TRAFFIC IMPACT STUDY

210 DEAN AVENUE DEVELOPMENT

School Development

City of Barrie, Simcoe County

Prepared for:

SIMCOE COUNTY DISTRICT SCHOOL BOARD
Midhurst, ON

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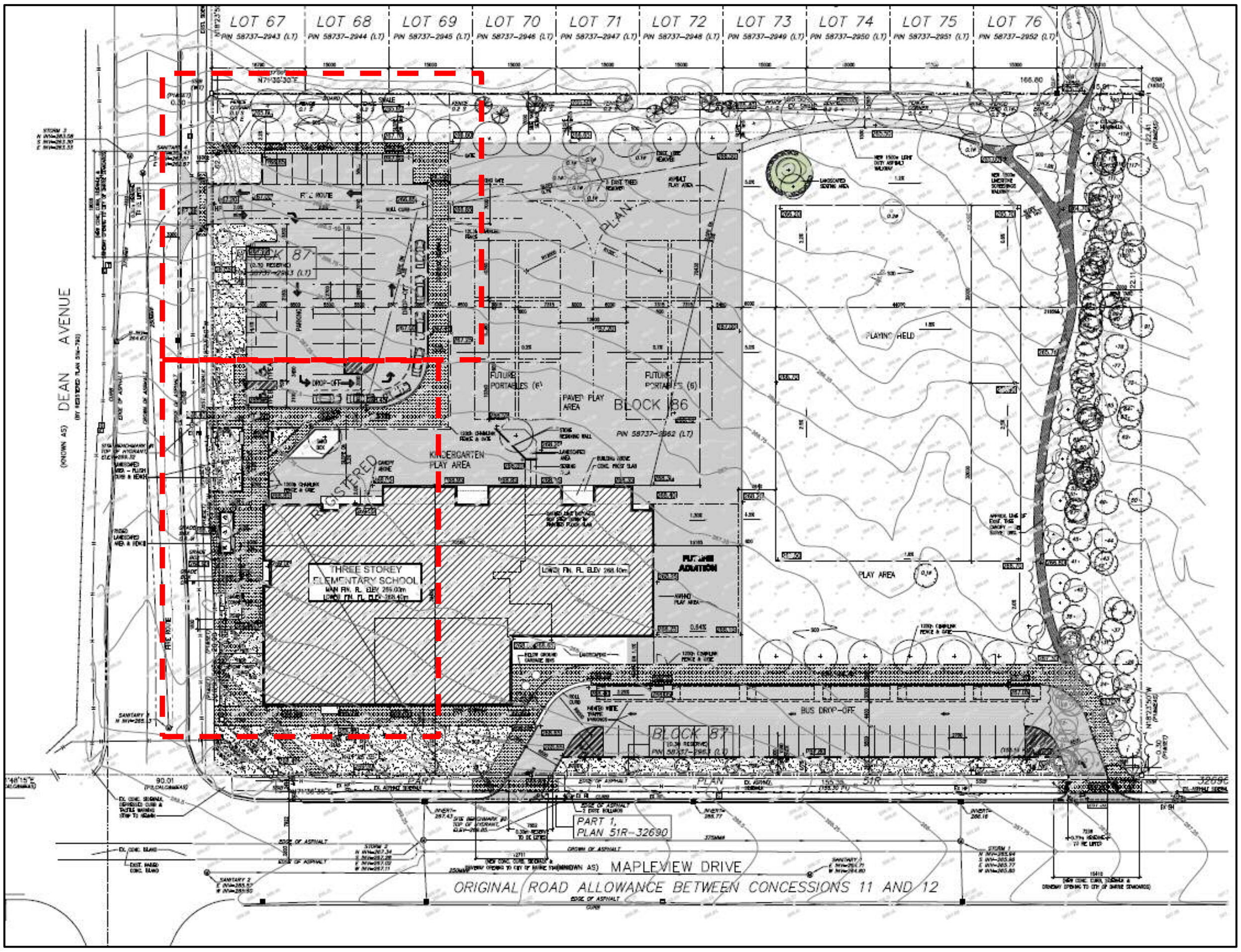
1.0 Introduction

LMM Engineering Inc. was retained by Simcoe County District School Board (SCDSB) to undertake a traffic impact study to evaluate the traffic impacts of the proposed elementary school on the site located at 210 Dean Avenue in the City of Barrie, ON. The site is located at the northeast quadrant of the intersection of Dean Avenue and Mapleview Drive East.

The proposed development consists of a three-storey school building with associated recreational space, parking lots and drop-off areas. The total gross floor area of the school is 5,857 sqm. The subject site is currently vacant. The site location map is shown in **Figure 1-1** and the proposed site plan is shown in **Figure 1-2** as well as *Appendix A*.

