 7**.

Figure 8 and **Figure 9** present the future total traffic volumes anticipated for 2022 and 2027 analysis years, respectively.

Figure 6 - Future (2022) Background Traffic

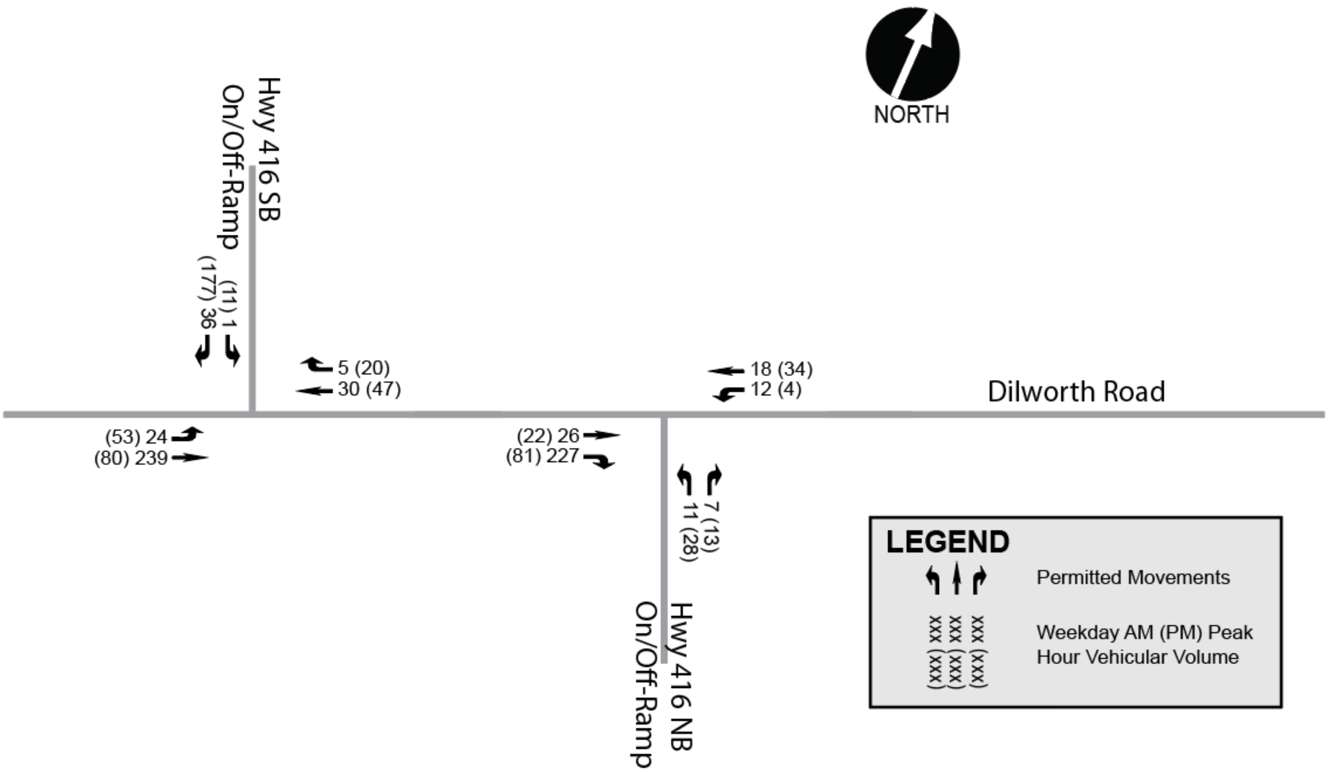


Figure 7 - Future (2027) Background Traffic

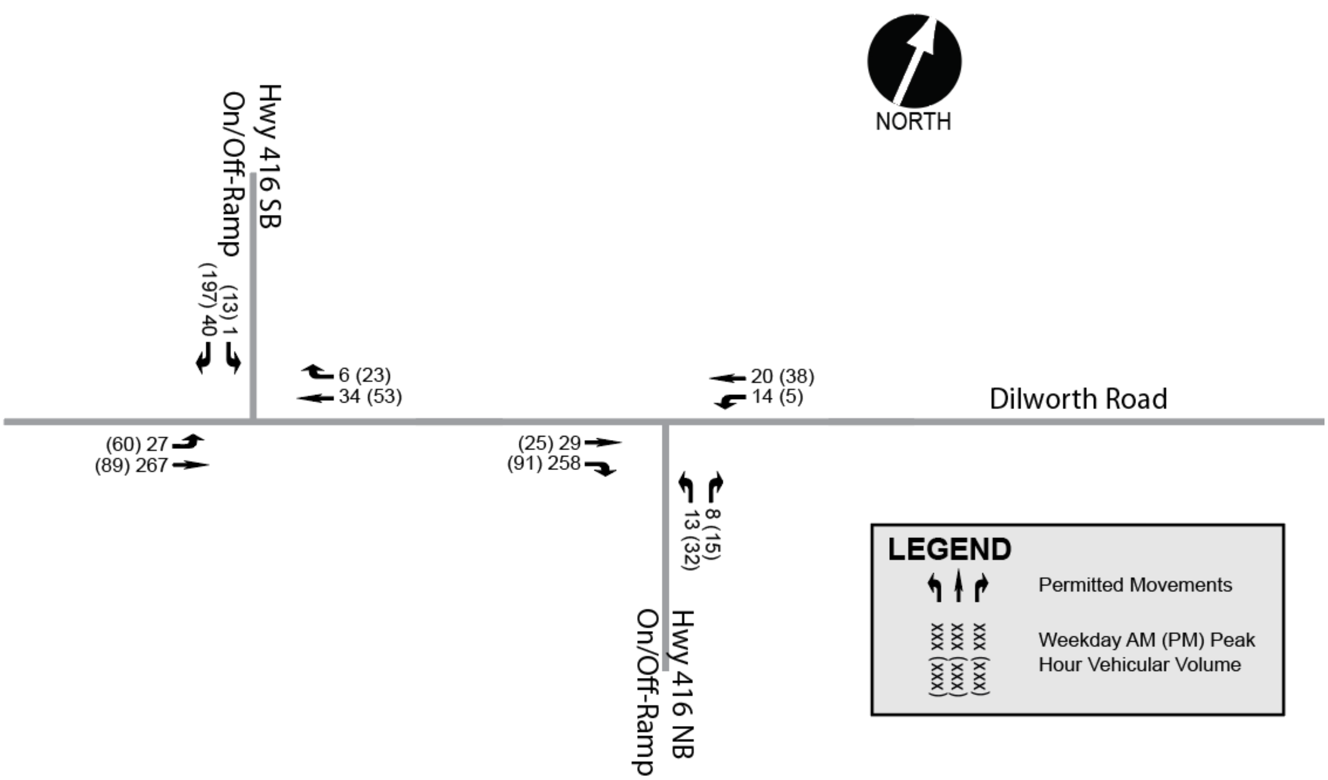


Figure 8 - Future (2022) Total Traffic

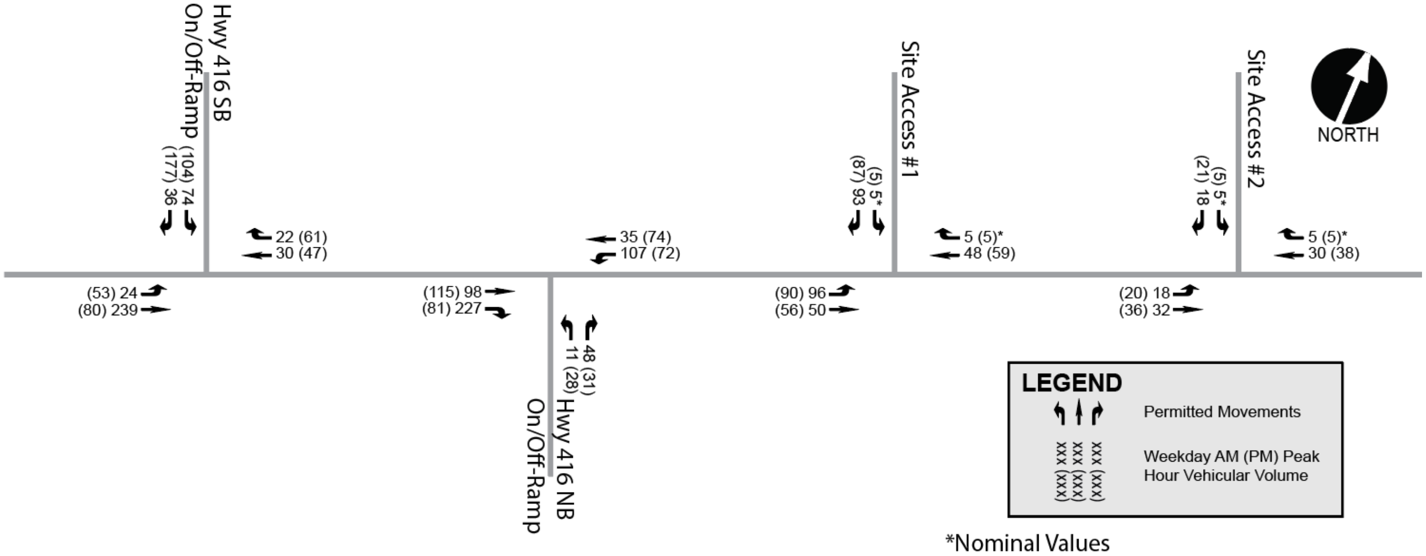
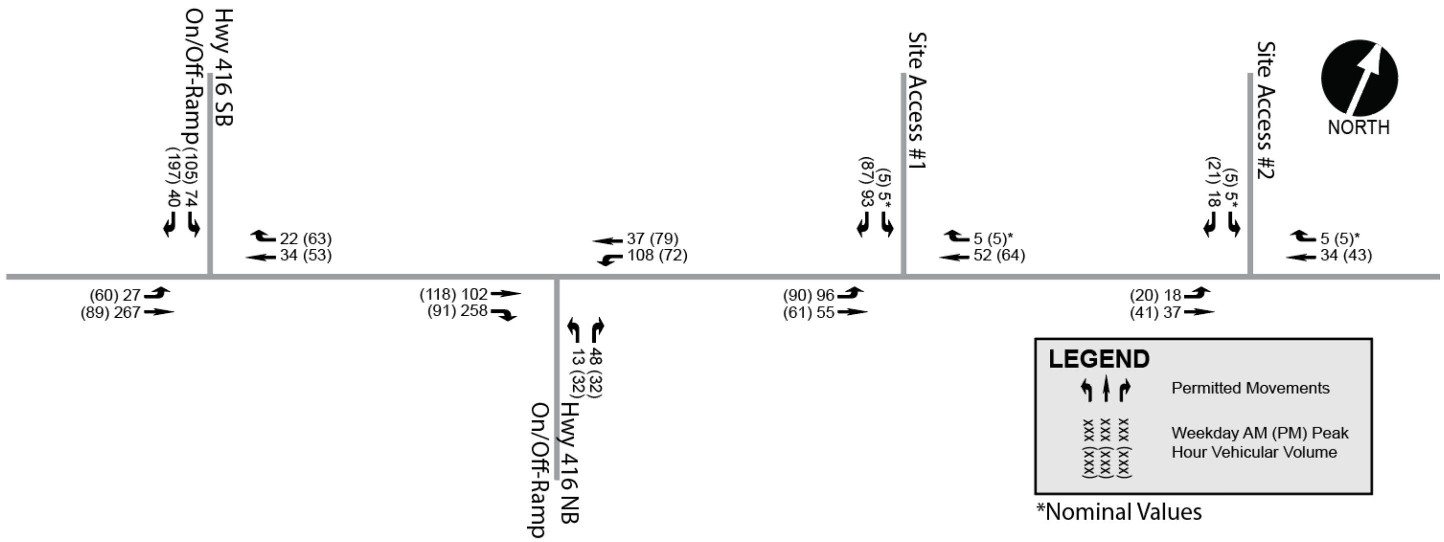


Figure 9 - Future (2027) Total Traffic



5 Analysis

5.1 Development Design

5.1.1 Design for Sustainable Modes

Due to the rural context of the site, there are no design features proposed specifically to support sustainable modes of transportation within the subject development. Further, there are no existing or planned active transportation facilities which require integration on the adjacent transportation network. As such, the TDM-Supportive Development Design and Infrastructure Checklist is not applicable for this particular development.

5.1.2 Circulation and Access

As discussed previously, Dilworth Road is identified as a Truck Route in the Official Plan along the site's frontage and is therefore expected to integrate well with adjacent road network.

The internal drive aisle of the site will be at least 6.0 metres wide and will therefore be designed to accommodate a Fire Route. Waste collection and delivery vehicles will be easily accommodated as well.

The oversized parking stalls in the northwest corner of the site are oriented at 45-degree angles and have been designed with internal drive aisle widths of approximately 19 metres which exceeds the minimum requirement of 11 metres prescribed in the Zoning By-law.

5.1.3 New Street Networks

Not Applicable: The New Street Networks element is exempt from this TIA, as defined in the study scope. This element is not required for Site Plan Control applications.

5.2 Parking

5.2.1 Parking Supply

Based on the size of the proposed gas station with convenience store, a minimum of 28 vehicle parking spaces are required, while the warehousing uses require at least 59 spaces for a total of 87 parking spaces, as prescribed for Area 'D' in the Zoning By-law. The conceptual site plan indicates that 97 vehicle parking spaces will be provided for the development and therefore the proposed parking supply is within the permissible range. Further review of the by-law indicates that at least 34 of the 97 spaces should be oversized and therefore the 60 oversized spaces proposed satisfy this requirement.

In accordance with Section 111 of the by-law, no bicycle parking spaces are required for the proposed development, as the site is not located within the limits of a village.

5.2.2 Spillover Parking

The minimum parking supply requirements specified in the Zoning By-law have been met, therefore no further review of parking is necessary for the purposes of this study.

5.3 Boundary Streets

5.3.1 Mobility

There are three existing boundary streets adjacent to the proposed development: Dilworth Road, Third Line Road South and Highway 416. As discussed in Section 3.4, segment-based MMLOS analysis for this study was limited to Dilworth Road along the site's frontage.

The results of the segment-based MMLOS analysis are summarized in **Table 7** below. Detailed results are provided in **Appendix I**.

Table 7 – Segment-based MMLOS - Existing & Proposed Conditions

LOCATION	LEVEL OF SERVICE BY MODE			
	PEDESTRIAN (PLOS)	BICYCLE (BLOS)	TRANSIT (TLOS)	TRUCK (TkLOS)
TARGET	N/A	N/A	N/A	C
SEGMENTS				
Dilworth Road – Site Frontage	F	F	N/A[1]	C

Notes: [1] Dilworth Road is not identified as a transit priority corridor in the TMP and is not served by regular transit service.

As shown in **Table 7** above, both the segment-based PLOS and BLOS are presently operating at 'F', however given the rural context of the area and lack of active transportation facilities on Dilworth Road adjacent to the site, no targets are identified in the MMLOS Guidelines for either mode.

In terms of transit, since Dilworth Road is not identified as an existing or planned transit priority corridor and is not served by regular transit service through the study area, a TLOS evaluation is not required in accordance with the MMLOS Guidelines.

The TkLOS target of 'C' is met on Dilworth Road within the site's frontage and is attributable to travel lane widths which can accommodate oversized vehicles which is appropriate given its Truck Route designation.

5.3.2 Road Safety

As discussed previously, there have been no reported collisions within the study area over the last 5 years, therefore no collision analysis was conducted for this study.

5.4 Access Intersections

5.4.1 Location and Design of Access

As discussed previously, two full-movement access driveways on Dilworth Road are proposed to service the subject development, including:

- Site Access #1 – The western site access driveway will provide direct access to the gas station with convenience store and commercial cardlock refuelling station.
- Site Access #2 – The eastern site access driveway will provide direct access to the mini-warehouse self-storage facility and cross-dock warehouse.

The proposed site access driveways, as described above, were reviewed with respect to the City of Ottawa Private Approach By-law 2003-447, with particular confirmation of the following items:

- Width: A private approach should have a minimum width of 2.4m and a maximum width of 9.0m.
  - According to the conceptual site plan presented in **Exhibit 2**, Site Access #1 will be 25m wide, while Site Access #2 is proposed with a width of 45 metres. The Private Approach Bylaw permits widths beyond 9 metres for transport loading areas and therefore both site access driveways are compliant with the bylaw. Despite generally meeting the bylaw requirements, however, it should be noted that this plan is highly conceptual and may not accurately represent the actual proposed site access driveway widths. As such, this component of the bylaw will be revisited during the Site Plan Control application stage. ✓
- Distance from Intersecting Road: For a commercial development on or within 46m of an arterial or major collector with between 50 and 99 parking spaces, the proposed private approach must be at least 30 metres from the nearest intersecting street line or another two-way private approach.
  - Site Access #1 is approximately 95 metres from the nearest intersecting street line and is therefore in compliance with the by-law. ✓
  - Site Access #2 is approximately 315 metres from the nearest intersecting street line and is therefore in compliance with the by-law. ✓
  - Both proposed site access driveways are separated by a distance of approximately 215 metres. ✓
- Quantity and Spacing of Private Approaches: For sites with frontage between 46 and 150 metres, the maximum number of private approaches is as follows, one (1) two-way private approach, and two (2) one-way private approaches, or two (2) two-way private approaches are allowed. For each additional 90 meters in excess of 150 meters, one (1) two-way private approach, or two (2) one-way approaches are allowed. Any two private approaches must be separated by at least 9.0m and can be reduced to 2.0m in the case of two one-way driveways. On lots that abut more than one roadway, these provisions apply to each frontage separately.
  - The proposed development has a frontage of approximately 590 metres, therefore both proposed two-way private approaches are compliant with the by-law. ✓
- Distance from Property Line: Private approaches must be at least 3.0m from the abutting property line, however this requirement can be reduced to 0.3m provided that the access is a safe distance from the access serving the adjacent property, sight lines are adequate and that it does not create a traffic hazard.
  - The proposed private approach exceeds the minimum distance required. ✓
- Grade of Private Approach: The grade of a private approach serving a parking area of more than 50 spaces must not exceed 2% within the private property for a distance of 9m from the highway/curb line.
  - This level of detail is not expected to be available until the Site Plan Control application stage and therefore this requirement will be assessed at that time.

The Transportation Association of Canada's (TAC) Geometric Design Guide for Canadian Roads (June 2017) suggests a minimum clear throat length of 30 metres for the proposed site access which coincides with the throat length indicated on **Exhibit 2**. As such, any internal spillback towards the site access is not expected to result in operational concerns on Dilworth Road.

5.4.2 Access Intersection Control

The proposed site access driveway on Dilworth Road will be stop-controlled.

5.4.3 Intersection Design (MMLOS)

Not Applicable – The proposed site access driveway will be unsignalized, therefore Multi-Modal Level of Service (MMLOS) analysis is not required.

5.5 Transportation Demand Management (TDM)

Not Applicable – The Transportation Demand Management (TDM) element is exempt from this TIA, as the proposed development is assumed to remain well below the minimum threshold of 60 employees and/or students on location at any given time.

5.5.1 Context for TDM

Not Applicable.

5.5.2 Need and Opportunity

Not Applicable.

5.5.3 TDM Program

Not Applicable.

5.6 Neighbourhood Traffic Management

5.6.1 Adjacent Neighbourhoods

Not Applicable – The site proposes a single direct connection to Dilworth Road, an arterial, and therefore will not be dependant on local or collector roads for access.

5.7 Transit

5.7.1 Route Capacity

Due to the rural context of the site, no transit service is currently provided or expected on the adjacent road network within the vicinity of the proposed development.

5.7.2 Transit Priority Measures

As discussed in the study scope, there are no Transit Priority Measures existing or planned within the study area during the timeframe of this study.

5.8 Review of Network Concept

Not Applicable – The Network Concept element is exempt from this TIA, as defined in the study scope. This element is not required for proposed developments expected to generate less than 200 person-trips beyond what is otherwise permitted by zoning during the weekday morning and afternoon peak hours.

5.9 Intersection Design

The following sections summarize the methodology and results of the multi-modal intersection capacity analysis conducted within the study area.

5.9.1 Intersection Control

5.9.1.1 Traffic Signal Warrants

Traffic signal warrants were completed for both ramp terminal intersections, as well as the proposed site access driveways. Based on the results of the analysis, warrants were not triggered at any of the study area intersections under Future (2027) Total Traffic conditions.

The results of the traffic signal warrant analysis are provided in **Appendix F**.

5.9.1.2 Roundabout Analysis

As per the City's Roundabout Implementation Policy, intersections that satisfy any of the following criteria should be screened utilizing the Roundabout Initial Feasibility Screening Tool:

- At any new City intersection
- Where traffic signals are warranted
- At intersections where capacity or safety problems are being experienced

Since neither of the study area intersections meet any of the above noted requirements, no roundabout analysis is required for this study. Further, the proposed site access will be configured as a stop-controlled intersection, as discussed previously, therefore no consideration will be given to implementing a roundabout at this location either.

5.9.2 Intersection Analysis Criteria (Automobile)

The following section outlines the City of Ottawa's methodology for determining motor vehicle Level of Service (LOS) at signalized and unsignalized intersections.