Figure 1-1 Site Location Map

Figure 1-2 Proposed Site Plan



2.0 Study Methodology

2.1 Study Area

In order to assess the traffic impacts of the proposed development, the following intersections were included in the Existing, Future Background, and Future Total conditions traffic operation evaluation:

- Mapleview Drive East at Dean Avenue
- Mapleview Drive East at Madelaine Drive
- Mapleview Drive East at Goodwin Drive

2.2 Agency Correspondence

Comments received from the City of Barrie in a site plan pre-consultation review have been considered in the preparation of this report. Additionally, projected traffic volumes and future lane configurations have been sourced from a Master Transportation Study and Functional Design Review submitted by LEA Consulting to the City in 2017.

A previous study was completed dated June 4th and submitted to the City for comments. Following this, a meeting was conducted between the applicants consultants and City staff wherein LMM was advised that a pedestrian signal was the City's preferred option for north-south pedestrian movements across Mapleview Drive east of Dean Avenue, with the retention of the raised concrete median on Mapleview Drive East. This report incorporates those comments in the analysis.

2.3 Study Horizons

The report analyzes existing conditions using a combination of interim traffic volumes from the Master Transportation Study as well as existing lane configurations in 2024. This data was used as it accounts for partial build-out of the Hewitt Lands immediately south of the study area. The report identifies the weekday PM period as the critical analysis period.

The report analyzes future total conditions using a combination of ultimate traffic volumes from the Master Transportation Study (2031) as well as recommended lane configurations. This data was used as it accounts for a full build-out of the Hewitt Lands immediately south of the study area, as well as other background traffic around Southeast Barrie including the build-out of proposed schools and other employment centres. The subject site is anticipated to be operational by this horizon year.

2.4 Intersection Capacity Evaluation Methodology

In this study, the methodology used for evaluating traffic operations at each of the subject intersections was based on the criteria set forth in the Transportation Research Board's Highway Capacity Manual, 2010 edition (HCM 7th Edition). Synchro 12 software, which utilizes the HCM 7th edition methodology, was used for the analysis. The following is a description of the methodology employed for the analysis of unsignalized and signalized intersections.

2.4.1 Unsignalized Intersections

For unsignalized intersections at which the side street or minor street is controlled by a stop sign, the criteria for evaluating traffic operations are the level of service (LOS) for the turning movements at the intersection and the level of service for the overall intersection. Level of service is based on the average controlled delay incurred at the intersection. Controlled delay for unsignalized intersections includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Several factors affect the controlled delay for unsignalized intersections, such as the availability and distribution of gaps in the conflicting traffic stream, critical gaps, and follow-up time for a vehicle in the queue.

Level of service is assigned a letter designation from A through F. Level of service A indicates excellent operations with little delay to motorists, while level of service F exists when there are insufficient gaps of acceptable size to allow vehicles on the side street to cross freely, resulting long total delays and long queues. The level of service criteria for two-way stop-controlled and all-way stop-controlled (unsignalized) intersections is given in **Table 2-1.**

Table 2-1 Level of Service Criteria for Unsignalized Intersections.

Level of Service	Average Control Delay (sec/veh)
A	≤ 10
B	> 10 and ≤ 15
C	> 15 and ≤ 25
D	> 25 and ≤ 35
E	> 35 and ≤ 50
F	> 50

At unsignalized intersections, movements with an average controlled delay of greater than 50 seconds are defined as critical movements. The Ministry of Transportation of Ontario (MTO) TIS guidelines indicate that for a ramp movement, a volume to capacity (v/c) ratio greater than 0.75 would be considered critical. For any other movement, a v/c ratio greater than 0.85 would be considered critical. For unsignalized intersections, the overall intersection operations are stated for the approach or movement with the worst level of service and highest delay.

2.4.2 Signalized Intersections

For signalized intersections, it is necessary to evaluate both capacity and level of service in order to evaluate the overall operation of the intersection. The capacity analysis of an intersection is performed by comparing the volume of traffic using the various lane groups at the intersection to the capacity of those lane groups. This results in a volume/capacity (v/c) ratio for each lane group. A v/c ratio greater than 1.0 indicates that the volume of traffic has exceeded the capacity available, resulting in a temporary excess of demand. Although the capacity of the entire intersection is not defined, a composite v/c ratio for the sum of the critical lane groups within the intersection is computed. This composite v/c ratio is an indication of the overall intersection efficiency.

Level of service for a signalized intersection is defined in terms of average controlled delay per vehicle, which is composed of initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. The levels of service criteria for signalized intersections, based on average controlled delay, are shown in **Table 2-2**. Level of service A indicates operations with very low controlled delay, while level of service F describes operations with extremely high average controlled delay. Level of service E is typically considered to be the limit of acceptable delay, and level of service F is considered unacceptable by most drivers.

Table 2-2 Level of Service Criteria for Signalized Intersections.

Level of Service	Average Control Delay (sec/veh)
A	≤ 10
B	> 10 and ≤20
C	> 20 and ≤35
D	> 35 and ≤55
E	> 55 and ≤80
F	> 80

At congested arterial signalized intersections, movements with a level of service (LOS) of 'F', with average controlled delay greater than 80 seconds are considered critical. Movements operating at LOS E are also flagged since these may become critical in the future. The Ministry of Transportation of Ontario (MTO) TIS guidelines indicate that for a ramp movement, a volume to capacity (v/c) ratio greater than 0.75 would be considered critical. For any other movement, a v/c ratio greater than 0.85 would be considered critical.