5.9.2.1 Signalized Intersections

In qualitative terms, the Level of Service (LOS) defines operational conditions within a traffic stream and their perception by motorists. A LOS definition generally describes these conditions in terms of such factors as delay, speed and travel time, freedom to manoeuvre, traffic interruptions, safety, comfort and convenience. LOS can also be related to the ratio of the volume to capacity (v/c) which is simply the relationship of the traffic volume (either measured or forecast) to the capability of the intersection or road section to accommodate a given traffic volume. This capability varies depending on the factors described above. LOS are given letter designations from 'A' to 'F'. LOS 'A' represents the best operating conditions and LOS 'E' represents the level at which the intersection or an approach to the intersection is carrying the maximum traffic volume that can, practicably, be accommodated. LOS 'F' indicates that the intersection is operating beyond its theoretical capacity.

The City of Ottawa has developed criteria as part of the Transportation Impact Assessment Guidelines, which directly relate the volume to capacity (v/c) ratio of a signalized intersection to a LOS designation. These criteria are shown in **Table 8** below.

Table 8 - LOS Criteria for Signalized Intersections

LOS	VOLUME TO CAPACITY RATIO (v/c)
A	0 to 0.60
B	0.61 to 0.70
C	0.71 to 0.80
D	0.81 to 0.90
E	0.91 to 1.00
F	> 1.00

The intersection capacity analysis technique provides an indication of the LOS for each movement at the intersection under consideration and for the intersection as a whole. The overall v/c ratio for an intersection is defined as the sum of equivalent volumes for all critical movements at the intersection divided by the sum of capacities for all critical movements.

The Level of Service calculation is based on locally-specific parameters as described in the TIA Guidelines and incorporates existing signal timing plans obtained from the City of Ottawa. The analysis existing conditions utilized a Peak Hour Factor (PHF) of 0.90, while future conditions considers optimized signal timing plans and use of a Peak Hour Factor (PHF) of 1.0 to recognize peak spreading beyond a 15-minute period in congested conditions.

5.9.2.2 Unsignalized Intersections

The capacity of an unsignalized intersection can also be expressed in terms of the LOS it provides. For an unsignalized intersection, the Level of Service is defined in terms of the average movement delays at the intersection. This is defined as the total elapsed time from when a vehicle stops at the end of the queue until the vehicle departs from the stop line; this includes the time required for a vehicle to travel from the last-in-queue position to the first-in-queue position. The average delay for any particular minor movement at the un-signalized intersection is a function of the capacity of the approach and the degree of saturation.

The Highway Capacity Manual 2010 (HCM), prepared by the Transportation Research Board, includes the following Levels of Service criteria for un-signalized intersections, related to average movement delays at the intersection, as indicated in **Table 9** below.

Table 9 - LOS Criteria for Unsignalized Intersections

LOS	DELAY (seconds)
A	<10
B	>10 and <15
C	>15 and <25
D	>25 and <35
E	>35 and <50
F	>50

The unsignalized intersection capacity analysis technique included in the HCM and used in the current study provides an indication of the Level of Service for each movement of the intersection under consideration. By this technique, the performance of the unsignalized intersection can be compared under varying traffic scenarios, using the Level of Service concept in a qualitative sense. One unsignalized intersection can be compared with another unsignalized intersection using this concept. Level of Service 'E' represents the capacity of the movement under consideration and generally, in large urban areas, Level of Service 'D' is considered to represent an acceptable operating condition. Level of Service 'E' is considered an acceptable operating condition for planning purposes for intersections located within Ottawa's Urban Core the downtown and its vicinity). Level of Service 'F' indicates that the movement is operating beyond its design capacity.

5.9.3 Intersection Capacity Analysis

Following the established intersection capacity analysis criteria described above, the existing and future conditions are analyzed during the weekday peak hour traffic volumes derived in this study.

The following section presents the results of the intersection capacity analysis. All tables summarize study area intersection LOS results during the weekday morning and afternoon peak hour periods.

The Synchro output files have been provided in **Appendix G**.

5.9.3.1 Existing (2021) Traffic

An intersection capacity analysis has been undertaken using the Existing (2021) Traffic volumes presented in **Figure 2**, yielding the following results:

Table 10 - Intersection Capacity Analysis: Existing (2021) Traffic

INTERSECTION	TRAFFIC CONTROL	AM PEAK HOUR		PM PEAK HOUR	
		OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)	OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)
Dilworth Road & Highway 416 NB On/Off-Ramp	Unsignalized	A (8.7s)	SBRL (8.7s)	A (9.6s)	SBRL (9.6s)
Dilworth Road & Highway 416 SB On/Off-Ramp	Unsignalized	A (8.8s)	NBRL (8.8s)	A (8.9s)	NBRL (8.9s)

Based on the above, both study area intersections are presently operating at an acceptable level of service (LOS 'D' or better) under Existing (2021) Traffic conditions.

5.9.3.2 Future (2022) Background Traffic

An intersection capacity analysis has been undertaken using the Future (2022) Background Traffic volumes presented in **Figure 6**, yielding the following results:

Table 11 - Intersection Capacity Analysis: 2022 Background Traffic

INTERSECTION	TRAFFIC CONTROL	AM PEAK HOUR		PM PEAK HOUR	
		OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)	OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)
Dilworth Road & Highway 416 NB On/Off-Ramp	Unsignalized	A (8.6s)	SBRL (8.6s)	A (9.4s)	SBRL (9.4s)
Dilworth Road & Highway 416 SB On/Off-Ramp	Unsignalized	A (8.8s)	NBRL (8.8s)	A (8.9s)	NBRL (8.9s)

Based on the above, both study area intersections are expected to continue operating at an acceptable level of service (LOS 'D' or better) under Future (2022) Background Traffic conditions.

5.9.3.3 Future (2027) Background Traffic

An intersection capacity analysis has been undertaken using the Future (2027) Background Traffic volumes presented in **Figure 7**, yielding the following results:

Table 12 - Intersection Capacity Analysis: 2027 Background Traffic

INTERSECTION	TRAFFIC CONTROL	AM PEAK HOUR		PM PEAK HOUR	
		OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)	OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)
Dilworth Road & Highway 416 NB On/Off-Ramp	Unsignalized	A (8.7s)	SBRL (8.7s)	A (9.6s)	SBRL (9.6s)
Dilworth Road & Highway 416 SB On/Off-Ramp	Unsignalized	A (8.8s)	NBRL (8.8s)	A (8.9s)	NBRL (8.9s)

Based on the above, both study area intersections are expected to continue operating at an acceptable level of service (LOS 'D' or better) under Future (2027) Background Traffic conditions.

5.9.3.4 Future (2022) Total Traffic

An intersection capacity analysis has been undertaken using the Future (2022) Total Traffic volumes presented in **Figure 8**, yielding the following results:

Table 13 - Intersection Capacity Analysis: 2022 Total Traffic

INTERSECTION	TRAFFIC CONTROL	AM PEAK HOUR		PM PEAK HOUR	
		OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)	OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)
Dilworth Road & Highway 416 NB On/Off-Ramp	Unsignalized	B (10.6s)	SBRL (10.6s)	B (10.9s)	SBRL (10.9s)
Dilworth Road & Highway 416 SB On/Off-Ramp	Unsignalized	A (9.5s)	NBRL (9.5s)	B (10.1s)	NBRL (10.1s)
Dilworth & Site Access #1	Unsignalized	A (8.9s)	SBRL (8.9s)	A (8.9s)	SBRL (8.9s)
Dilworth & Site Access #2	Unsignalized	A (8.5s)	SBRL (8.5s)	A (8.6s)	SBRL (8.6s)

Based on the above, all four study area intersections are expected to operate at an acceptable level of service (LOS ‘D’ or better) under Future (2022) Total Traffic conditions.

5.9.3.5 Future (2027) Total Traffic

An intersection capacity analysis has been undertaken using the Future (2027) Total Traffic volumes presented in **Figure 9**, yielding the following results:

Table 14 - Intersection Capacity Analysis: 2027 Total Traffic

INTERSECTION	TRAFFIC CONTROL	AM PEAK HOUR		PM PEAK HOUR	
		OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)	OVERALL LOS (V/C OR DELAY)	CRITICAL MOVEMENTS (V/C OR DELAY)
Dilworth Road & Highway 416 NB On/Off-Ramp	Unsignalized	B (10.8s)	SBRL (10.8s)	B (11.3s)	SBRL (11.3s)
Dilworth Road & Highway 416 SB On/Off-Ramp	Unsignalized	A (9.6s)	NBRL (9.6s)	B (10.3s)	NBRL (10.3s)
Dilworth & Site Access #1	Unsignalized	A (8.9s)	SBRL (8.9s)	A (8.9s)	SBRL (8.9s)
Dilworth & Site Access #2	Unsignalized	A (8.5s)	SBRL (8.5s)	A (8.6s)	SBRL (8.6s)

Based on the above, all four study area intersections are expected to operate at an acceptable level of service (LOS ‘D’ or better) under Future (2027) Total Traffic conditions.

5.9.4 Intersection Design (MMLOS)

Not Applicable – As verified through intersection capacity analyses presented in the preceding sections, intersection Multi-Modal Level of Service (MMLOS) analysis is not required, since none of the study area intersections are expected to trigger the need for traffic signals within the timeframe of this study.

5.10 Geometric Review

The following section provides a review of all geometric requirements for the study area intersections.

5.10.1 Sight Distance and Corner Clearances

The proposed site access driveway are located on a segment of Dilworth Road with a minor vertical curve to the west and a gradual horizontal curve to the east. Despite these constraints, the, both locations are expected to allow for visibility in excess of the 160-metre distance suggested by TAC for a road with a 90 km/h design speed. Provided that vegetation is kept clear of the intersection sightlines, sight distances and corner clearances are not expected to be a concern for the proposed development's site access driveway.

5.10.2 Auxiliary Lane Analyses

5.10.2.1 Auxiliary Left-Turn Lane Requirements (Unsignalized)

Auxiliary left-turn lane analyses for all unsignalized study area intersections were completed under the Future (2027) Total Traffic conditions. The operating speed on Dilworth Road was assumed to be 90 km/h, representing 10 km/h over the posted speed limit.

The MTO Geometric Design Standards for Ontario Highways left-turn warrant was applied to main street approaches at all unsignalized intersections using the highest left-turn volume from either the weekday morning or afternoon peak hour. The results have been summarized in **Table 10**Table 15 below.

Table 15 - Auxiliary Left-Turn Lane Analysis at Unsignalized Intersections

INTERSECTION	APPROACH	VOLUME ADVANCING (V_A)	VOLUME OPPOSING (V_O)	% LEFT TURN IN V_A[1]	EXISTING PARALLEL LANE LENGTH (M)	STORAGE DEFICIENCY (M)
Dilworth & Hwy 416 SB On/Off-Ramp	EB	294	56	10%	75	Existing Storage Adequate
Dilworth & Hwy 416 NB On/Off-Ramp	WB	145	360	40%	85	Existing Storage Adequate
Dilworth & Site Access #1	EB	150	52	40%	-	No Storage Required
Dilworth & Site Access #2	EB	61	43	35%	-	No Storage Required

Note: [1] MTO left-turn warrant graphs do not exceed 40% turning vehicles relative to approach volumes.

Based on the analysis presented in **Table 15** above, the existing auxiliary left-turn lanes at the Highway 416 ramp terminal intersections are expected to sufficiently accommodate Future (2027) Total Traffic conditions, while the proposed site access driveways do not warrant an auxiliary left-turn lane.

TAC also recommends consideration be given to implementing a left-turn slip lane when volumes do not warrant full left-turn lanes. Based on the estimated east-west traffic volumes on Dilworth Road, which are expected to remain well below the general capacity threshold of 1,000 vehicle per hour per lane (vphpl) assumed for arterial roads, sufficient gaps are likely to be available to accommodate the relatively low volume of inbound left-turning traffic during the weekday peak hours. As such, a left-turn slip lane is not required at the site access either.

5.10.2.2 Auxiliary Right-Turn Lane Requirements (Unsignalized)

The Transportation Association of Canada (TAC) suggests that auxiliary right-turn lanes be considered "when the volume of decelerating or accelerating vehicles compared with through vehicles causes undue hazard." Consideration for auxiliary right-turn lanes is typically given when the right-turning traffic exceeds 10% of the through volume and is at least 60 vehicles per hour.

The Highway 416 & Dilworth Road ramp terminal intersections are presently configured with right-turn auxiliary lanes which are capable of accommodating 95th percentile queues under Future (2027) Total Traffic conditions. With regards to the proposed site access driveways, site-generated traffic volumes on the westbound approaches are expected to be nominal and therefore right-turn auxiliary lanes are not required. As such, no additional auxiliary right-turn lanes are needed on the adjacent road network as a result of projected background or site-generated traffic volumes within the timeframe of this study.

5.11 Summary of Improvements Indicated and Modification Options

Based on the intersection capacity analyses conducted for this study, all four study area intersections are expected to operate at an acceptable level of service beyond the 2027 horizon year.

An analysis of auxiliary lane requirements found that auxiliary storage lanes at both existing ramp terminal intersections are expected to sufficiently accommodate future travel demands within the timeframe of this study. Further, no auxiliary left- or right-turn lanes would be required to support site-generated travel demand on Dilworth Road at either of the proposed site access driveway locations.

6 Conclusion

The proposed development includes a gas station with convenience store, a mini-warehouse self-storage facility, a cross-dock warehouse and a commercial cardlock fuelling station at 2095 Dilworth Road, Ottawa. The results of the trip generation exercise conducted as part of this study indicate that 157 and 148 new two-way vehicular trips are expected during the weekday morning and afternoon peak hours, respectively. Pass-by traffic generated by the gas station use was also considered in the analysis, with up to 68 and 70 trips expected to occur during the weekday morning and afternoon peak hours, respectively. The mode share targets applied in this study were based on the Rural Southwest Traffic Assessment Zone (TAZ) and further refined to reflect the auto-oriented nature of the proposed development. The site-generated traffic projections were divided amongst two proposed site access driveways which will help to mitigate the potential for traffic operational issues from occurring on the adjacent road network.

A segment-based multi-modal analysis identified deficiencies for sustainable modes on Dilworth Road adjacent to the site. It should be noted, however, that due to the rural context of the site and auto dependency of uses proposed, no improvements are required to safely accommodate the transportation demands of the subject development.

Based on the intersection capacity analyses conducted for this study, all four study area intersections are expected to operate at an acceptable level of service beyond the 2027 horizon year.

Queuing analysis conducted under Future (2027) Total Traffic conditions provided further indication that traffic operational issues are not expected to be a concern at any of the study area intersections within the timeframe of this study. Auxiliary left- or right-turn lanes at both existing ramp terminal intersections are expected to sufficiently accommodate future travel demands within the timeframe of this study. Further, the analysis did not identify the need for any auxiliary lanes to support site-generated traffic volumes on Dilworth Road at either Site Access #1 or Site Access #2.

As all study area intersections were shown to operate well within the capacity constraints of the adjacent transportation network, an RMA will <u>not</u> be required. Further, a post-development Monitoring Plan has been deemed unnecessary to support this study.

Based on the findings of this study, it is the overall opinion of IBI Group that the proposed development will integrate well with and can be safely accommodated by the adjacent transportation network.

Appendix A – City Circulation Comments

Step 1 & 2 Submission (Screening & Scoping) – Circulation Comments & Response

Report Submitted: May 10, 2021
Comments Received: May 14, 2021
Transportation Project Manager: Mike Giampa

1) No comments were received from the City as part of the Step 1 & 2: Screening & Scoping for the 2095 Dilworth Road Transportation Impact Assessment (TIA).

Step 3 Submission (Forecasting) – Circulation Comments & Response

Report Submitted: June 15, 2021
Comments Received: July 12, 2021
Transportation Project Manager: Mike Giampa

Transportation Engineering Services

1) 4.1.2.3 Mode Share Proportions: In Table 5, the determined mode share targets on Auto Passenger may be too high as the proposed site is in a rural area with many of the trips generated by Auto drivers.

   - **IBI Response: It is acknowledged that the Auto Passenger mode share may have been previously overestimated at 19%, therefore it is proposed to reduce its allocation to 13% and maintain consistency with the weighted average from the Rural Southwest TAZ. Consequently, the remaining sustainable modes derived from the TAZ were redistributed directly to Auto Driver, resulting in its increase from 81% to 87%.**

2) Section 4.1.3 Trip Distribution and Assignment: In Figure 4, the total exiting trips from both proposed accesses for the AM peak is 71, which contradicts the numbers been provided in Table 6 for 75. Update the figures in both Figure 4 and 5 based on the assumptions made in previous sections.

   - **IBI Response: Figures 4 and 5 have been revised accordingly to ensure that vehicle trips illustrated correspond with the values presented previously in Table 5.**

Appendix B – TIA Screening Form

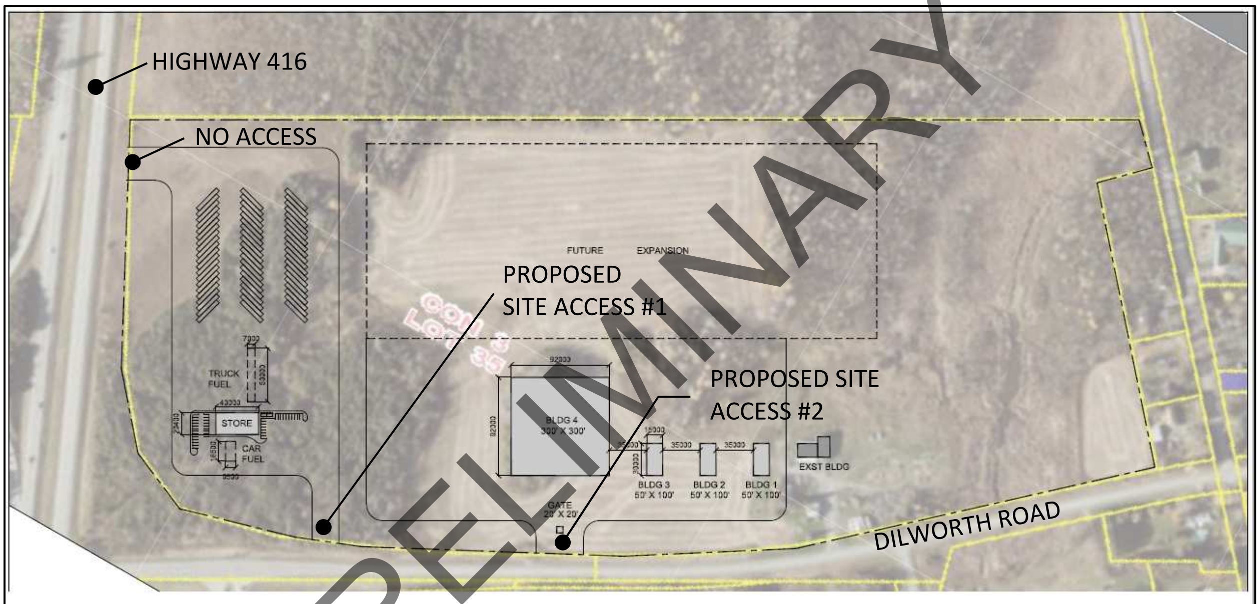
Transportation Impact Assessment Screening Form

City of Ottawa 2017 TIA Guidelines Screening Form

1. Description of Proposed Development

Municipal Address	2095 Dilworth Road, Ottawa, ON
Description of Location	North Gower-Kars Community – Northeast corner of Highway 416 & Dilworth Road 
Land Use Classification	Rural Commercial
Development Size (units)	Proposed Gas Station with Convenience Store – 6 to 8 pumps
Development Size (m^2)	Self-Storage Facility – 9,755 m^2 (105,000 ft^2)
Number of Accesses and Locations	• Two (2) full-movement private approaches on Dilworth Road
Phase of Development	Single-phase
Buildout Year	2022

If available, <u>please attach a sketch of the development or site plan</u> to this form.


HIGHWAY 416
NO ACCESS
FUTURE
EXPANSION
PROPOSED
SITE ACCESS #1
PROPOSED SITE
ACCESS #2
TRUCK
FUEL
STORE
CAR
FUEL
BLDG 4
300' X 300'
BLDG 3
50' X 100'
BLDG 2
50' X 100'
BLDG 1
50' X 100'
EXST BLDG
GATE
20' X 20'
DILWORTH ROAD
PRELIMINARY

2. Trip Generation Trigger

Considering the Development's Land Use type and Size (as filled out in the previous section), please refer to the Trip Generation Trigger checks below.

Land Use Type	Minimum Development Size
Single-family homes	40 units
Townhomes or apartments	90 units
Office	3,500 m^2
Industrial	5,000 m^2
Fast-food restaurant or coffee shop	100 m^2
Destination retail	1,000 m^2
Gas station or convenience market	75 m^2 ✓

** If the development has a land use type other than what is presented in the table above, estimates of person-trip generation may be made based on average trip generation characteristics represented in the current edition of the Institute of Transportation Engineers (ITE) Trip Generation Manual.*

Based on the above, the Trip Generation Trigger is satisfied.

3. Location Triggers

	Yes	No
Does the development propose a new driveway to a boundary street that is designated as part of the City's Transit Priority, Rapid Transit or Spine Bicycle Networks?		✓
Is the development in a Design Priority Area (DPA) or Transit-oriented Development (TOD) zone?*		✓

**DPA and TOD are identified in the City of Ottawa Official Plan (DPA in Section 2.5.1 and Schedules A and B; TOD in Annex 6). See Chapter 4 for a list of City of Ottawa Planning and Engineering documents that support the completion of TIA).*

Based on the above, the Location Trigger is <u>not</u> satisfied.

4. Safety Triggers

	Yes	No
Are posted speed limits on a boundary street are 80 km/hr or greater?	✓	
Are there any horizontal/vertical curvatures on a boundary street limits sight lines at a proposed driveway?	✓	
Is the proposed driveway within the area of influence of an adjacent traffic signal or roundabout (i.e. within 300 m of intersection in rural conditions, or within 150 m of intersection in urban/ suburban conditions)?		✓
Is the proposed driveway within auxiliary lanes of an intersection?		✓
Does the proposed driveway make use of an existing median break that serves an existing site?		✓
Is there is a documented history of traffic operations or safety concerns on the boundary streets within 500 m of the development?		✓
Does the development include a drive-thru facility?		✓

Based on the above, the Safety Trigger is satisfied.

5. Summary

	Yes	No
Does the development satisfy the Trip Generation Trigger?	✓	
Does the development satisfy the Location Trigger?		✓
Does the development satisfy the Safety Trigger?	✓	

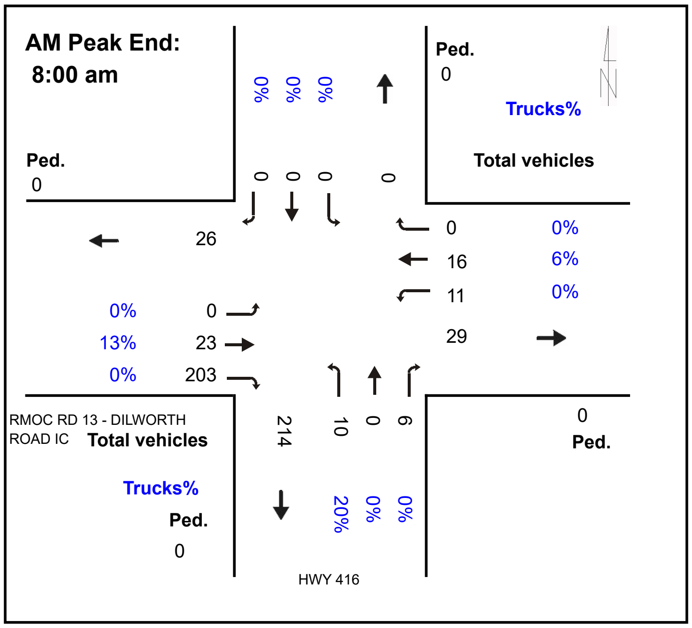
Based on the results of the TIA Screening Form, the Trip Generation and Safety Triggers are satisfied. As such, a TIA is required for the proposed development at 2095 Dilworth Road.

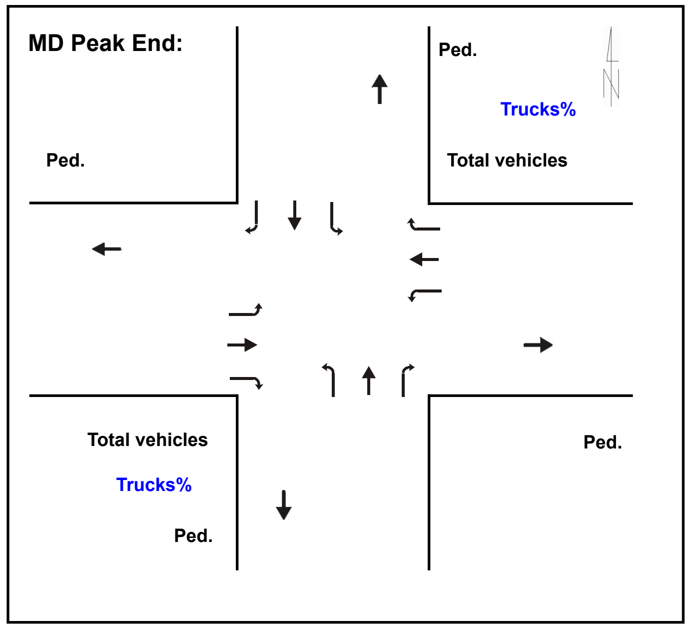
Appendix C – Traffic Data

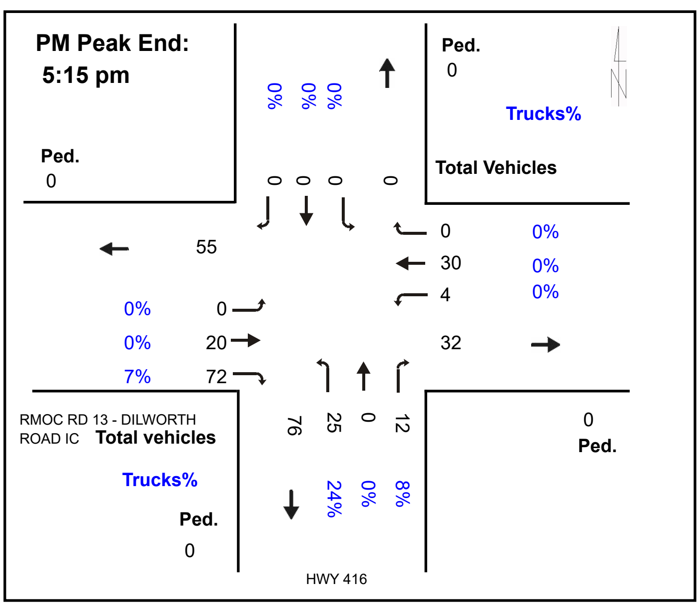
HWY 416 @ RMOC RD 13 - DILWORTH ROAD IC

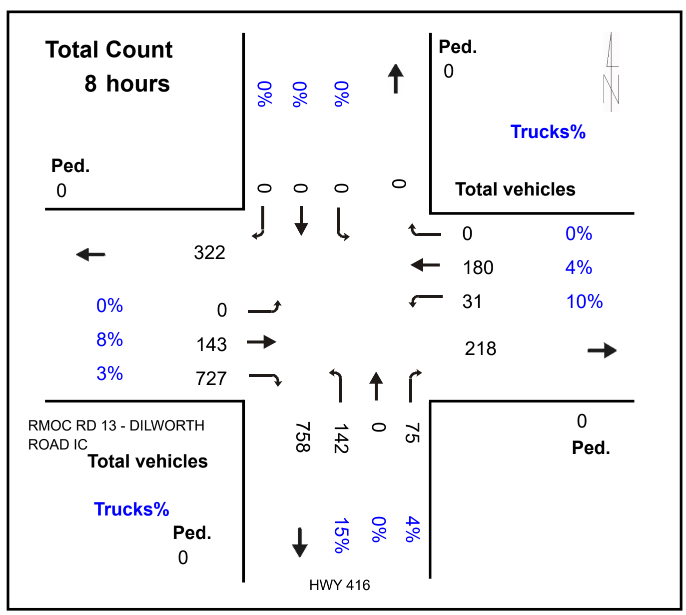
Eastern

Intersection ID:491850000(--E--) **Count Day: Thursday** **Count Date: 24-May-2018**





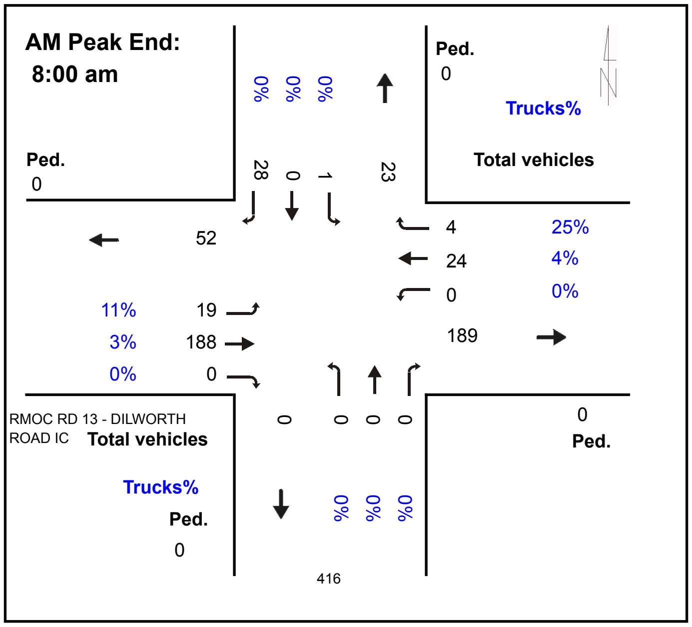


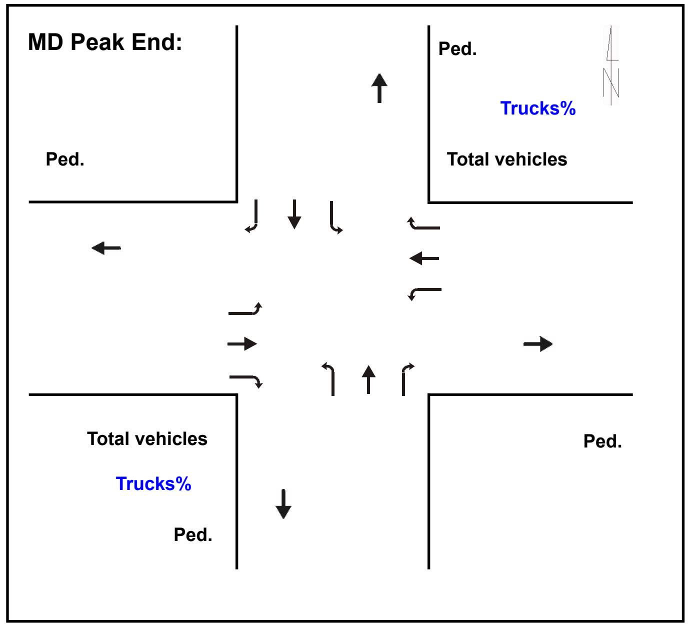


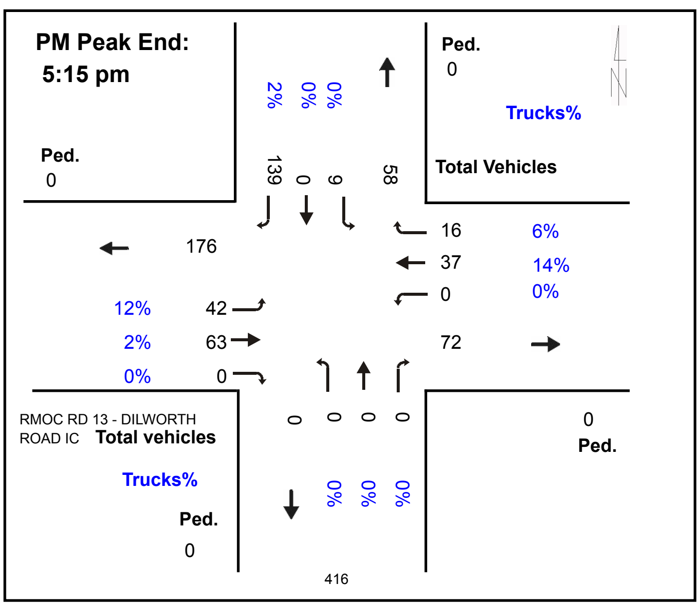
HWY 416 @ RMOC RD 13 - DILWORTH ROAD IC

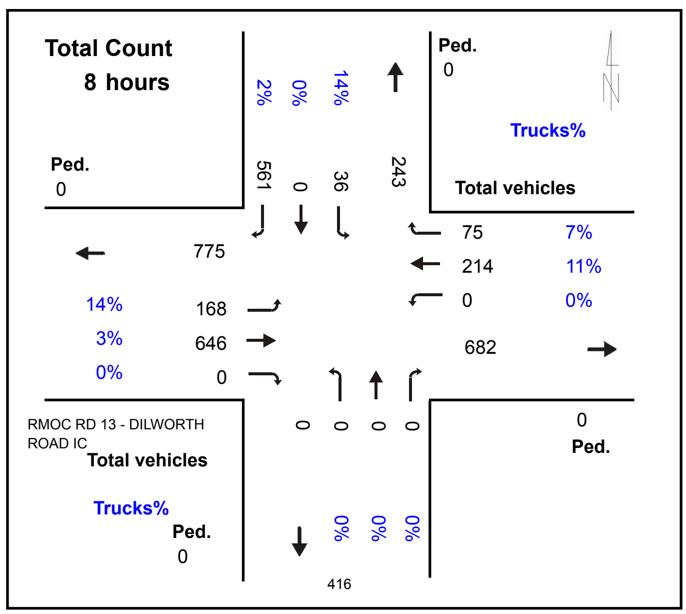
Eastern

Intersection ID:491850000(--W--) **Count Day: Thursday** **Count Date: 12-Sep-2013**









Highway	Location Description	Dist. (KM)	Year	Pattern Type	AADT	SADT	SAWDT	WADT	AR
			2008	IR	21,700	26,900	26,300	18,400	0.4
			2009	IR	22,500	25,600	24,800	20,400	0.3
			2010	IR	24,100	27,300	26,400	21,900	0.3
			2011	IR	25,200	30,000	29,500	21,400	N/A
			2012	IR	26,400	31,400	30,600	22,700	N/A
			2013	IR	27,500	32,800	35,300	23,400	N/A
			2014	IR	28,700	34,200	34,200	24,400	N/A
			2015	IR	26,300	31,300	31,300	22,400	N/A
			2016	IR	30,300	36,000	36,000	25,700	N/A
416	L &G RD 19 - RIDEAU RIVER ROAD IC	2.1	1997	IR	7,900	10,100	9,700	6,650	0.0
			1998	IR	8,300	10,500	10,100	7,000	0.2
			1999	IR	8,800	11,100	10,700	7,400	0.7
			2000	IR	10,300	13,000	12,500	8,700	1.2
			2001	IR	11,700	14,700	14,200	9,850	0.5
			2002	IR	13,200	16,700	16,000	11,100	1.4
			2003	IR	14,700	18,500	17,800	12,500	1.0
			2004	IR	16,200	20,100	19,300	13,700	0.9
			2005	IR	17,600	21,800	20,900	14,900	1.1
			2006	IR	19,000	23,500	22,500	16,100	0.8
			2007	IR	20,100	24,900	24,700	17,000	0.9
			2008	IR	21,200	26,200	25,700	18,000	0.3
			2009	IR	22,000	25,000	24,200	19,900	0.3
			2010	IR	25,100	30,800	27,600	20,200	0.5
			2011	IR	25,400	30,200	29,700	21,600	N/A
			2012	IR	26,800	31,800	31,000	23,000	N/A
			2013	IR	28,100	33,400	36,000	23,900	N/A
			2014	IR	29,400	35,000	35,000	25,000	N/A
			2015	IR	27,400	32,600	32,600	23,300	N/A
			2016	IR	31,400	37,300	37,300	26,700	N/A
416	RMOC RD 13 - DILWORTH ROAD IC	6.4	1997	CR	9,500	12,400	10,800	7,700	0.0
			1998	CR	10,000	12,900	11,400	8,100	0.3
			1999	CR	10,700	13,800	12,100	8,650	0.5
			2000	CR	15,100	18,500	17,700	12,800	0.5

Highway	Location Description	Dist. (KM)	Year	Pattern Type	AADT	SADT	SAWDT	WADT	AR
			2001	CR	14,400	17,700	16,800	14,400	0.4
			2002	CR	15,500	19,100	18,200	13,100	0.6
			2003	CR	16,300	20,000	19,100	13,900	0.5
			2004	CR	16,900	20,600	19,800	14,300	0.7
			2005	CR	17,900	21,700	20,900	15,100	0.2
			2006	CR	18,900	22,900	22,000	16,000	0.3
			2007	CR	20,000	24,200	24,200	16,900	0.4
			2008	CR	20,000	24,200	24,000	16,900	0.4
			2009	CR	21,900	26,400	25,400	18,500	0.3
			2010	CR	22,900	27,400	26,500	19,400	0.4
			2011	CR	23,900	28,000	28,200	21,300	N/A
			2012	CR	24,900	29,900	29,400	21,200	N/A
			2013	CR	23,500	28,200	28,900	20,000	N/A
			2014	CR	26,900	31,700	31,500	22,900	N/A
			2015	CR	27,400	32,300	32,100	23,300	N/A
			2016	CR	28,400	33,500	33,200	24,100	N/A
416	RMOC RD 6 - ROGER STEVENS DRIVE IC	8.4	1998	CR	10,000	11,300	10,700	8,950	0.7
			1999	CR	10,500	11,900	11,200	9,350	0.7
			2000	CR	16,200	19,900	19,000	13,700	0.4
			2001	CR	15,000	18,500	17,600	15,000	0.3
			2002	CR	16,100	19,800	18,900	13,600	0.3
			2003	CR	17,600	21,600	20,600	15,000	0.5
			2004	CR	18,800	22,900	22,000	15,900	0.3
			2005	CR	20,300	24,600	23,700	17,100	0.4
			2006	CR	22,000	26,700	25,600	18,600	0.4
			2007	CR	22,600	27,400	27,300	19,000	0.4
			2008	CR	23,800	28,800	28,600	20,100	0.3
			2009	CR	25,700	30,900	29,800	21,700	0.3
			2010	CR	25,100	30,100	29,000	21,200	0.3
			2011	CR	27,800	32,500	32,800	24,700	N/A
			2012	CR	29,000	34,900	34,300	24,700	N/A
			2013	CR	30,300	36,400	37,300	25,800	N/A
			2014	CR	31,600	37,300	37,000	26,900	N/A

Appendix D – Trip Generation Data

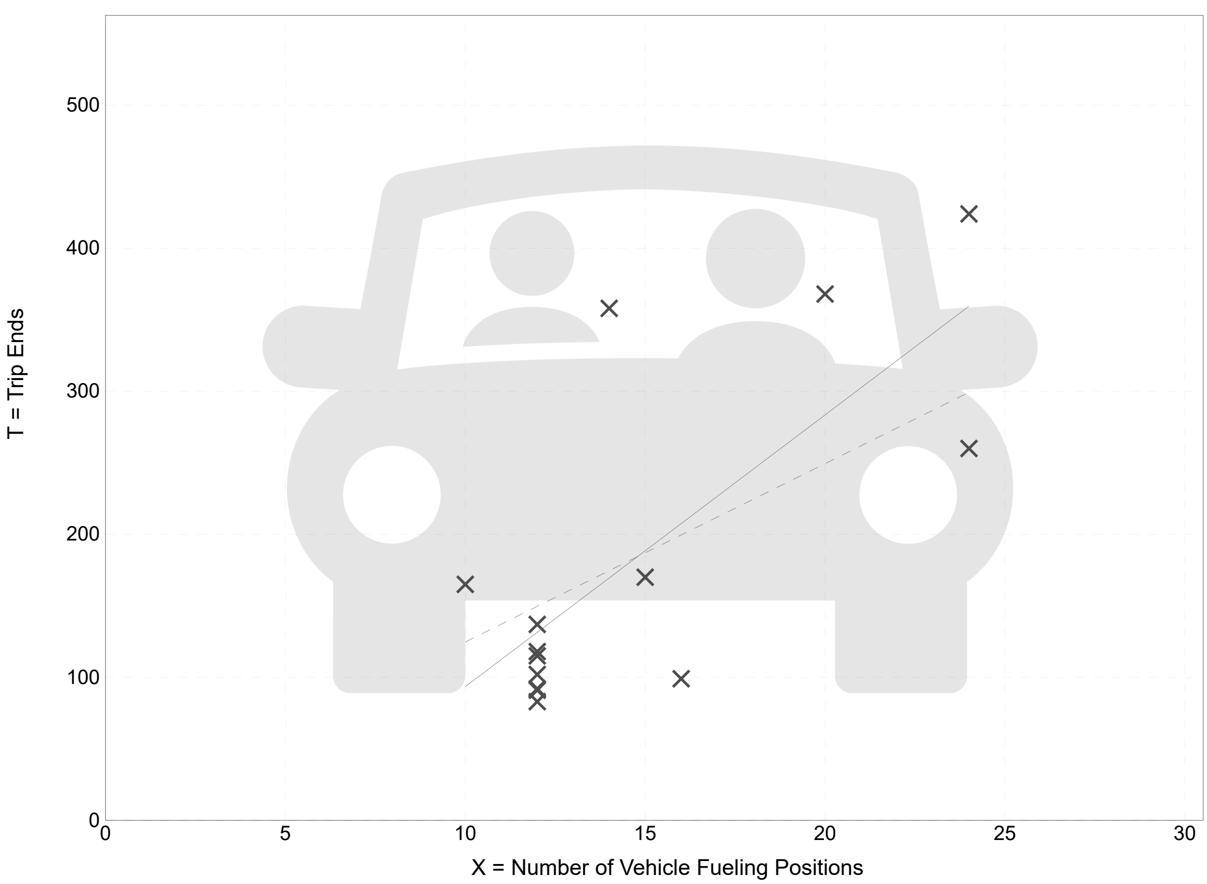
Gasoline/Service Station With Convenience Market
(945)

Vehicle Trip Ends vs:	**Vehicle Fueling Positions**
On a:	**Weekday,**
	Peak Hour of Adjacent Street Traffic,
	One Hour Between 7 and 9 a.m.
Setting/Location:	**General Urban/Suburban**
Number of Studies:	14
Avg. Num. of Vehicle Fueling Positions:	15
Directional Distribution:	51% entering, 49% exiting

Vehicle Trip Generation per Vehicle Fueling Position

Average Rate	Range of Rates	Standard Deviation
12.47	6.19 - 25.57	5.56

Data Plot and Equation



Fitted Curve Equation: T = 19.00(X) - 96.53 **R^2= 0.55**

Trip Gen Manual, 10th Ed + Supplement • Institute of Transportation Engineers

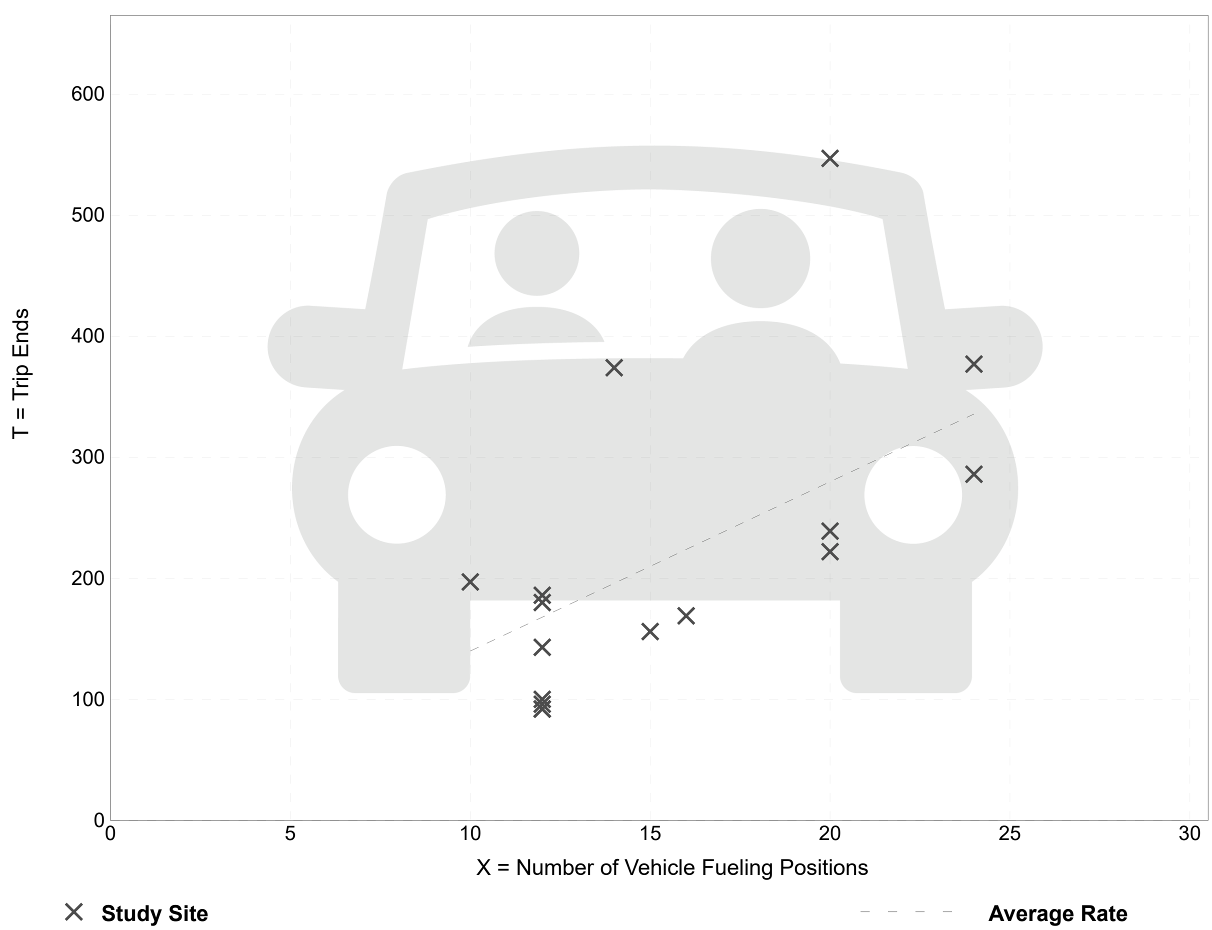
Gasoline/Service Station With Convenience Market
(945)

Vehicle Trip Ends vs:	**Vehicle Fueling Positions**
On a:	**Weekday,** **Peak Hour of Adjacent Street Traffic,** **One Hour Between 4 and 6 p.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	16
Avg. Num. of Vehicle Fueling Positions:	15
Directional Distribution:	51% entering, 49% exiting

Vehicle Trip Generation per Vehicle Fueling Position

Average Rate	Range of Rates	Standard Deviation
13.99	7.67 - 27.35	6.18

Data Plot and Equation



Fitted Curve Equation: Not Given **R^2= ******

Trip Gen Manual, 10th Ed + Supplement • Institute of Transportation Engineers

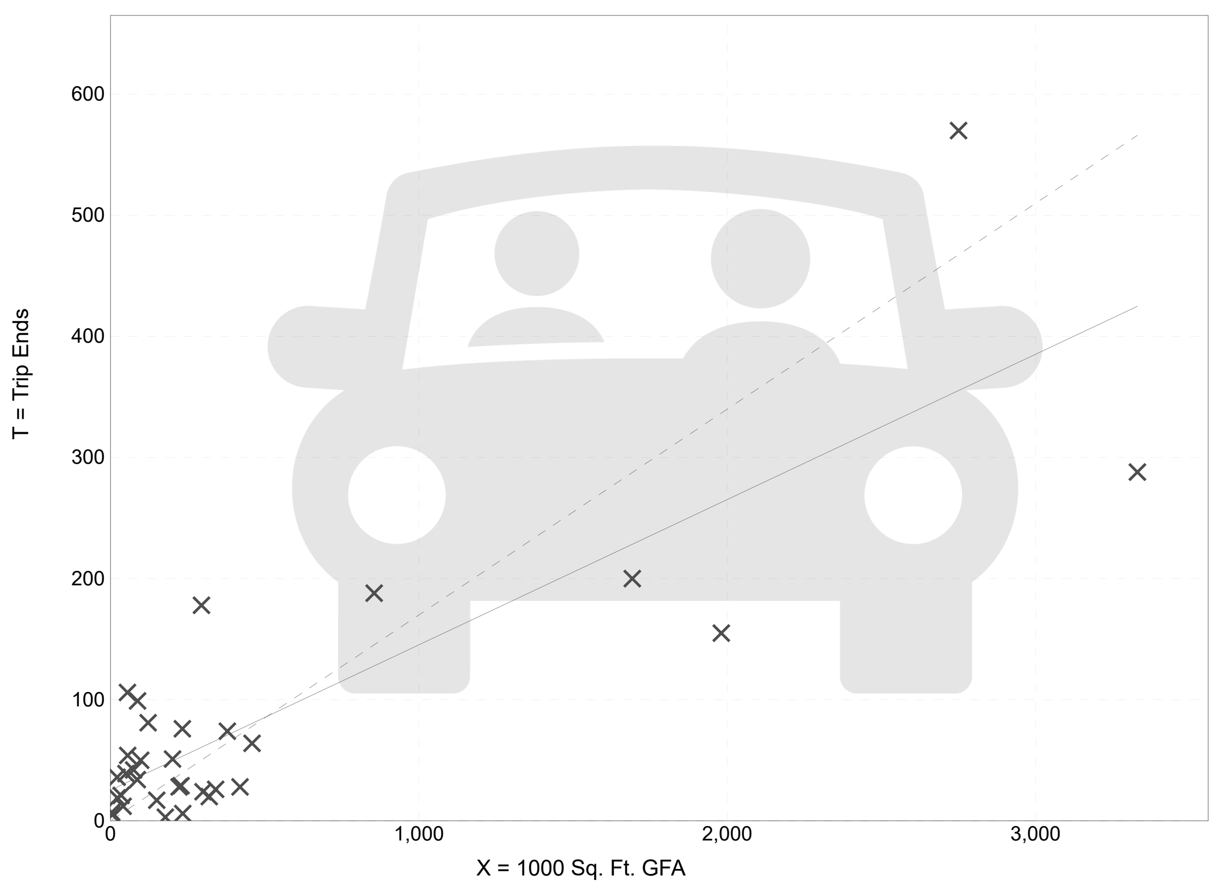
Warehousing
(150)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday, Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	34
Avg. 1000 Sq. Ft. GFA:	451
Directional Distribution:	77% entering, 23% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
0.17	0.02 - 1.93	0.20

Data Plot and Equation



× **Study Site** —— **Fitted Curve** - - - **Average Rate**

Fitted Curve Equation: $T = 0.12(X) + 25.32$ **$R^2 = 0.69$**

Trip Gen Manual, 10th Ed + Supplement • Institute of Transportation Engineers

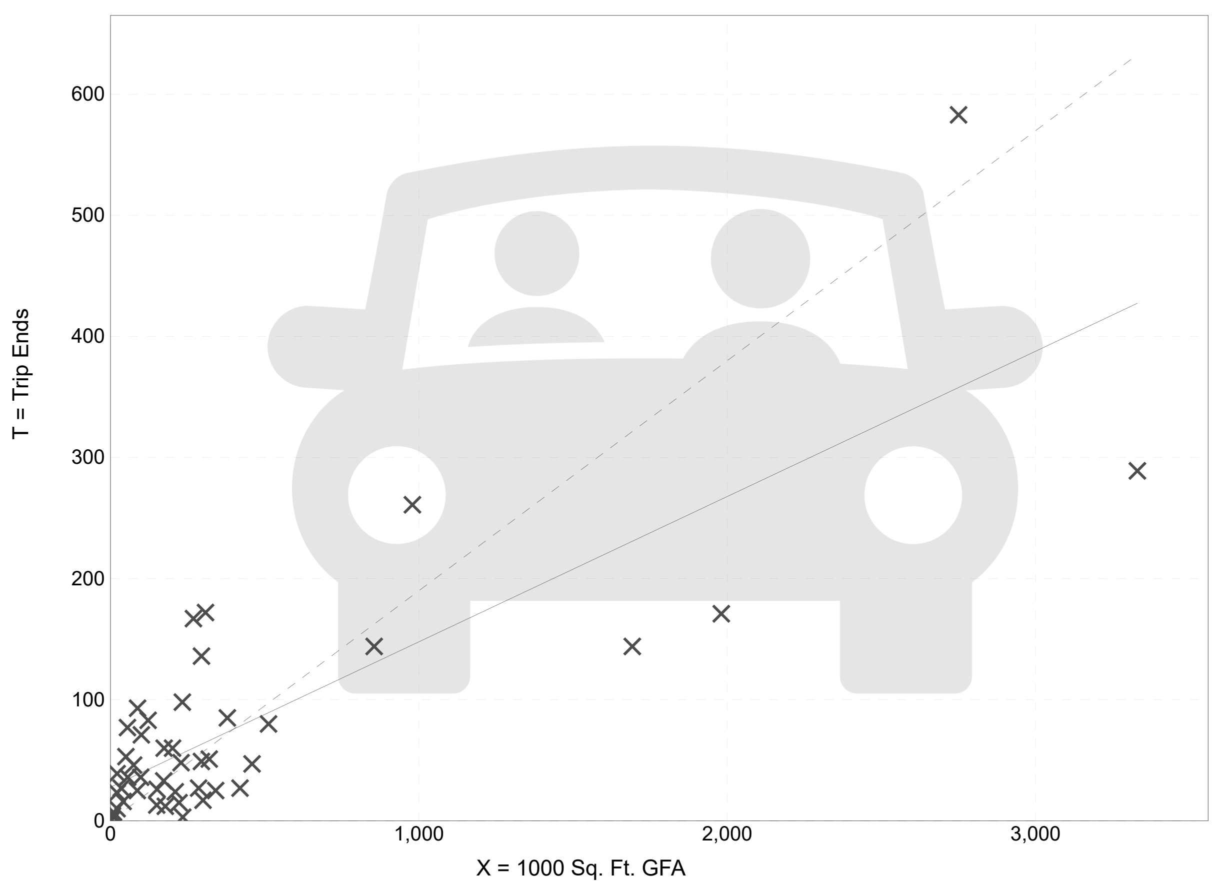
Warehousing
(150)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday, Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	47
Avg. 1000 Sq. Ft. GFA:	400
Directional Distribution:	27% entering, 73% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
0.19	0.01 - 1.80	0.18

Data Plot and Equation



Study Site **Fitted Curve** **Average Rate**

Fitted Curve Equation: $T = 0.12(X) + 27.82$ **$R^2 = 0.65$**

Trip Gen Manual, 10th Ed + Supplement • Institute of Transportation Engineers

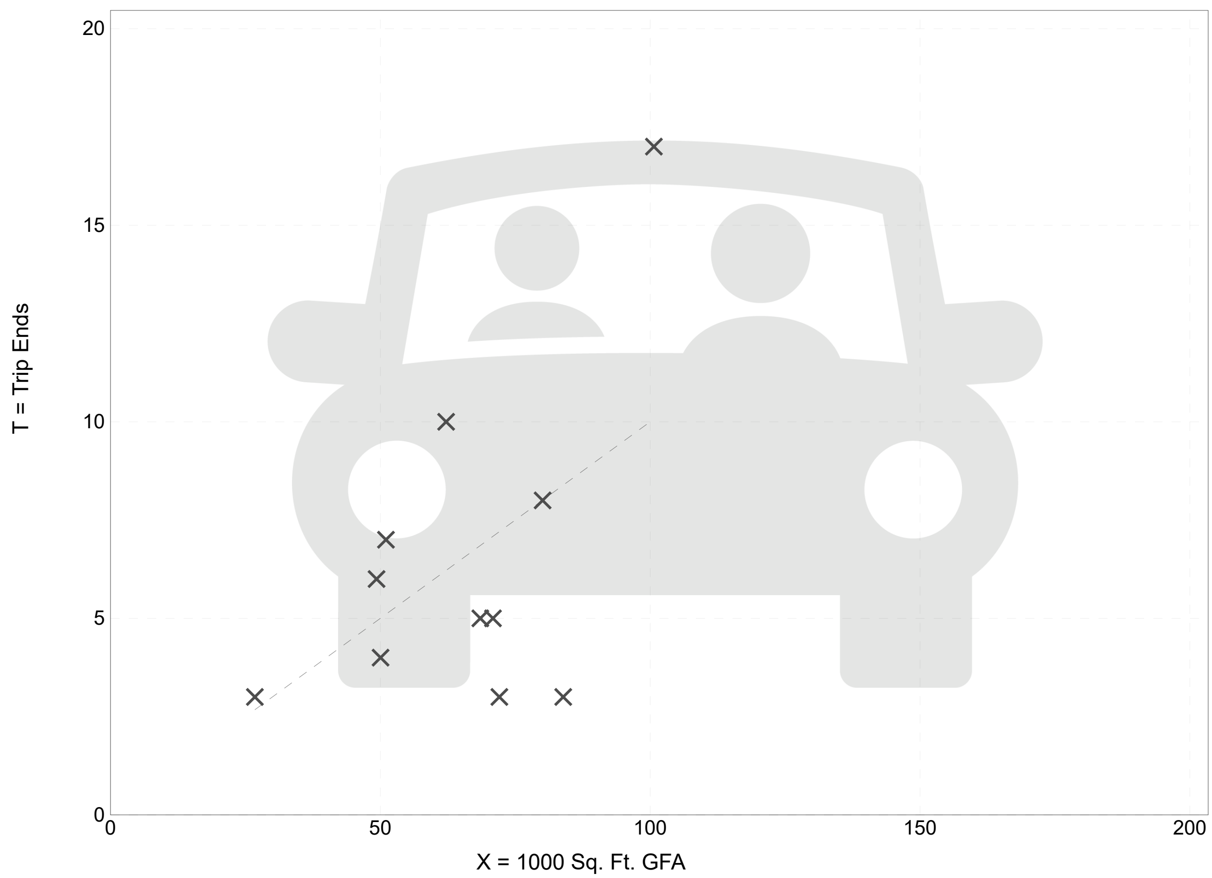
Mini-Warehouse
(151)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday, Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	11
Avg. 1000 Sq. Ft. GFA:	65
Directional Distribution:	60% entering, 40% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
0.10	0.04 - 0.17	0.05

Data Plot and Equation



× **Study Site** - - - - **Average Rate**

Fitted Curve Equation: Not Given **R^2= *******

Trip Gen Manual, 10th Ed + Supplement • Institute of Transportation Engineers

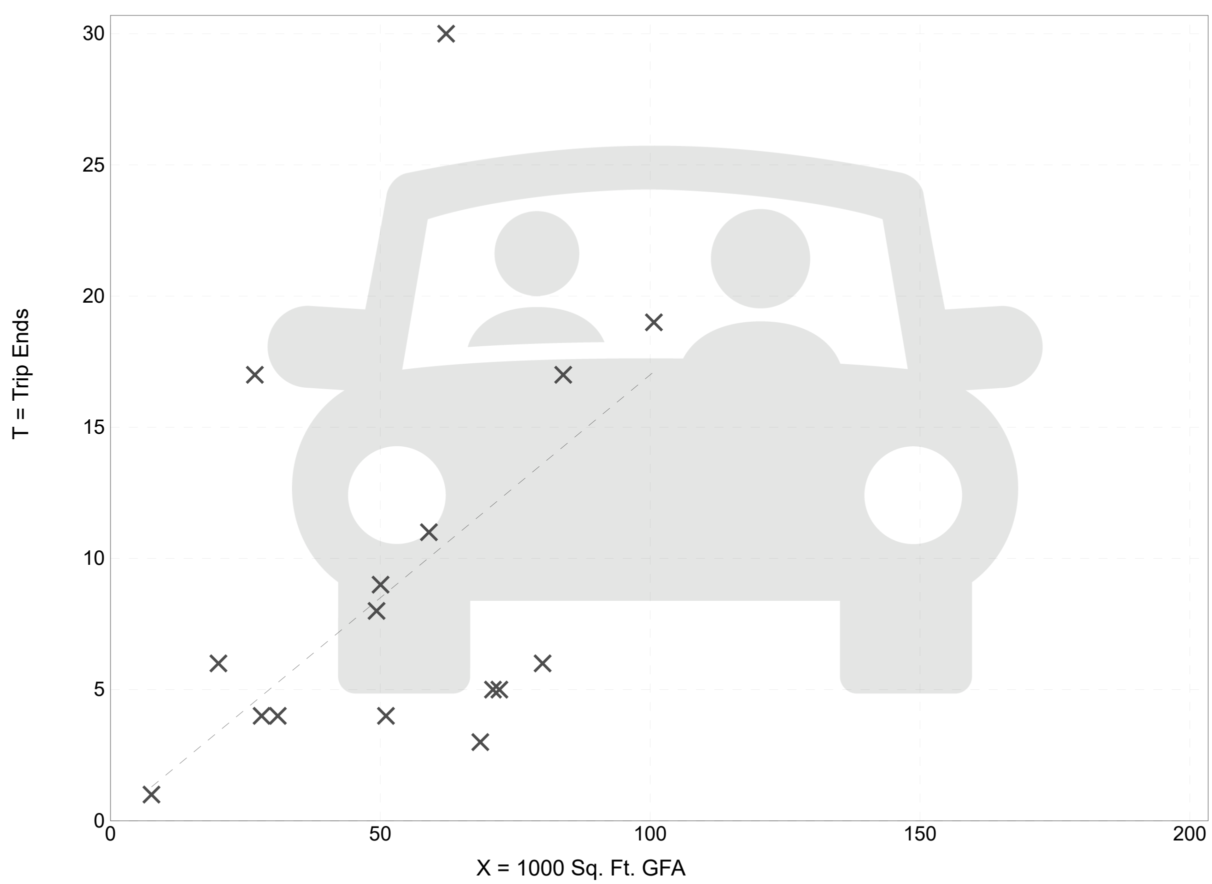
Mini-Warehouse
(151)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday, Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	16
Avg. 1000 Sq. Ft. GFA:	54
Directional Distribution:	47% entering, 53% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
0.17	0.04 - 0.64	0.14

Data Plot and Equation



Study Site **Average Rate**

Fitted Curve Equation: Not Given **R^2= ******

Trip Gen Manual, 10th Ed + Supplement • Institute of Transportation Engineers

CHARBONNEAU
ENGINEERING LLC

MEMORANDUM

Date: March 2, 2020

To: Mike Odren, RLA
Associate Principal
Olson Engineering, Inc.
222 East Evergreen Blvd
Vancouver WA 98660

From: Frank Charbonneau, PE, PTOE

Subject: Trip Generation Assessment FL2024
Minit Management Development
NW Paradise Park Road, La Center

This memo will serve as the trip generation assessment documenting the number of vehicular trips that will be produced by the proposed Minit Management development. The four acre site at address #2814 NW 319th Street is located in the northeast quadrant of NW La Center Road and the I-5 northbound on-ramp.

The development project will demolish the existing convenience store and gas station facilities and construct several new buildings consisting of 11,600 square feet of general retail, fast foot restaurant with drive-through totaling 2,800 square feet, convenience market with coffee drive-through totaling 4,510 square feet, and a 101 unit hotel. Parking on the site for 184 spaces will be provided, including eight ADA parking stalls. A copy of the project's site plan is attached to this memo.

The site we be served by three driveway accesses connecting to the perimeter road (NW Paradise Park Road) on the property's north and east sides. The nearest major intersections include NW La Center Road at the I-5 northbound off-ramp which is configured as a round-about and NW Paradise Park Road at NW La Center Road. This intersection is controlled by stop signing on the northbound Paradise Park Road approach and on the southbound Paradise Road approach.

The City of La Center issued a pre-application conference report (2019-018-PAC) dated June 11, 2019 documenting the application's process and requirements. The staff report detailed that the development agreement between the City and Minit Management LLC dated March 2016 vested a total of 199 PM peak hour trips for the site. As a result it was necessary to submit a trip generation assessment to verify the trip projection.

The number of trips were calculated based on the proposed building uses and sizes. Trip credits were applied for the existing facilities that will be demolished including the convenience market and gas station and a cardlock fueling station. The trip calculations were determined for the weekday average daily traffic (ADT) and the weekday AM and PM peak hours.

The analysis used the ITE Trip Generation manual (10th edition, year 2017).

For the proposed site uses several ITE land use categories were applied including #310 (Hotel), #820 (shopping center), #852 (convenience market), #934 (fast food restaurant with drive-through), and #938 (coffee drive-through). For the existing uses ITE code #853 for convenience market was used and historical rates for Pacific Pride Cardlock were applied for the cardlock fueling station.

A summary of the site's trip generation is provided in the following tables. Table 1 provides the trip generation for the site's existing uses. Table 2 provides the trip generation for the proposed site uses. Table 3 lists the net site trips for the development.

Table 1 Existing Land Uses Trip Generation Summary

ITE Land Use	Units	Weekday						
		ADT	AM Peak Hour			PM Peak Hour		
			Total	Enter	Exit	Total	Enter	Exit
Convenience Mkt with Gas (#853)	6 fueling posiitons							
Generation Rate [1]		322.50	20.76	50%	50%	23.04	50%	50%
Total Driveway Trips		**1,935**	**125**	63	62	**138**	69	69
Pass-By Trips [2] (AM Peak=63%; PM Peak=66%)			79	40	39	91	46	45
New Site Trips			**46**	23	23	**47**	23	24
Cardlock Fueling Station	12 fueling positions							
Generation Rate [3]			4.44	50%	50%	2.96	50%	50%
Total Driveway Trips		**1445**	**53**	27	26	**36**	18	18
Pass-By Trips [2] (AM Peak=58%; PM Peak=42%)			31	16	15	15	8	7
New Trips			**22**	11	11	**21**	10	11
Total Site Trips			**178**	90	88	**174**	87	87
Pass-by Trips			**110**	56	54	**106**	54	52
New Trips [4]		**3,380**	**68**	34	34	**68**	33	35

1 Source: *Trip Generation* , 10th Edition, ITE, 2017, average rates.

2 Pass-by percentage based on *Trip Generation Handbook, 3nd Edition* , ITE, 2017.

3 Source: Independent surveys at Tarr Inc. Pacific Pride. AM trip rate = 1.5x calculated PM trip rate, ADT = 70% of ITE #944 Gas Station Rate.

4 New Trips = Total Trips - Internal Trips - Pass-by Trips.

Table 2 Proposed Land Uses Trip Generation Summary

ITE Land Use	Units	Weekday						
		ADT	AM Peak Hour			PM Peak Hour		
			Total	Enter	Exit	Total	Enter	Exit
Convenience Mkt [Open 15-16 hours] (#852)	4,410 sq. ft.							
Generation Rate [1,2]		345.70	31.02	50%	50%	34.57	49%	51%
Total Driveway Trips		**1,525**	**137**	69	68	**152**	74	78
Internal Trips [3] (AM Peak=16%; PM Peak=36%)			22	11	11	55	27	28
Pass-By Trips [4] (AM Peak=63%; PM Peak=66%)			72	36	36	64	31	33
New Site Trips		**1,525**	**43**	22	21	**33**	16	17
Shopping Center (#820)	11,600 sq. ft.							
Generation Rate [2]		37.75	0.94	62%	38%	3.81	48%	52%
Total Driveway Trips		**438**	**11**	7	4	**44**	21	23
Internal Trips [3] (AM Peak=16%; PM Peak=36%)			2	1	1	16	8	8
Pass-By Trips [4] (AM Peak=N/A; PM Peak=34%)						10	5	5
New Site Trips [4]		**438**	**9**	6	3	**18**	8	10
Hotel (#310)	101 rooms							
Generation Rate [2]		8.36	0.47	59%	41%	0.60	51%	49%
Total Driveway Trips		**844**	**47**	28	19	**61**	31	30
Internal Trips [3] (AM Peak=16%; PM Peak=36%)			8	4	4	22	11	11
New Site Trips			39	24	15	39	20	19
Fast-Food with Drive-Through (#934)	2,800 sq. ft.							
Generation Rate [2]		470.95	40.19	51%	49%	32.67	52%	48%
Total Driveway Trips		**1,319**	**113**	58	55	**91**	48	43
Internal Trips [3] (AM Peak=16%; PM Peak=36%)			19	10	9	33	17	16
Pass-By Trips [4] (AM Peak=49%; PM Peak=50%)			46	24	22	29	15	14
New Trips			**48**	24	24	**29**	16	13
Coffee/Donut Shop with Drive-Through & No Indoor Seating (#938)	100 sq. ft.							
Generation Rate [2]		2000.00	337.04	50%	50%	83.33	50%	50%
Total Driveway Trips		**200**	**34**	17	17	**8**	4	4
Internal Trips [3] (AM Peak=16%; PM Peak=36%)		0	6	3	3	3	2	1
Pass-By Trips [4,5] (AM Peak=83%; PM Peak=83%)		166	23	12	11	4	2	2
New Site Trips		**34**	**5**	2	3	**1**	0	1
Total Site Trips		**4,326**	**342**	179	163	**356**	178	178
Internal Trips			**57**	29	28	**129**	65	64
Pass-by Trips			**141**	72	69	**107**	53	54
New Trips			**144**	78	66	**120**	60	60

1 ADT trip rate estimated as ten times the PM peak hour trip rate.

2 Source: *Trip Generation*, 10th Edition, ITE, 2017, average rates.

3 Internal capture calculated with unconstrained internal capture rates presented in the Center for Urban Transportation Research (CUTR) *Trip Internalization in Multi-Use Developments*, April 2014, FDOT.

4 Pass-by percentage based on Trip Generation Handbook, 3nd Edition, ITE, 2017.

5 The weekday PM peak pass-by rate used to calculate the daily and weekday AM peak pass-by trips.

6 New Trips = Total Trips - Internal Trips - Pass-by Trips.

Table 3 presents the net trip generation results (proposed site trips – existing site trips) for the development project. When the new facility is developed it is projected that the site will generate a net of 76 trips in the AM peak hour 52 trips in the PM peak hour. The ADT is projected to increase by 946 trips per day.

Table 3 Net New Trips

Site Uses	Weekday Peak Hour						Weekday ADT
	AM Peak Hour			PM Peak Hour			
	Total	Enter	Exit	Total	Enter	Exit	
Proposed Site [1]	**144**	78	66	**120**	60	60	4,326
Existing Site [2]	**-68**	-34	-34	**-68**	-33	-35	3,380
Net New Trips [3]	**76**	44	32	**52**	27	25	946

[1] Refer to Table 2.

[2] Refer to Table 1.

[3] Net New Trips = Proposed Site Trips - Existing Site Trips.

It is recommended that the City of La Center support the proposed development without the application of traffic impact fees as the projected number of site trips falls below the vested number of peak hour trips (199 trips) identified in the City's development agreement with Minit Management.

If you should need any additional traffic engineering support on this project or if there are any further questions, please contact Frank Charbonneau, PE, PTOE at 503.293.1118 or email Frank@CharbonneauEngineer.com.

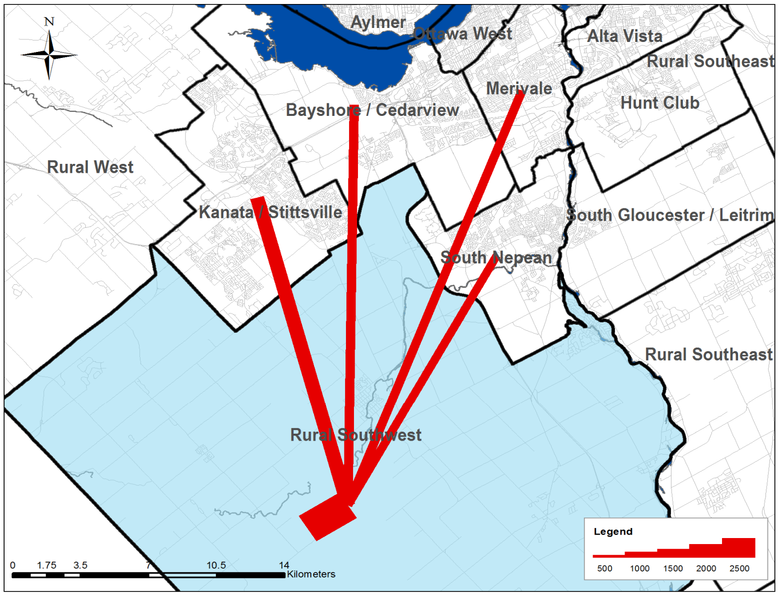
Attachment

- Site Plan

Travel Patterns

Top Five Destinations of Trips from Rural Southwest

AM Peak Period



Summary of Trips to and from Rural Southwest

AM Peak Period (6:30 - 8:59) Districts	Destinations of Trips From District	% Total	Origins of Trips To District	% Total
Ottawa Centre	620	5%	40	0%
Ottawa Inner Area	580	5%	150	2%
Ottawa East	120	1%	20	0%
Beacon Hill	90	1%	0	0%
Alta Vista	690	6%	160	2%
Hunt Club	220	2%	180	2%
Merivale	840	7%	200	2%
Ottawa West	400	3%	80	1%
Bayshore / Cedarview	810	7%	190	2%
Orléans	70	1%	70	1%
Rural East	0	0%	20	0%
Rural Southeast	390	3%	520	6%
South Gloucester / Leitrim	220	2%	120	1%
South Nepean	970	8%	580	7%
Rural Southwest	4,280	34%	4,280	53%
Kanata / Stittsville	1,850	15%	1,130	14%
Rural West	80	1%	160	2%
Île de Hull	120	1%	0	0%
Hull Périphérie	70	1%	30	0%
Plateau	0	0%	0	0%
Aylmer	0	0%	60	1%
Rural Northwest	0	0%	0	0%
Pointe Gatineau	0	0%	10	0%
Gatineau Est	0	0%	10	0%
Rural Northeast	0	0%	0	0%
Buckingham / Masson-Angers	0	0%	0	0%
Ontario Sub-Total:	12,230	98%	7,900	99%
Québec Sub-Total:	190	2%	110	1%
Total:	12,420	100%	8,010	100%

Trips by Trip Purpose

24 Hours	From District		To District		Within District	
Work or related	7,730	27%	3,170	11%	1,930	11%
School	2,200	8%	1,000	4%	2,640	15%
Shopping	3,390	12%	1,450	5%	1,610	9%
Leisure	3,560	13%	2,420	9%	1,700	9%
Medical	1,000	4%	660	2%	130	1%
Pick-up / drive passenger	1,980	7%	1,250	4%	750	4%
Return Home	7,290	26%	17,280	61%	7,960	44%
Other	1,130	4%	930	3%	1,250	7%
Total:	28,280	100%	28,160	100%	17,970	100%

AM Peak (06:30 - 08:59)	From District		To District		Within District	
Work or related	4,820	59%	1,900	51%	1,110	26%
School	1,830	22%	960	26%	2,290	54%
Shopping	140	2%	20	1%	40	1%
Leisure	280	3%	220	6%	90	2%
Medical	210	3%	90	2%	0	0%
Pick-up / drive passenger	500	6%	230	6%	290	7%
Return Home	130	2%	190	5%	180	4%
Other	240	3%	80	2%	280	7%
Total:	8,150	100%	3,690	100%	4,280	100%

PM Peak (15:30 - 17:59)	From District		To District		Within District	
Work or related	260	5%	120	1%	60	2%
School	50	1%	0	0%	0	0%
Shopping	480	10%	390	5%	250	7%
Leisure	940	19%	760	9%	300	9%
Medical	10	0%	10	0%	30	1%
Pick-up / drive passenger	550	11%	360	4%	100	3%
Return Home	2,410	48%	6,370	77%	2,480	73%
Other	290	6%	220	3%	180	5%
Total:	4,990	100%	8,230	100%	3,400	100%

Peak Period (%)	Total:	% of 24 Hours	Within District (%)
24 Hours	74,410		24%
AM Peak Period	16,120	22%	27%
PM Peak Period	16,620	22%	20%

Trips by Primary Travel Mode

24 Hours	From District		To District		Within District	
Auto Driver	20,550	73%	20,370	72%	9,040	50%
Auto Passenger	4,420	16%	4,490	16%	2,460	14%
Transit	1,100	4%	1,130	4%	60	0%
Bicycle	60	0%	80	0%	250	1%
Walk	100	0%	120	0%	1,630	9%
Other	2,030	7%	1,960	7%	4,530	25%
Total:	28,260	100%	28,150	100%	17,970	100%

AM Peak (06:30 - 08:59)	From District		To District		Within District	
Auto Driver	5,620	69%	2,280	61%	1,630	38%
Auto Passenger	910	11%	340	9%	420	10%
Transit	410	5%	270	7%	10	0%
Bicycle	20	0%	20	1%	30	1%
Walk	40	0%	20	1%	190	4%
Other	1,150	14%	800	21%	1,990	47%
Total:	8,150	100%	3,730	100%	4,270	100%

PM Peak (15:30 - 17:59)	From District		To District		Within District	
Auto Driver	3,620	73%	6,060	74%	1,660	49%
Auto Passenger	860	17%	1,430	17%	510	15%
Transit	290	6%	430	5%	30	1%
Bicycle	40	1%	20	0%	80	2%
Walk	0	0%	80	1%	330	10%
Other	180	4%	220	3%	780	23%
Total:	4,990	100%	8,240	100%	3,390	100%

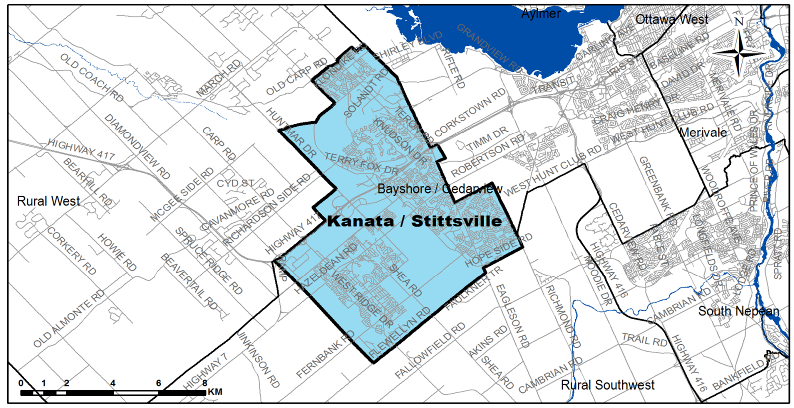
Avg Vehicle Occupancy	From District	To District	Within District
24 Hours	1.22	1.22	1.27
AM Peak Period	1.16	1.15	1.26
PM Peak Period	1.24	1.24	1.31

Transit Modal Split	From District	To District	Within District
24 Hours	4%	4%	1%
AM Peak Period	6%	9%	0%
PM Peak Period	6%	5%	1%

Kanata - Stittsville

Demographic Characteristics

Population	105,210	Actively Travelled	83,460
Employed Population	49,640	Number of Vehicles	64,540
Households	38,010	Area (km^2)	82.6



Occupation Status (age 5+)	Male	Female	Total
Full Time Employed	24,670	19,590	44,260
Part Time Employed	1,540	3,840	5,380
Student	13,630	13,410	27,040
Retiree	6,480	8,350	14,820
Unemployed	850	940	1,790
Homemaker	160	3,310	3,470
Other	350	1,010	1,360
Total:	47,690	50,440	98,120

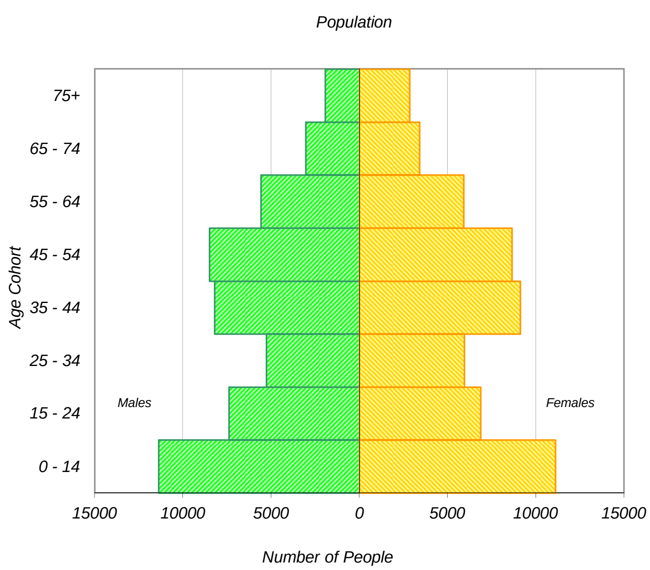
Traveller Characteristics	Male	Female	Total
Transit Pass Holders	5,940	6,920	12,860
Licensed Drivers	36,280	36,790	73,070
Telecommuters	200	380	580
Trips made by residents	135,300	143,330	278,630

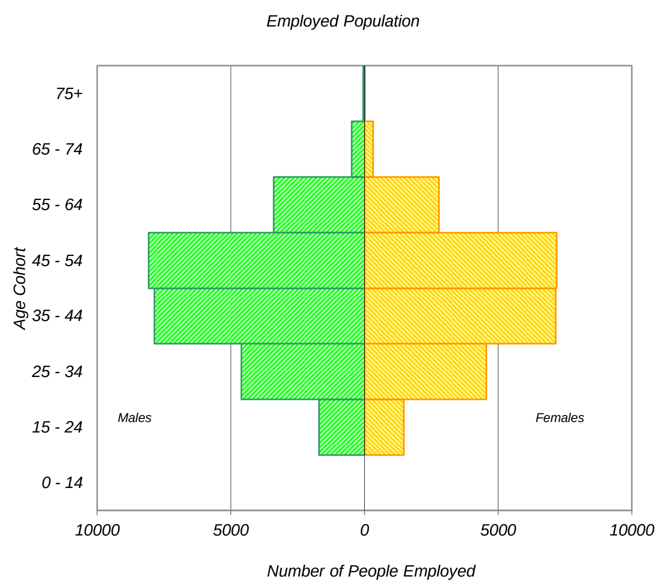
Household Size		
1 person	5,810	15%
2 persons	11,660	31%
3 persons	7,490	20%
4 persons	8,890	23%
5+ persons	4,160	11%
Total:	38,010	100%

Households by Vehicle Availability		
0 vehicles	1,050	3%
1 vehicle	14,090	37%
2 vehicles	19,110	50%
3 vehicles	3,000	8%
4+ vehicles	770	2%
Total:	38,010	100%

Selected Indicators	
Daily Trips per Person (age 5+)	2.84
Vehicles per Person	0.61
Number of Persons per Household	2.77
Daily Trips per Household	7.33
Vehicles per Household	1.70
Workers per Household	1.31
Population Density (Pop/km2)	1270

Households by Dwelling Type		
Single-detached	21,610	57%
Semi-detached	3,890	10%
Townhouse	10,550	28%
Apartment/Condo	1,960	5%
Total:	38,010	100%





* In 2005 data was only collected for household members aged 11+ therefore these results cannot be compared to the 2011 data.

Appendix E – MMLOS Analyses

Multi-Modal Level of Service - Segments Form

Consultant	**IBI Group**	**Project**	**2095 Dilworth Road**
Scenario	**Existing & Future Conditions**	**Date**	**08-Jun-21**
Comments			

SEGMENTS		**Dilworth Road**	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Section 7	Section 8	Section 9
Pedestrian	Sidewalk Width Boulevard Width	**-**	no sidewalk n/a								
	Avg Daily Curb Lane Traffic Volume		≤ 3000								
	Operating Speed On-Street Parking		> 60 km/h no								
	Exposure to Traffic PLoS		**F**	-	-	-	-	-	-	-	-
	Effective Sidewalk Width										
	Pedestrian Volume										
	Crowding PLoS		-	-	-	-	-	-	-	-	-
	Level of Service		**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**
Bicycle	Type of Cycling Facility	**F**	Mixed Traffic								
	Number of Travel Lanes		≤ 2 (no centreline)								
	Operating Speed		≥ 60 km/h								
	# of Lanes & Operating Speed LoS		**F**	-	-	-	-	-	-	-	-
	Bike Lane (+ Parking Lane) Width										
	Bike Lane Width LoS		-	-	-	-	-	-	-	-	-
	Bike Lane Blockages										
	Blockage LoS		-	-	-	-	-	-	-	-	-
	Median Refuge Width (no median = < 1.8 m)		< 1.8 m refuge								
	No. of Lanes at Unsignalized Crossing		≤ 3 lanes								
	Sidestreet Operating Speed		≥ 65 km/h								
	Unsignalized Crossing - Lowest LoS		**E**	-	-	-	-	-	-	-	-
	Level of Service		**F**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**
Transit	Facility Type	**-**									
	Friction or Ratio Transit:Posted Speed										
	Level of Service		**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**
Truck	Truck Lane Width	**C**	≤ 3.5 m								
	Travel Lanes per Direction		1								
	Level of Service		**C**	**-**	**-**	**-**	**-**	**-**	**-**	**-**	**-**

Appendix F – Intersection Control Warrants

OTM BOOK 12* - TRAFFIC SIGNAL WARRANT

Project: 2095 Dilworth Road **Date:** July 13, 2021

Project #: 134297

Location: Dilworth Road at Hwy 416 SB On/Off Ramp
(Major Roadway) *(Minor Roadway)*

Orientation: East/West North/South

Municipality: City of Ottawa **Scenario:** Future (2027) Total Traffic

Justification 1 - Minimum Vehicle Volume

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, all approaches	480	720	600	900	577 96%	289 48%	289 48%	289 48%	799 100%	400 67%	400 67%	400 67%	68%
B. Vehicle volume along minor roads	120	170	180	255	114 63%	57 32%	57 32%	57 32%	303 100%	151 84%	151 84%	151 84%	64%

Justification 2 - Delay to Cross Traffic

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, along artery	480	720	600	900	463 77%	232 39%	232 39%	232 39%	496 83%	248 41%	248 41%	248 41%	50%
B. Combined vehicle and pedestrian volume crossing artery from minor roads	50	70	50	70	74 100%	37 74%	37 74%	37 74%	105 100%	53 100%	53 100%	53 100%	90%

Justification 3 - Volume/Delay Combination

JUSTIFICATION	SATISFIED TO 80% OR MORE?	BOTH SATISFIED TO 80% OR MORE?
Justification 1 - Minimum Vehicular Volume	NO	NO
Justification 2 - Delay to Cross Traffic	NO	

Justification 7 - Projected Volumes

WARRANT	DESCRIPTION	MINIMUM REQUIREMENT				COMPLIANCE		
		FREE FLOW	RESTRICTED FLOW	ADJUSTED FREE FLOW	ADJUSTED RESTRICTED FLOW	SECTIONAL		ENTIRE %
						AHV	%	
1. MINIMUM VEHICULAR VOLUME	A. Vehicle volumes, all approaches (Average Hour)	480	720	720	1080	344	48%	48%
	B. Vehicle volume along minor roads (Average Hour)	120	170	216	306	104	48%	
2. DELAY TO CROSS TRAFFIC	A. Vehicle volumes, along artery (Average Hour)	480	720	720	1080	240	33%	33%
	B. Combined vehicle and pedestrian volume crossing artery from minor roads (Average Hour)	50	75	60	90	45	75%	

Projected Traffic Volumes:

Average Hourly Volume (AHV) Equation: AHV = (amPHV + pmPHV)/4

AM Peak Hour Volumes

			↖	67	
40	0	74	←	102	
↙	↓	↘	↙	0	
	27	↗	↖	↑	↗
	267	→	0	0	0
	0	↘			

PM Peak Hour Volumes

			↖	190	
197	0	105	←	158	
↙	↓	↘	↙	0	
	60	↗	↖	↑	↗
	89	→	0	0	0
	0	↘			

Average Hourly Volumes (AHV)

			↖	64	
59	0	45	←	65	
↙	↓	↘	↙	0	
	22	↗	↖	↑	↗
	89	→	0	0	0
	0	↘			

Eight Hour Traffic Volumes:

Hour	Major Road						Minor Road						Ped*
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	
7:00 AM	27	267	0	0	102	67	0	0	0	74	0	40	0
8:00 AM	13	133	0	0	51	34	0	0	0	37	0	20	0
9:00 AM	13	133	0	0	51	34	0	0	0	37	0	20	0
10:00 AM	13	133	0	0	51	34	0	0	0	37	0	20	0
3:00 PM	60	89	0	0	158	190	0	0	0	105	0	197	0
4:00 PM	30	45	0	0	79	95	0	0	0	53	0	99	0
5:00 PM	30	45	0	0	79	95	0	0	0	53	0	99	0
6:00 PM	30	45	0	0	79	95	0	0	0	53	0	99	0

** Number of pedestrians crossing the major road*

Notes:

1. Vehicle volume warrant (1A) and (2A) for intersections of roadways having two or more moving lanes in one direction should be 25% higher than the values given above.

2+ Lanes per Direction

2. Warrant values for free flow apply when the 85th percentile speed of artery traffic equals or exceeds 70 km/h or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000. Warrant values for restricted flow apply to large urban communities when the 85th percentile speed of artery traffic does not exceed 70 km/h.

Free Flow

3. The lowest sectional percentage governs the entire warrant.

4. For "T" intersections the warrant values for the minor road should be increased by 50% (Warrant 1B only).

3-legged Intersection

5. All flow values for Justification 1 and 2 are to be increased by 20% in the case of new intersections, Justification 3 is to only be used for existing intersections and all flow values for Warrant 1 and Warrant 2 of Justification 7 are to be increased by 20% for existing intersections and by 50% in the case of new intersections.

Existing Intersection

6. The crossing volumes are defined as the sum of:
   (a) Left-turns from both minor road approaches.
   (b) The heaviest through volume from the minor road.
   (c) 50% of the heavier left turn movement from major road when both of the following are met:
      (i) the left-turn volume >120 vph
      (ii) the left-turn volume plus the opposing volume >720 vph
   (d) Pedestrians crossing the main road.

CONCLUSION: The intersection does NOT meet the minimum warrants for traffic control signals.

* "Ontario Traffic Manual, Book 12 (March 2012)", Ontario Ministry of Transportation.

IBI

OTM BOOK 12* - TRAFFIC SIGNAL WARRANT

Project: 2095 Dilworth Road **Date:** July 13, 2021

Project #: 134297

Location: Dilworth Road (*Major Roadway*) at Hwy 416 NB On/Off Ramp (*Minor Roadway*)

Orientation: East/West North/South

Municipality: City of Ottawa **Scenario:** Future (2027) Total Traffic

Justification 1 - Minimum Vehicle Volume

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, all approaches	480	720	600	900	679 100%	340 57%	340 57%	340 57%	656 100%	328 55%	328 55%	328 55%	67%
B. Vehicle volume along minor roads	120	170	180	255	61 34%	31 17%	31 17%	31 17%	64 36%	32 18%	32 18%	32 18%	22%

Justification 2 - Delay to Cross Traffic

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, along artery	480	720	600	900	618 100%	309 51%	309 51%	309 51%	592 99%	296 49%	296 49%	296 49%	63%
B. Combined vehicle and pedestrian volume crossing artery from minor roads	50	70	50	70	13 25%	6 13%	6 13%	6 13%	32 64%	16 32%	16 32%	16 32%	28%

Justification 3 - Volume/Delay Combination

JUSTIFICATION	SATISFIED TO 80% OR MORE?	BOTH SATISFIED TO 80% OR MORE?
Justification 1 - Minimum Vehicular Volume	NO	NO
Justification 2 - Delay to Cross Traffic	NO	

Justification 7 - Projected Volumes

WARRANT	DESCRIPTION	MINIMUM REQUIREMENT				COMPLIANCE		
		FREE FLOW	RESTRICTED FLOW	ADJUSTED FREE FLOW	ADJUSTED RESTRICTED FLOW	SECTIONAL		ENTIRE %
						AHV	%	
1. MINIMUM VEHICULAR VOLUME	A. Vehicle volumes, all approaches (Average Hour)	480	720	720	1080	333	46%	14%
	B. Vehicle volume along minor roads (Average Hour)	120	170	216	306	31	14%	
2. DELAY TO CROSS TRAFFIC	A. Vehicle volumes, along artery (Average Hour)	480	720	720	1080	302	42%	18%
	B. Combined vehicle and pedestrian volume crossing artery from minor roads (Average Hour)	50	75	60	90	11	18%	

Projected Traffic Volumes:

Average Hourly Volume (AHV) Equation: AHV = (amPHV + pmPHV)/4

AM Peak Hour Volumes

			↖	45	
0	0	0	←	105	
↙	↓	↘	↙	108	
	0	↗	↖	↑	↗
	102	→	13	0	48
	258	↘			

PM Peak Hour Volumes

			↖	126	
0	0	0	←	184	
↙	↓	↘	↙	72	
	0	↗	↖	↑	↗
	118	→	32	0	32
	91	↘			

Average Hourly Volumes (AHV)

			↖	43	
0	0	0	←	72	
↙	↓	↘	↙	45	
	0	↗	↖	↑	↗
	55	→	11	0	20
	87	↘			

Eight Hour Traffic Volumes:

Hour	Major Road						Minor Road						Ped*
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	
7:00 AM	0	102	258	108	105	45	13	0	48	0	0	0	0
8:00 AM	0	51	129	54	53	22	6	0	24	0	0	0	0
9:00 AM	0	51	129	54	53	22	6	0	24	0	0	0	0
10:00 AM	0	51	129	54	53	22	6	0	24	0	0	0	0
3:00 PM	0	118	91	72	184	126	32	0	32	0	0	0	0
4:00 PM	0	59	46	36	92	63	16	0	16	0	0	0	0
5:00 PM	0	59	46	36	92	63	16	0	16	0	0	0	0
6:00 PM	0	59	46	36	92	63	16	0	16	0	0	0	0

** Number of pedestrians crossing the major road*

Notes:

1. Vehicle volume warrant (1A) and (2A) for intersections of roadways having two or more moving lanes in one direction should be 25% higher than the values given above.

2+ Lanes per Direction

2. Warrant values for free flow apply when the 85th percentile speed of artery traffic equals or exceeds 70 km/h or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000. Warrant values for restricted flow apply to large urban communities when the 85th percentile speed of artery traffic does not exceed 70 km/h.

Free Flow

3. The lowest sectional percentage governs the entire warrant.

4. For "T" intersections the warrant values for the minor road should be increased by 50% (Warrant 1B only).

3-legged Intersection

5. All flow values for Justification 1 and 2 are to be increased by 20% in the case of new intersections, Justification 3 is to only be used for existing intersections and all flow values for Warrant 1 and Warrant 2 of Justification 7 are to be increased by 20% for existing intersections and by 50% in the case of new intersections.

Existing Intersection

6. The crossing volumes are defined as the sum of:
   - (a) Left-turns from both minor road approaches.
   - (b) The heaviest through volume from the minor road.
   - (c) 50% of the heavier left turn movement from major road when both of the following are met:
     - (i) the left-turn volume >120 vph
     - (ii) the left-turn volume plus the opposing volume >720 vph
   - (d) Pedestrians crossing the main road.

CONCLUSION: **The intersection does NOT meet the minimum warrants for traffic control signals.**

* "Ontario Traffic Manual, Book 12 (March 2012)", Ontario Ministry of Transportation.

OTM BOOK 12* - TRAFFIC SIGNAL WARRANT

Project: 2095 Dilworth Road **Date:** July 13, 2021

Project #: 134297

Location: Dilworth Road (*Major Roadway*) at Site Access #1 (*Minor Roadway*)

Orientation: East/West North/South

Municipality: City of Ottawa **Scenario:** Future (2027) Total Traffic

Justification 1 - Minimum Vehicle Volume

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, all approaches	480	720	720	1080	408 57%	204 28%	204 28%	204 28%	533 74%	266 37%	266 37%	266 37%	41%
B. Vehicle volume along minor roads	120	170	216	306	93 43%	47 22%	47 22%	47 22%	87 40%	43 20%	43 20%	43 20%	26%

Justification 2 - Delay to Cross Traffic

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, along artery	480	720	720	1080	315 44%	158 22%	158 22%	158 22%	446 62%	223 31%	223 31%	223 31%	33%
B. Combined vehicle and pedestrian volume crossing artery from minor roads	50	70	60	84	0 0%	0 0%	0 0%	0 0%	0 0%	0 0%	0 0%	0 0%	0%

Justification 3 - Volume/Delay Combination

JUSTIFICATION	SATISFIED TO 80% OR MORE?	BOTH SATISFIED TO 80% OR MORE?
Justification 1 - Minimum Vehicular Volume	N/A	N/A
Justification 2 - Delay to Cross Traffic	N/A	

Justification 7 - Projected Volumes

WARRANT	DESCRIPTION	MINIMUM REQUIREMENT				COMPLIANCE		
		FREE FLOW	RESTRICTED FLOW	ADJUSTED FREE FLOW	ADJUSTED RESTRICTED FLOW	SECTIONAL AHV	SECTIONAL %	ENTIRE %
1. MINIMUM VEHICULAR VOLUME	A. Vehicle volumes, all approaches (Average Hour)	480	720	900	1350	235	26%	17%
	B. Vehicle volume along minor roads (Average Hour)	120	170	270	383	45	17%	
2. DELAY TO CROSS TRAFFIC	A. Vehicle volumes, along artery (Average Hour)	480	720	900	1350	190	21%	0%
	B. Combined vehicle and pedestrian volume crossing artery from minor roads (Average Hour)	50	75	75	113	0	0%	

Projected Traffic Volumes:

Average Hourly Volume (AHV) Equation: AHV = (amPHV + pmPHV)/4

AM Peak Hour Volumes

			↖	45	
93	0	0	←	120	
↙	↓	↘	↙	0	
	96	↗	↖	↑	↗
	55	→	0	0	0
	0	↘			

PM Peak Hour Volumes

			↖	126	
87	0	0	←	169	
↙	↓	↘	↙	0	
	90	↗	↖	↑	↗
	61	→	0	0	0
	0	↘			

Average Hourly Volumes (AHV)

			↖	43	
45	0	0	←	72	
↙	↓	↘	↙	0	
	46	↗	↖	↑	↗
	29	→	0	0	0
	0	↘			

Eight Hour Traffic Volumes:

Hour	Major Road						Minor Road						Ped*
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	
7:00 AM	96	55	0	0	120	45	0	0	0	0	0	93	0
8:00 AM	48	27	0	0	60	22	0	0	0	0	0	47	0
9:00 AM	48	27	0	0	60	22	0	0	0	0	0	47	0
10:00 AM	48	27	0	0	60	22	0	0	0	0	0	47	0
3:00 PM	90	61	0	0	169	126	0	0	0	0	0	87	0
4:00 PM	45	30	0	0	85	63	0	0	0	0	0	43	0
5:00 PM	45	30	0	0	85	63	0	0	0	0	0	43	0
6:00 PM	45	30	0	0	85	63	0	0	0	0	0	43	0

** Number of pedestrians crossing the major road*

Notes:

1. Vehicle volume warrant (1A) and (2A) for intersections of roadways having two or more moving lanes in one direction should be 25% higher than the values given above.

2+ Lanes per Direction

2. Warrant values for free flow apply when the 85th percentile speed of artery traffic equals or exceeds 70 km/h or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000. Warrant values for restricted flow apply to large urban communities when the 85th percentile speed of artery traffic does not exceed 70 km/h.

Free Flow

3. The lowest sectional percentage governs the entire warrant.

4. For "T" intersections the warrant values for the minor road should be increased by 50% (Warrant 1B only).

3-legged Intersection

5. All flow values for Justification 1 and 2 are to be increased by 20% in the case of new intersections, Justification 3 is to only be used for existing intersections and all flow values for Warrant 1 and Warrant 2 of Justification 7 are to be increased by 20% for existing intersections and by 50% in the case of new intersections.

New Intersection

6. The crossing volumes are defined as the sum of:
   - (a) Left-turns from both minor road approaches.
   - (b) The heaviest through volume from the minor road.
   - (c) 50% of the heavier left turn movement from major road when both of the following are met:
     - (i) the left-turn volume >120 vph
     - (ii) the left-turn volume plus the opposing volume >720 vph
   - (d) Pedestrians crossing the main road.

CONCLUSION: **The intersection does NOT meet the minimum warrants for traffic control signals.**

* "Ontario Traffic Manual, Book 12 (March 2012)", Ontario Ministry of Transportation.

IBI

OTM BOOK 12* - TRAFFIC SIGNAL WARRANT

Project: 2095 Dilworth Road **Date:** July 13, 2021

Project #: 134297

Location: Dilworth Road at Site Access #2

(Major Roadway) *(Minor Roadway)*

Orientation: East/West North/South

Municipality: City of Ottawa **Scenario:** Future (2027) Total Traffic

Justification 1 - Minimum Vehicle Volume

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, all approaches	480	720	720	1080	219 30%	110 15%	110 15%	110 15%	356 49%	178 25%	178 25%	178 25%	25%
B. Vehicle volume along minor roads	120	170	216	306	18 8%	9 4%	9 4%	9 4%	21 10%	11 5%	11 5%	11 5%	6%

Justification 2 - Delay to Cross Traffic

WARRANT	MINIMUM REQUIREMENT				COMPLIANCE								SECTIONAL PERCENT
	FREE FLOW	RESTR. FLOW	ADJUST. FREE FLOW	ADJUST. RESTR. FLOW	7:00 AM	8:00 AM	9:00 AM	10:00 AM	3:00 PM	4:00 PM	5:00 PM	6:00 PM	
A. Vehicle volumes, along artery	480	720	720	1080	202 28%	101 14%	101 14%	101 14%	335 47%	168 23%	168 23%	168 23%	23%
B. Combined vehicle and pedestrian volume crossing artery from minor roads	50	70	60	84	0 0%	0 0%	0 0%	0 0%	0 0%	0 0%	0 0%	0 0%	0%

Justification 3 - Volume/Delay Combination

JUSTIFICATION	SATISFIED TO 80% OR MORE?	BOTH SATISFIED TO 80% OR MORE?
Justification 1 - Minimum Vehicular Volume	N/A	N/A
Justification 2 - Delay to Cross Traffic	N/A	

Justification 7 - Projected Volumes

WARRANT	DESCRIPTION	MINIMUM REQUIREMENT				COMPLIANCE		
		FREE FLOW	RESTRICTED FLOW	ADJUSTED FREE FLOW	ADJUSTED RESTRICTED FLOW	SECTIONAL AHV	SECTIONAL %	ENTIRE %
1. MINIMUM VEHICULAR VOLUME	A. Vehicle volumes, all approaches (Average Hour)	480	720	900	1350	144	16%	4%
	B. Vehicle volume along minor roads (Average Hour)	120	170	270	383	10	4%	
2. DELAY TO CROSS TRAFFIC	A. Vehicle volumes, along artery (Average Hour)	480	720	900	1350	134	15%	0%
	B. Combined vehicle and pedestrian volume crossing artery from minor roads (Average Hour)	50	75	75	113	0	0%	

Projected Traffic Volumes:

Average Hourly Volume (AHV) Equation: AHV = (amPHV + pmPHV)/4

AM Peak Hour Volumes

			↖	45	
18	0	0	←	102	
↙	↓	↘	↙	0	
	18	↗	↖	↑	↗
	37	→	0	0	0
	0	↘			

PM Peak Hour Volumes

			↖	126	
21	0	0	←	148	
↙	↓	↘	↙	0	
	20	↗	↖	↑	↗
	41	→	0	0	0
	0	↘			

Average Hourly Volumes (AHV)

			↖	43	
10	0	0	←	63	
↙	↓	↘	↙	0	
	9	↗	↖	↑	↗
	19	→	0	0	0
	0	↘			

Eight Hour Traffic Volumes:

Hour	Major Road						Minor Road						Ped*
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	
7:00 AM	18	37	0	0	102	45	0	0	0	0	0	18	0
8:00 AM	9	18	0	0	51	22	0	0	0	0	0	9	0
9:00 AM	9	18	0	0	51	22	0	0	0	0	0	9	0
10:00 AM	9	18	0	0	51	22	0	0	0	0	0	9	0
3:00 PM	20	41	0	0	148	126	0	0	0	0	0	21	0
4:00 PM	10	20	0	0	74	63	0	0	0	0	0	11	0
5:00 PM	10	20	0	0	74	63	0	0	0	0	0	11	0
6:00 PM	10	20	0	0	74	63	0	0	0	0	0	11	0

** Number of pedestrians crossing the major road*

Notes:

1. Vehicle volume warrant (1A) and (2A) for intersections of roadways having two or more moving lanes in one direction should be 25% higher than the values given above.

2+ Lanes per Direction

2. Warrant values for free flow apply when the 85th percentile speed of artery traffic equals or exceeds 70 km/h or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000. Warrant values for restricted flow apply to large urban communities when the 85th percentile speed of artery traffic does not exceed 70 km/h.

Free Flow

3. The lowest sectional percentage governs the entire warrant.

4. For "T" intersections the warrant values for the minor road should be increased by 50% (Warrant 1B only).

3-legged Intersection

5. All flow values for Justification 1 and 2 are to be increased by 20% in the case of new intersections, Justification 3 is to only be used for existing intersections and all flow values for Warrant 1 and Warrant 2 of Justification 7 are to be increased by 20% for existing intersections and by 50% in the case of new intersections.

New Intersection

6. The crossing volumes are defined as the sum of:
   - (a) Left-turns from both minor road approaches.
   - (b) The heaviest through volume from the minor road.
   - (c) 50% of the heavier left turn movement from major road when both of the following are met:
     - (i) the left-turn volume >120 vph
     - (ii) the left-turn volume plus the opposing volume >720 vph
   - (d) Pedestrians crossing the main road.

CONCLUSION: The intersection does NOT meet the minimum warrants for traffic control signals.

* "Ontario Traffic Manual, Book 12 (March 2012)", Ontario Ministry of Transportation.

Appendix G – Intersection Capacity Analyses

Existing (2021) Traffic

Intersection						
Int Delay, s/veh	1.5					

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	24	233	30	5	1	35
Future Vol, veh/h	24	233	30	5	1	35
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	90	90	90	90	90	90
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	27	259	33	6	1	39

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	39	0	-	0	346	33
Stage 1	-	-	-	-	33	-
Stage 2	-	-	-	-	313	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1571	-	-	-	651	1041
Stage 1	-	-	-	-	989	-
Stage 2	-	-	-	-	741	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1571	-	-	-	640	1041
Mov Cap-2 Maneuver	-	-	-	-	640	-
Stage 1	-	-	-	-	972	-
Stage 2	-	-	-	-	741	-

Approach	EB	WB	SB
HCM Control Delay, s	0.7	0	8.7
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1571	-	-	-	1023
HCM Lane V/C Ratio	0.017	-	-	-	0.039
HCM Control Delay (s)	7.3	-	-	-	8.7
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0.1	-	-	-	0.1

Intersection	
Int Delay, s/veh	0.9

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	25	221	12	17	11	7
Future Vol, veh/h	25	221	12	17	11	7
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	90	90	90	90	90	90
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	28	246	13	19	12	8

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	274	0	73	28
Stage 1	-	-	-	-	28	-
Stage 2	-	-	-	-	45	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1289	-	931	1047
Stage 1	-	-	-	-	995	-
Stage 2	-	-	-	-	977	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1289	-	922	1047
Mov Cap-2 Maneuver	-	-	-	-	922	-
Stage 1	-	-	-	-	995	-
Stage 2	-	-	-	-	967	-

Approach	EB	WB	NB
HCM Control Delay, s	0	3.2	8.8
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	967	-	-	1289	-
HCM Lane V/C Ratio	0.021	-	-	0.01	-
HCM Control Delay (s)	8.8	-	-	7.8	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Intersection						
Int Delay, s/veh	5.7					

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	52	78	46	20	11	172
Future Vol, veh/h	52	78	46	20	11	172
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	90	90	90	90	90	90
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	58	87	51	22	12	191

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	73	0	-	0	254	51
Stage 1	-	-	-	-	51	-
Stage 2	-	-	-	-	203	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1527	-	-	-	735	1017
Stage 1	-	-	-	-	971	-
Stage 2	-	-	-	-	831	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1527	-	-	-	707	1017
Mov Cap-2 Maneuver	-	-	-	-	707	-
Stage 1	-	-	-	-	934	-
Stage 2	-	-	-	-	831	-

Approach	EB		WB		SB	
HCM Control Delay, s	3		0		9.6	
HCM LOS					A	

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1527	-	-	-	991
HCM Lane V/C Ratio	0.038	-	-	-	0.205
HCM Control Delay (s)	7.5	-	-	-	9.6
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0.1	-	-	-	0.8

Intersection						
Int Delay, s/veh	2.2					

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	22	78	4	33	27	13
Future Vol, veh/h	22	78	4	33	27	13
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	90	90	90	90	90	90
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	24	87	4	37	30	14

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	111	0	69	24
Stage 1	-	-	-	-	24	-
Stage 2	-	-	-	-	45	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1479	-	936	1052
Stage 1	-	-	-	-	999	-
Stage 2	-	-	-	-	977	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1479	-	933	1052
Mov Cap-2 Maneuver	-	-	-	-	933	-
Stage 1	-	-	-	-	999	-
Stage 2	-	-	-	-	974	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.8	8.9
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	969	-	-	1479	-
HCM Lane V/C Ratio	0.046	-	-	0.003	-
HCM Control Delay (s)	8.9	-	-	7.4	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Future (2022) Background Traffic

Intersection	
Int Delay, s/veh	1.5

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	24	239	30	5	1	36
Future Vol, veh/h	24	239	30	5	1	36
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	24	239	30	5	1	36

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	35	0	-	0	317	30
Stage 1	-	-	-	-	30	-
Stage 2	-	-	-	-	287	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1576	-	-	-	676	1044
Stage 1	-	-	-	-	993	-
Stage 2	-	-	-	-	762	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1576	-	-	-	666	1044
Mov Cap-2 Maneuver	-	-	-	-	666	-
Stage 1	-	-	-	-	978	-
Stage 2	-	-	-	-	762	-

Approach	EB	WB	SB
HCM Control Delay, s	0.7	0	8.6
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1576	-	-	-	1028
HCM Lane V/C Ratio	0.015	-	-	-	0.036
HCM Control Delay (s)	7.3	-	-	-	8.6
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0	-	-	-	0.1

Intersection						
Int Delay, s/veh	0.8					
Movement	**EBT**	**EBR**	**WBL**	**WBT**	**NBL**	**NBR**
Lane Configurations						
Traffic Vol, veh/h	26	227	12	18	11	7
Future Vol, veh/h	26	227	12	18	11	7
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	26	227	12	18	11	7

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	253	0	68	26
Stage 1	-	-	-	-	26	-
Stage 2	-	-	-	-	42	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1312	-	937	1050
Stage 1	-	-	-	-	997	-
Stage 2	-	-	-	-	980	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1312	-	929	1050
Mov Cap-2 Maneuver	-	-	-	-	929	-
Stage 1	-	-	-	-	997	-
Stage 2	-	-	-	-	971	-

Approach	EB	WB	NB
HCM Control Delay, s	0	3.1	8.8
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	973	-	-	1312	-
HCM Lane V/C Ratio	0.018	-	-	0.009	-
HCM Control Delay (s)	8.8	-	-	7.8	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Intersection	
Int Delay, s/veh	5.6

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	53	80	47	20	11	177
Future Vol, veh/h	53	80	47	20	11	177
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	53	80	47	20	11	177

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	67	0	-	0	233	47
Stage 1	-	-	-	-	47	-
Stage 2	-	-	-	-	186	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1535	-	-	-	755	1022
Stage 1	-	-	-	-	975	-
Stage 2	-	-	-	-	846	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1535	-	-	-	729	1022
Mov Cap-2 Maneuver	-	-	-	-	729	-
Stage 1	-	-	-	-	941	-
Stage 2	-	-	-	-	846	-

Approach	EB	WB	SB
HCM Control Delay, s	3	0	9.4
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1535	-	-	-	999
HCM Lane V/C Ratio	0.035	-	-	-	0.188
HCM Control Delay (s)	7.4	-	-	-	9.4
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0.1	-	-	-	0.7

Intersection	
Int Delay, s/veh	2.2

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	22	81	4	34	28	13
Future Vol, veh/h	22	81	4	34	28	13
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	22	81	4	34	28	13

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	103	0	64	22
Stage 1	-	-	-	-	22	-
Stage 2	-	-	-	-	42	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1489	-	942	1055
Stage 1	-	-	-	-	1001	-
Stage 2	-	-	-	-	980	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1489	-	939	1055
Mov Cap-2 Maneuver	-	-	-	-	939	-
Stage 1	-	-	-	-	1001	-
Stage 2	-	-	-	-	977	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.8	8.9
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	973	-	-	1489	-
HCM Lane V/C Ratio	0.042	-	-	0.003	-
HCM Control Delay (s)	8.9	-	-	7.4	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Future (2027) Background Traffic

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	27	267	34	6	1	40
Future Volume (vph)	27	267	34	6	1	40
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Storage Length (m)	50.0			50.0	0.0	0.0
Storage Lanes	1			1	1	0
Taper Length (m)	7.6				7.6	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt				0.850	0.868	
Flt Protected	0.950				0.999	
Satd. Flow (prot)	1695	1784	1784	1517	1547	0
Flt Permitted	0.950				0.999	
Satd. Flow (perm)	1695	1784	1784	1517	1547	0
Link Speed (k/h)		80	80		50	
Link Distance (m)		230.7	352.9		128.7	
Travel Time (s)		10.4	15.9		9.3	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	27	267	34	6	1	40
Shared Lane Traffic (%)						
Lane Group Flow (vph)	27	267	34	6	41	0
Sign Control		Free	Free		Stop	

Intersection Summary	
Area Type: Other	
Control Type: Unsignalized	
Intersection Capacity Utilization 24.8%	ICU Level of Service A
Analysis Period (min) 15	

Lane Group	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Volume (vph)	29	258	14	20	13	8
Future Volume (vph)	29	258	14	20	13	8
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Storage Length (m)		35.0	60.0		0.0	0.0
Storage Lanes		1	1		1	0
Taper Length (m)			7.6		7.6	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt		0.850			0.949	
Flt Protected			0.950		0.970	
Satd. Flow (prot)	1784	1517	1695	1784	1643	0
Flt Permitted			0.950		0.970	
Satd. Flow (perm)	1784	1517	1695	1784	1643	0
Link Speed (k/h)	80			80	50	
Link Distance (m)	352.9			112.7	118.0	
Travel Time (s)	15.9			5.1	8.5	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	29	258	14	20	13	8
Shared Lane Traffic (%)						
Lane Group Flow (vph)	29	258	14	20	21	0
Sign Control	Free			Free	Stop	

Intersection Summary

Area Type: Other
Control Type: Unsignalized
Intersection Capacity Utilization 26.9% ICU Level of Service A
Analysis Period (min) 15

Intersection	
Int Delay, s/veh	5.7

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	60	89	53	23	13	197
Future Vol, veh/h	60	89	53	23	13	197
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	60	89	53	23	13	197

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	76	0	-	0	262	53
Stage 1	-	-	-	-	53	-
Stage 2	-	-	-	-	209	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1523	-	-	-	727	1014
Stage 1	-	-	-	-	970	-
Stage 2	-	-	-	-	826	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1523	-	-	-	699	1014
Mov Cap-2 Maneuver	-	-	-	-	699	-
Stage 1	-	-	-	-	932	-
Stage 2	-	-	-	-	826	-

Approach	EB	WB	SB
HCM Control Delay, s	3	0	9.6
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1523	-	-	-	986
HCM Lane V/C Ratio	0.039	-	-	-	0.213
HCM Control Delay (s)	7.5	-	-	-	9.6
HCM Lane LOS	A	-	-	-	A
HCM 95th %tile Q(veh)	0.1	-	-	-	0.8

Intersection						
Int Delay, s/veh	2.2					

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	25	91	5	38	32	15
Future Vol, veh/h	25	91	5	38	32	15
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	25	91	5	38	32	15

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	116	0	73	25
Stage 1	-	-	-	-	25	-
Stage 2	-	-	-	-	48	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1473	-	931	1051
Stage 1	-	-	-	-	998	-
Stage 2	-	-	-	-	974	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1473	-	928	1051
Mov Cap-2 Maneuver	-	-	-	-	928	-
Stage 1	-	-	-	-	998	-
Stage 2	-	-	-	-	971	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0.9	8.9
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	964	-	-	1473	-
HCM Lane V/C Ratio	0.049	-	-	0.003	-
HCM Control Delay (s)	8.9	-	-	7.5	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.2	-	-	0	-

Future (2022) Total Traffic

Intersection	
Int Delay, s/veh	3.2

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	24	239	30	22	74	36
Future Vol, veh/h	24	239	30	22	74	36
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	24	239	30	22	74	36

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	52	0	-	0	317	30
Stage 1	-	-	-	-	30	-
Stage 2	-	-	-	-	287	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1554	-	-	-	676	1044
Stage 1	-	-	-	-	993	-
Stage 2	-	-	-	-	762	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1554	-	-	-	666	1044
Mov Cap-2 Maneuver	-	-	-	-	666	-
Stage 1	-	-	-	-	978	-
Stage 2	-	-	-	-	762	-

Approach	EB	WB	SB
HCM Control Delay, s	0.7	0	10.6
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1554	-	-	-	756
HCM Lane V/C Ratio	0.015	-	-	-	0.146
HCM Control Delay (s)	7.4	-	-	-	10.6
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0	-	-	-	0.5

Intersection	
Int Delay, s/veh	2.7

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	98	227	107	35	11	48
Future Vol, veh/h	98	227	107	35	11	48
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	98	227	107	35	11	48

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	325	0	347	98
Stage 1	-	-	-	-	98	-
Stage 2	-	-	-	-	249	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1235	-	650	958
Stage 1	-	-	-	-	926	-
Stage 2	-	-	-	-	792	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1235	-	593	958
Mov Cap-2 Maneuver	-	-	-	-	593	-
Stage 1	-	-	-	-	926	-
Stage 2	-	-	-	-	723	-

Approach	EB	WB	NB
HCM Control Delay, s	0	6.2	9.5
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	859	-	-	1235	-
HCM Lane V/C Ratio	0.069	-	-	0.087	-
HCM Control Delay (s)	9.5	-	-	8.2	-
HCM Lane LOS	A	-	-	A	-
HCM 95th %tile Q(veh)	0.2	-	-	0.3	-

Intersection	
Int Delay, s/veh	5.4

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	96	50	48	0	0	93
Future Vol, veh/h	96	50	48	0	0	93
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	96	50	48	0	0	93

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	48	0	-	0	290	48
Stage 1	-	-	-	-	48	-
Stage 2	-	-	-	-	242	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1559	-	-	-	701	1021
Stage 1	-	-	-	-	974	-
Stage 2	-	-	-	-	798	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1559	-	-	-	657	1021
Mov Cap-2 Maneuver	-	-	-	-	657	-
Stage 1	-	-	-	-	913	-
Stage 2	-	-	-	-	798	-

Approach	EB	WB	SB
HCM Control Delay, s	4.9	0	8.9
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1559	-	-	-	1021
HCM Lane V/C Ratio	0.062	-	-	-	0.091
HCM Control Delay (s)	7.5	0	-	-	8.9
HCM Lane LOS	A	A	-	-	A
HCM 95th %tile Q(veh)	0.2	-	-	-	0.3

Intersection	
Int Delay, s/veh	2.9

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	18	32	30	0	0	18
Future Vol, veh/h	18	32	30	0	0	18
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	18	32	30	0	0	18

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	30	0	-	0	98	30
Stage 1	-	-	-	-	30	-
Stage 2	-	-	-	-	68	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1583	-	-	-	901	1044
Stage 1	-	-	-	-	993	-
Stage 2	-	-	-	-	955	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1583	-	-	-	890	1044
Mov Cap-2 Maneuver	-	-	-	-	890	-
Stage 1	-	-	-	-	981	-
Stage 2	-	-	-	-	955	-

Approach	EB	WB	SB
HCM Control Delay, s	2.6	0	8.5
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1583	-	-	-	1044
HCM Lane V/C Ratio	0.011	-	-	-	0.017
HCM Control Delay (s)	7.3	0	-	-	8.5
HCM Lane LOS	A	A	-	-	A
HCM 95th %tile Q(veh)	0	-	-	-	0.1

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	53	80	47	61	104	177
Future Volume (vph)	53	80	47	61	104	177
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Storage Length (m)	50.0			50.0	0.0	0.0
Storage Lanes	1			1	1	0
Taper Length (m)	7.6				7.6	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt				0.850	0.915	
Flt Protected	0.950				0.982	
Satd. Flow (prot)	1789	1883	1883	1601	1692	0
Flt Permitted	0.950				0.982	
Satd. Flow (perm)	1789	1883	1883	1601	1692	0
Link Speed (k/h)		80	80		50	
Link Distance (m)		230.7	352.9		128.7	
Travel Time (s)		10.4	15.9		9.3	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	53	80	47	61	104	177
Shared Lane Traffic (%)						
Lane Group Flow (vph)	53	80	47	61	281	0
Sign Control		Free	Free		Stop	

Intersection Summary

Area Type: Other
Control Type: Unsignalized
Intersection Capacity Utilization 32.9% ICU Level of Service A
Analysis Period (min) 15

Lane Group	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Volume (vph)	115	81	72	74	28	31
Future Volume (vph)	115	81	72	74	28	31
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Storage Length (m)		35.0	60.0		0.0	0.0
Storage Lanes		1	1		1	0
Taper Length (m)			7.6		7.6	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt		0.850			0.929	
Flt Protected			0.950		0.977	
Satd. Flow (prot)	1883	1601	1789	1883	1709	0
Flt Permitted			0.950		0.977	
Satd. Flow (perm)	1883	1601	1789	1883	1709	0
Link Speed (k/h)	80			80	50	
Link Distance (m)	352.9			112.7	118.0	
Travel Time (s)	15.9			5.1	8.5	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	115	81	72	74	28	31
Shared Lane Traffic (%)						
Lane Group Flow (vph)	115	81	72	74	59	0
Sign Control	Free			Free	Stop	

Intersection Summary	
Area Type: Other	
Control Type: Unsignalized	
Intersection Capacity Utilization 20.8%	ICU Level of Service A
Analysis Period (min) 15	

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	90	56	59	0	0	87
Future Volume (vph)	90	56	59	0	0	87
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt					0.865	
Flt Protected		0.970				
Satd. Flow (prot)	0	1731	1784	0	1543	0
Flt Permitted		0.970				
Satd. Flow (perm)	0	1731	1784	0	1543	0
Link Speed (k/h)		80	80		80	
Link Distance (m)		112.7	168.0		103.7	
Travel Time (s)		5.1	7.6		4.7	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	90	56	59	0	0	87
Shared Lane Traffic (%)						
Lane Group Flow (vph)	0	146	59	0	87	0
Sign Control		Free	Free		Stop	

Intersection Summary	
Area Type: Other	
Control Type: Unsignalized	
Intersection Capacity Utilization 27.4%	ICU Level of Service A
Analysis Period (min) 15	

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	20	36	38	0	0	21
Future Volume (vph)	20	36	38	0	0	21
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt					0.865	
Flt Protected		0.982				
Satd. Flow (prot)	0	1850	1883	0	1629	0
Flt Permitted		0.982				
Satd. Flow (perm)	0	1850	1883	0	1629	0
Link Speed (k/h)		80	80		50	
Link Distance (m)		168.0	230.8		99.5	
Travel Time (s)		7.6	10.4		7.2	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	20	36	38	0	0	21
Shared Lane Traffic (%)						
Lane Group Flow (vph)	0	56	38	0	21	0
Sign Control		Free	Free		Stop	

Intersection Summary		
Area Type:	Other	
Control Type: Unsignalized		
Intersection Capacity Utilization 19.7%		ICU Level of Service A
Analysis Period (min) 15		

Future (2027) Total Traffic

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	27	267	34	22	74	40
Future Volume (vph)	27	267	34	22	74	40
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Storage Length (m)	50.0			50.0	0.0	0.0
Storage Lanes	1			1	1	0
Taper Length (m)	7.6				7.6	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt				0.850	0.953	
Flt Protected	0.950				0.969	
Satd. Flow (prot)	1695	1784	1784	1517	1648	0
Flt Permitted	0.950				0.969	
Satd. Flow (perm)	1695	1784	1784	1517	1648	0
Link Speed (k/h)		80	80		50	
Link Distance (m)		230.7	352.9		128.7	
Travel Time (s)		10.4	15.9		9.3	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	27	267	34	22	74	40
Shared Lane Traffic (%)						
Lane Group Flow (vph)	27	267	34	22	114	0
Sign Control		Free	Free		Stop	

Intersection Summary	
Area Type: Other	
Control Type: Unsignalized	
Intersection Capacity Utilization 28.4%	ICU Level of Service A
Analysis Period (min) 15	

Lane Group	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Volume (vph)	102	258	108	37	13	48
Future Volume (vph)	102	258	108	37	13	48
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Storage Length (m)		35.0	60.0		0.0	0.0
Storage Lanes		1	1		1	0
Taper Length (m)			7.6		7.6	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt		0.850			0.894	
Flt Protected			0.950		0.989	
Satd. Flow (prot)	1784	1517	1695	1784	1578	0
Flt Permitted			0.950		0.989	
Satd. Flow (perm)	1784	1517	1695	1784	1578	0
Link Speed (k/h)	80			80	50	
Link Distance (m)	352.9			112.7	118.0	
Travel Time (s)	15.9			5.1	8.5	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	102	258	108	37	13	48
Shared Lane Traffic (%)						
Lane Group Flow (vph)	102	258	108	37	61	0
Sign Control	Free			Free	Stop	

Intersection Summary

Area Type: Other
Control Type: Unsignalized
Intersection Capacity Utilization 29.8% ICU Level of Service A
Analysis Period (min) 15

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	96	55	52	0	0	93
Future Volume (vph)	96	55	52	0	0	93
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt					0.865	
Flt Protected		0.969				
Satd. Flow (prot)	0	1729	1784	0	1543	0
Flt Permitted		0.969				
Satd. Flow (perm)	0	1729	1784	0	1543	0
Link Speed (k/h)		80	80		80	
Link Distance (m)		112.7	168.0		103.7	
Travel Time (s)		5.1	7.6		4.7	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	96	55	52	0	0	93
Shared Lane Traffic (%)						
Lane Group Flow (vph)	0	151	52	0	93	0
Sign Control		Free	Free		Stop	

Intersection Summary	
Area Type: Other	
Control Type: Unsignalized	
Intersection Capacity Utilization 28.1%	ICU Level of Service A
Analysis Period (min) 15	

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	18	37	34	0	0	18
Future Volume (vph)	18	37	34	0	0	18
Ideal Flow (vphpl)	1800	1800	1800	1800	1800	1800
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt					0.865	
Flt Protected		0.984				
Satd. Flow (prot)	0	1756	1784	0	1543	0
Flt Permitted		0.984				
Satd. Flow (perm)	0	1756	1784	0	1543	0
Link Speed (k/h)		80	80		50	
Link Distance (m)		168.0	230.8		99.5	
Travel Time (s)		7.6	10.4		7.2	
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2%	2%	2%	2%	2%	2%
Adj. Flow (vph)	18	37	34	0	0	18
Shared Lane Traffic (%)						
Lane Group Flow (vph)	0	55	34	0	18	0
Sign Control		Free	Free		Stop	

Intersection Summary		
Area Type:	Other	
Control Type: Unsignalized		
Intersection Capacity Utilization 19.8%		ICU Level of Service A
Analysis Period (min) 15		

Intersection	
Int Delay, s/veh	6.8

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	60	89	53	63	105	197
Future Vol, veh/h	60	89	53	63	105	197
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	500	-	-	500	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	60	89	53	63	105	197

Major/Minor	Major1			Major2	Minor2	
Conflicting Flow All	116	0	-	0	262	53
Stage 1	-	-	-	-	53	-
Stage 2	-	-	-	-	209	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1473	-	-	-	727	1014
Stage 1	-	-	-	-	970	-
Stage 2	-	-	-	-	826	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1473	-	-	-	697	1014
Mov Cap-2 Maneuver	-	-	-	-	697	-
Stage 1	-	-	-	-	930	-
Stage 2	-	-	-	-	826	-

Approach	EB		WB		SB	
HCM Control Delay, s	3		0		11.3	
HCM LOS					B	

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1473	-	-	-	876
HCM Lane V/C Ratio	0.041	-	-	-	0.345
HCM Control Delay (s)	7.5	-	-	-	11.3
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0.1	-	-	-	1.5

Intersection	
Int Delay, s/veh	2.9

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑	↱	↰	↑	Y	
Traffic Vol, veh/h	118	91	72	79	32	32
Future Vol, veh/h	118	91	72	79	32	32
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	350	600	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	118	91	72	79	32	32

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	209	0	341	118
Stage 1	-	-	-	-	118	-
Stage 2	-	-	-	-	223	-
Critical Hdwy	-	-	4.12	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	-	-	2.218	-	3.518	3.318
Pot Cap-1 Maneuver	-	-	1362	-	655	934
Stage 1	-	-	-	-	907	-
Stage 2	-	-	-	-	814	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1362	-	620	934
Mov Cap-2 Maneuver	-	-	-	-	620	-
Stage 1	-	-	-	-	907	-
Stage 2	-	-	-	-	771	-

Approach	EB	WB	NB
HCM Control Delay, s	0	3.7	10.3
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	745	-	-	1362	-
HCM Lane V/C Ratio	0.086	-	-	0.053	-
HCM Control Delay (s)	10.3	-	-	7.8	-
HCM Lane LOS	B	-	-	A	-
HCM 95th %tile Q(veh)	0.3	-	-	0.2	-

Intersection	
Int Delay, s/veh	4.8

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	90	61	64	0	0	87
Future Vol, veh/h	90	61	64	0	0	87
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	90	61	64	0	0	87

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	64	0	-	0	305	64
Stage 1	-	-	-	-	64	-
Stage 2	-	-	-	-	241	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1538	-	-	-	687	1000
Stage 1	-	-	-	-	959	-
Stage 2	-	-	-	-	799	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1538	-	-	-	645	1000
Mov Cap-2 Maneuver	-	-	-	-	645	-
Stage 1	-	-	-	-	901	-
Stage 2	-	-	-	-	799	-

Approach	EB	WB	SB
HCM Control Delay, s	4.5	0	8.9
HCM LOS			A

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1538	-	-	-	1000
HCM Lane V/C Ratio	0.059	-	-	-	0.087
HCM Control Delay (s)	7.5	0	-	-	8.9
HCM Lane LOS	A	A	-	-	A
HCM 95th %tile Q(veh)	0.2	-	-	-	0.3

Intersection						
Int Delay, s/veh	2.6					

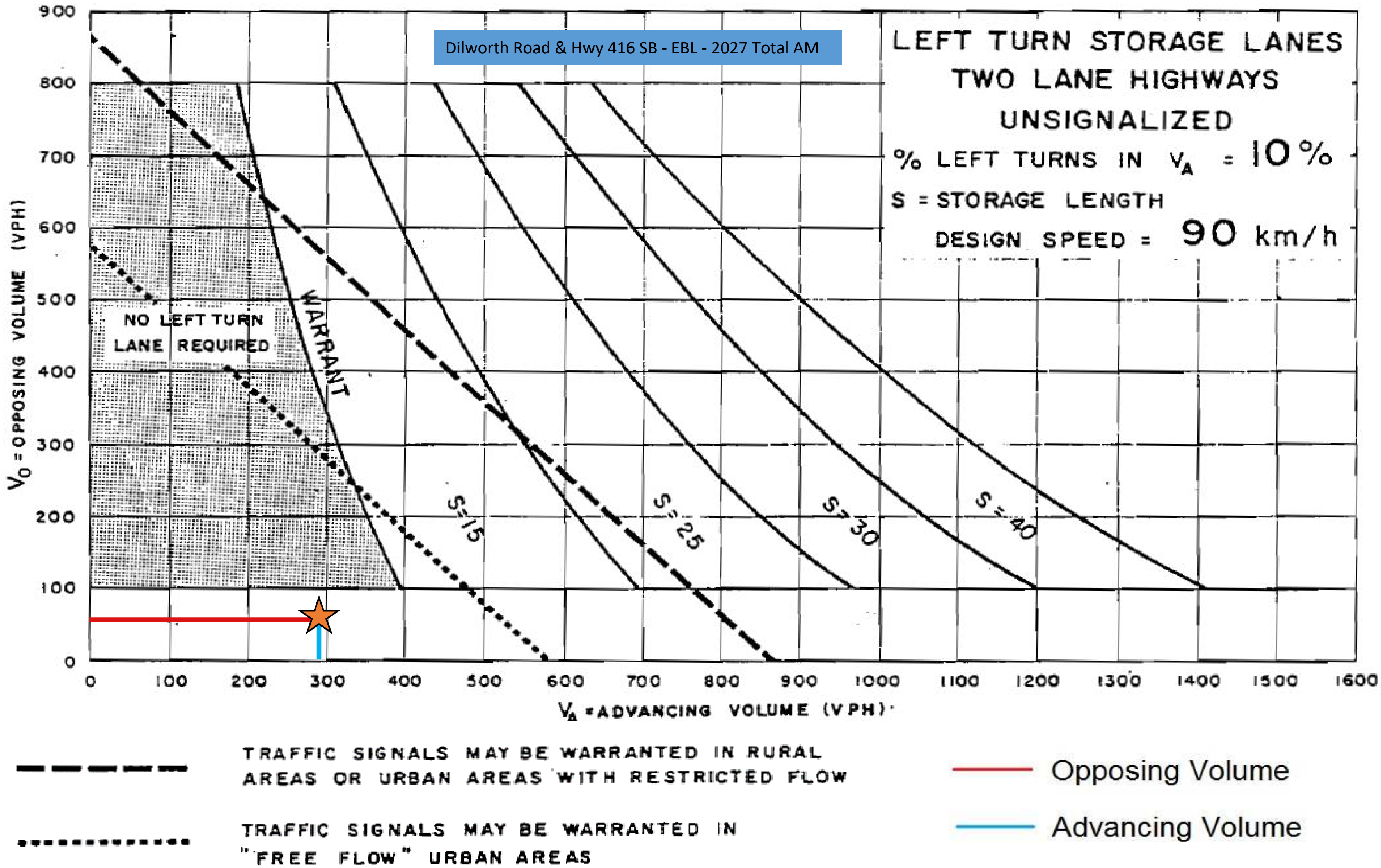
Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	20	41	43	0	0	21
Future Vol, veh/h	20	41	43	0	0	21
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	100	100	100	100	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	20	41	43	0	0	21

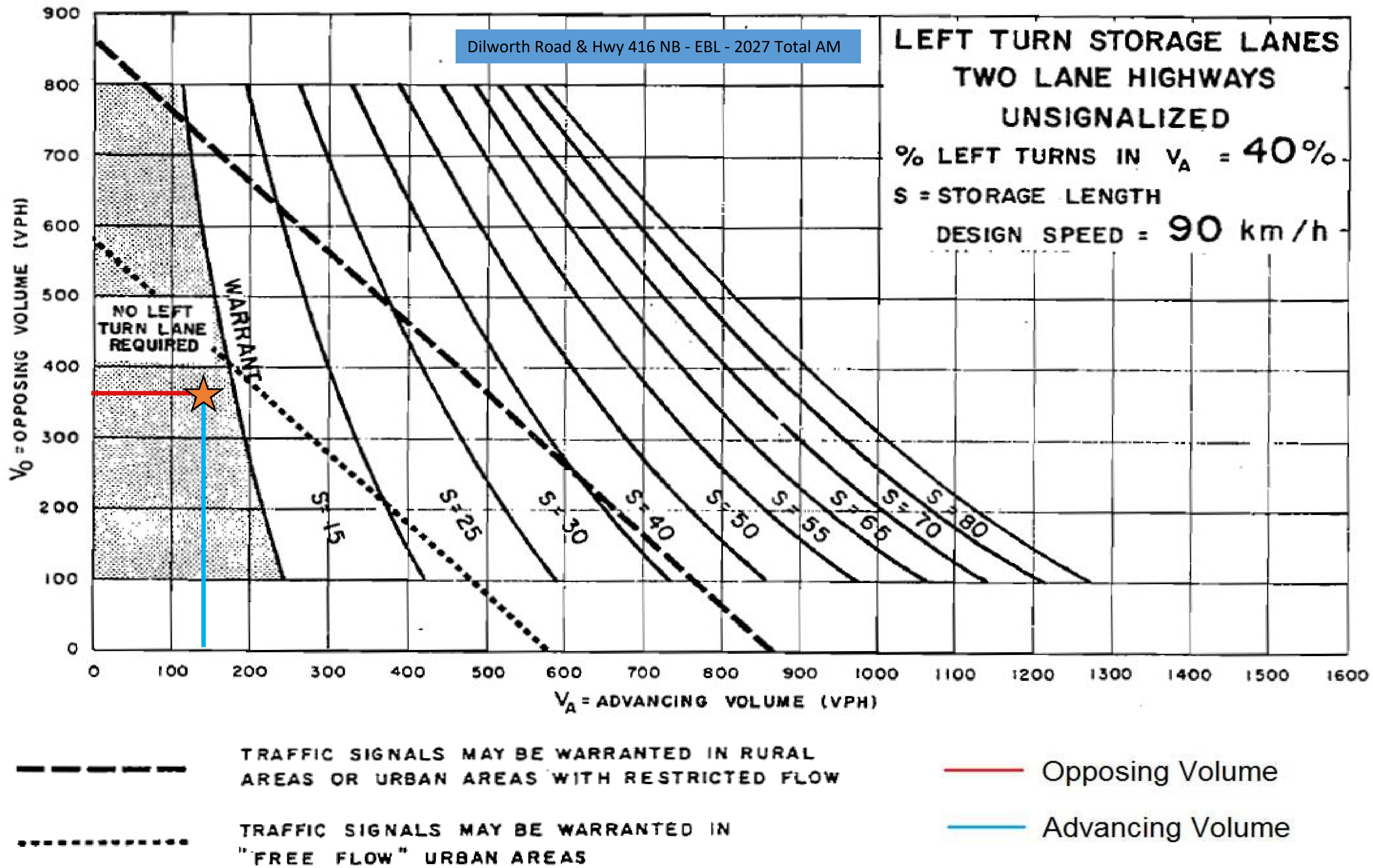
Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	43	0	-	0	124	43
Stage 1	-	-	-	-	43	-
Stage 2	-	-	-	-	81	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1566	-	-	-	871	1027
Stage 1	-	-	-	-	979	-
Stage 2	-	-	-	-	942	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1566	-	-	-	860	1027
Mov Cap-2 Maneuver	-	-	-	-	860	-
Stage 1	-	-	-	-	966	-
Stage 2	-	-	-	-	942	-

Approach	EB	WB	SB
HCM Control Delay, s	2.4	0	8.6
HCM LOS			A

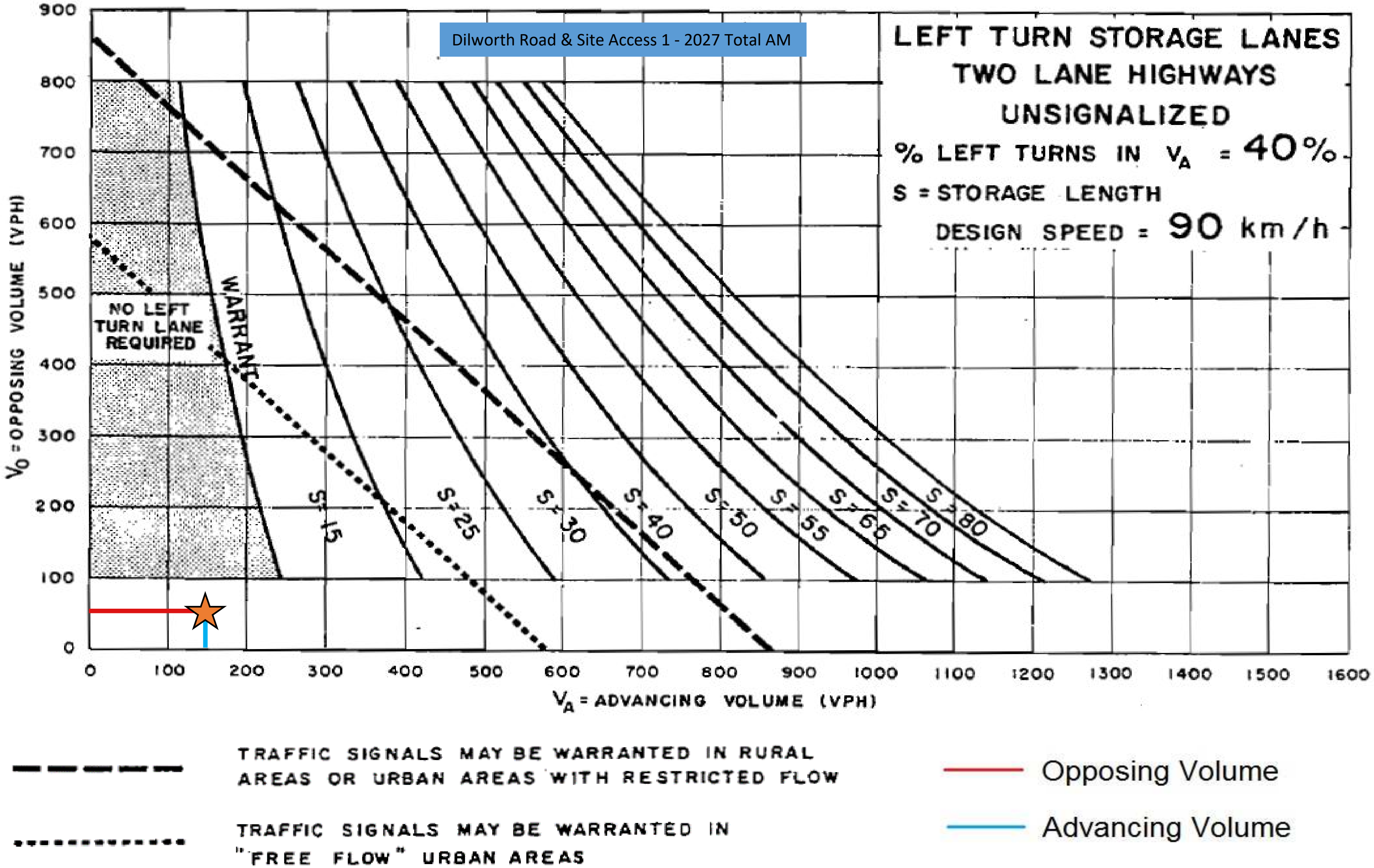
Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1566	-	-	-	1027
HCM Lane V/C Ratio	0.013	-	-	-	0.02
HCM Control Delay (s)	7.3	0	-	-	8.6
HCM Lane LOS	A	A	-	-	A
HCM 95th %tile Q(veh)	0	-	-	-	0.1

Appendix H – Auxiliary Lane Analyses

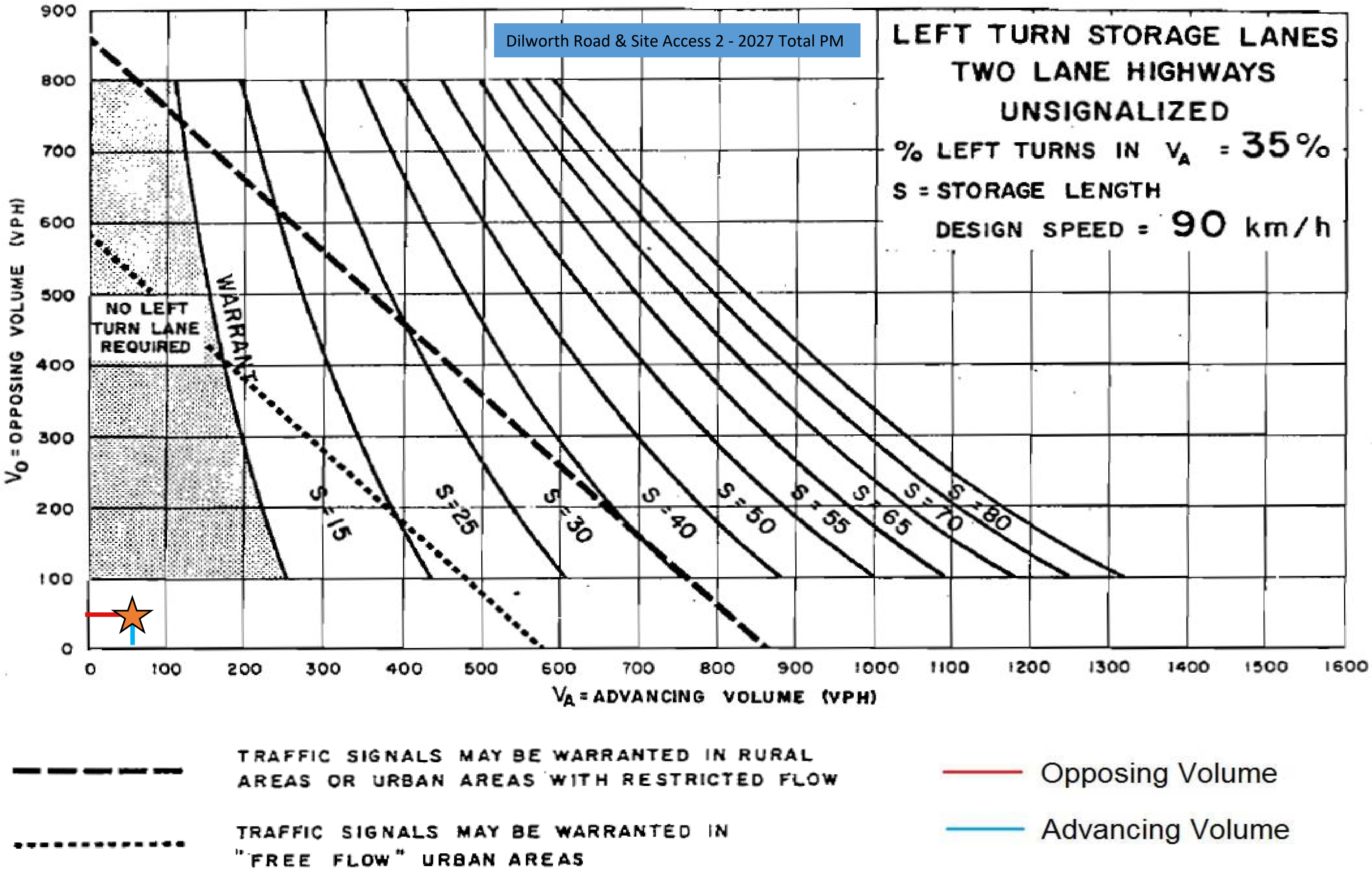

Dilworth Road & Hwy 416 SB - EBL - 2027 Total AM
LEFT TURN STORAGE LANES
TWO LANE HIGHWAYS
UNSIGNALIZED
% LEFT TURNS IN V_A = 10 %
S = STORAGE LENGTH
DESIGN SPEED = 90 km/h
NO LEFT TURN
LANE REQUIRED
WARRANT
S=15
S=25
S=30
S=40
V_O = OPPOSING VOLUME (VPH)
V_A = ADVANCING VOLUME (VPH)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
TRAFFIC SIGNALS MAY BE WARRANTED IN RURAL AREAS OR URBAN AREAS WITH RESTRICTED FLOW
TRAFFIC SIGNALS MAY BE WARRANTED IN "FREE FLOW" URBAN AREAS
Opposing Volume
Advancing Volume



Dilworth Road & Hwy 416 NB - EBL - 2027 Total AM
LEFT TURN STORAGE LANES
TWO LANE HIGHWAYS
UNSIGNALIZED
% LEFT TURNS IN V_A = 40%
S = STORAGE LENGTH
DESIGN SPEED = 90 km/h
NO LEFT TURN LANE REQUIRED
WARRANT
S=15
S=25
S=30
S=40
S=50
S=55
S=65
S=70
S=80
V_0 = OPPOSING VOLUME (VPH)
V_A = ADVANCING VOLUME (VPH)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
TRAFFIC SIGNALS MAY BE WARRANTED IN RURAL AREAS OR URBAN AREAS WITH RESTRICTED FLOW
TRAFFIC SIGNALS MAY BE WARRANTED IN "FREE FLOW" URBAN AREAS
Opposing Volume
Advancing Volume



Dilworth Road & Site Access 1 - 2027 Total AM
LEFT TURN STORAGE LANES
TWO LANE HIGHWAYS
UNSIGNALIZED
% LEFT TURNS IN V_A = 40%
S = STORAGE LENGTH
DESIGN SPEED = 90 km/h
NO LEFT TURN LANE REQUIRED
WARRANT
S=15
S=25
S=30
S=40
S=50
S=55
S=65
S=70
S=80
V_O = OPPOSING VOLUME (VPH)
V_A = ADVANCING VOLUME (VPH)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
TRAFFIC SIGNALS MAY BE WARRANTED IN RURAL AREAS OR URBAN AREAS WITH RESTRICTED FLOW
TRAFFIC SIGNALS MAY BE WARRANTED IN "FREE FLOW" URBAN AREAS
Opposing Volume
Advancing Volume


Dilworth Road & Site Access 2 - 2027 Total PM
LEFT TURN STORAGE LANES
TWO LANE HIGHWAYS
UNSIGNALIZED
% LEFT TURNS IN V_A = 35%
S = STORAGE LENGTH
DESIGN SPEED = 90 km/h
NO LEFT TURN LANE REQUIRED
WARRANT
S=15
S=25
S=30
S=40
S=50
S=55
S=65
S=70
S=80
V_O = OPPOSING VOLUME (VPH)
V_A = ADVANCING VOLUME (VPH)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
TRAFFIC SIGNALS MAY BE WARRANTED IN RURAL AREAS OR URBAN AREAS WITH RESTRICTED FLOW
TRAFFIC SIGNALS MAY BE WARRANTED IN "FREE FLOW" URBAN AREAS
Opposing Volume
Advancing Volume