3.0 EXISTING CONDITION

The subject site is currently vacant. The site location map is shown in Figure 1-1 and the proposed site plan is shown in Figure 1-2 and *Appendix A*.

An inventory of the surrounding roads and highway facilities in the vicinity of the site was compiled and is summarized as follows:

Mapleview Drive East is a 4-lane east-west arterial road with a posted speed limit of 50 km/h within the vicinity of the site. There is a sidewalk on the north side of the road, within the vicinity of the site. There are no dedicated bicycle facilities along the road. It is noted that following the existing conditions review for this analysis, the speed limit in the eastbound direction between Madelaine Drive and Dean Avenue was updated to 60 km/hr.

Dean Avenue is a 2-lane north-south minor collector road with a speed limit of 50 km/hr. There are sidewalks on the east and west sides of the road. There are painted bicycle lanes in both directions of travel. The intersection of Dean Avenue / Greer Avenue at Mapleview Drive E is an unsignalized intersection with two-way stop-control and a raised concrete median along the east-west direction (Mapleview Drive E). Due to the turn restrictions imposed by the raised median, left turns are prohibited for all approaches. The north and south approaches feature stop controls while the east-west movement is free of controls.

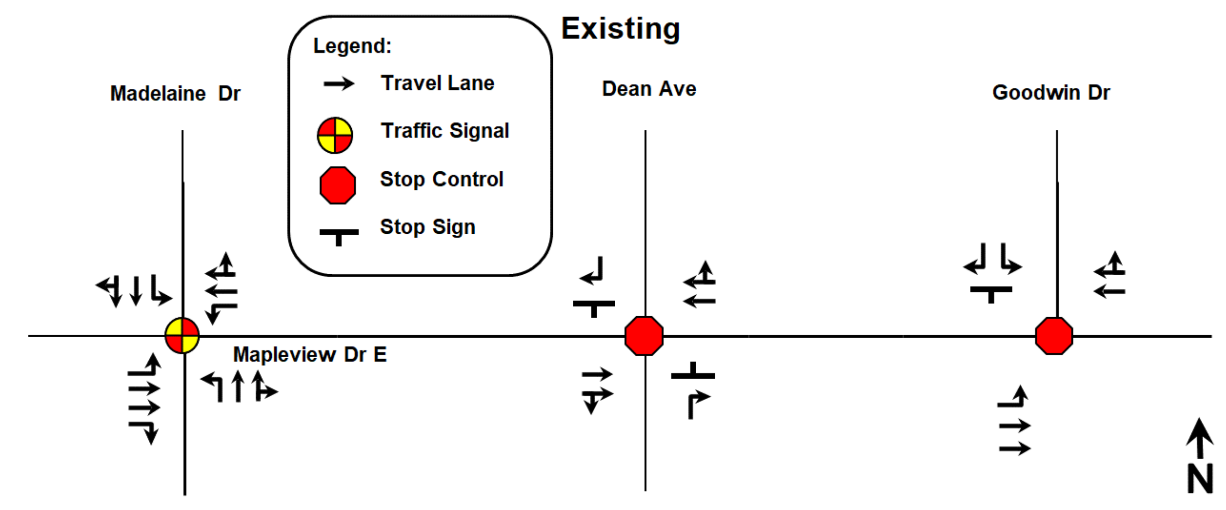
Madelaine Drive is a 4-lane north-south major collector road with a speed limit of 50 km/hr. There are sidewalks on the east and west sides of the road. There are no dedicated bicycle facilities along the road. Parking is permitted along the curb lanes in both directions of travel. The intersection of Madelaine Drive at Mapleview Drive E is a 4-way, signalized intersection. There are painted crosswalks on all approaches with pedestrian crossing signal heads. Traffic signal heads for all approaches feature a dedicated left turn arrow.

Goodwin Drive is a 2-lane north-south local road with a speed limit of 50 km/hr. There is a sidewalk on the east side of the road. There are no dedicated bicycle facilities along this road. The intersection of Goodwin Drive & Mapleview Drive E is a 3-leg, unsignalized intersection with stop-controls on the north approach. The east-west movement is free of controls. There is a marked crosswalk on the north approach. There are no pedestrian crossing facilities at the east and west approaches.

Access to the development is proposed via Dean Avenue and Mapleview Drive E. There is one full-movement access proposed on Dean Avenue, approximately 110m north of Mapleview Drive E.
There are two proposed accesses along Mapleview Drive E. The west access functions as a right-out access (egress only) and is located approximately 40m east of the curb return of Dean Avenue. The east access functions as a right-in access (ingress only) and is located approximately 120m east of the curb return of Dean Avenue.

The existing lane configuration of the study area intersections is shown in **Figure 3-1.**

Figure 3-1 Lane Configuration (N.T.S.)



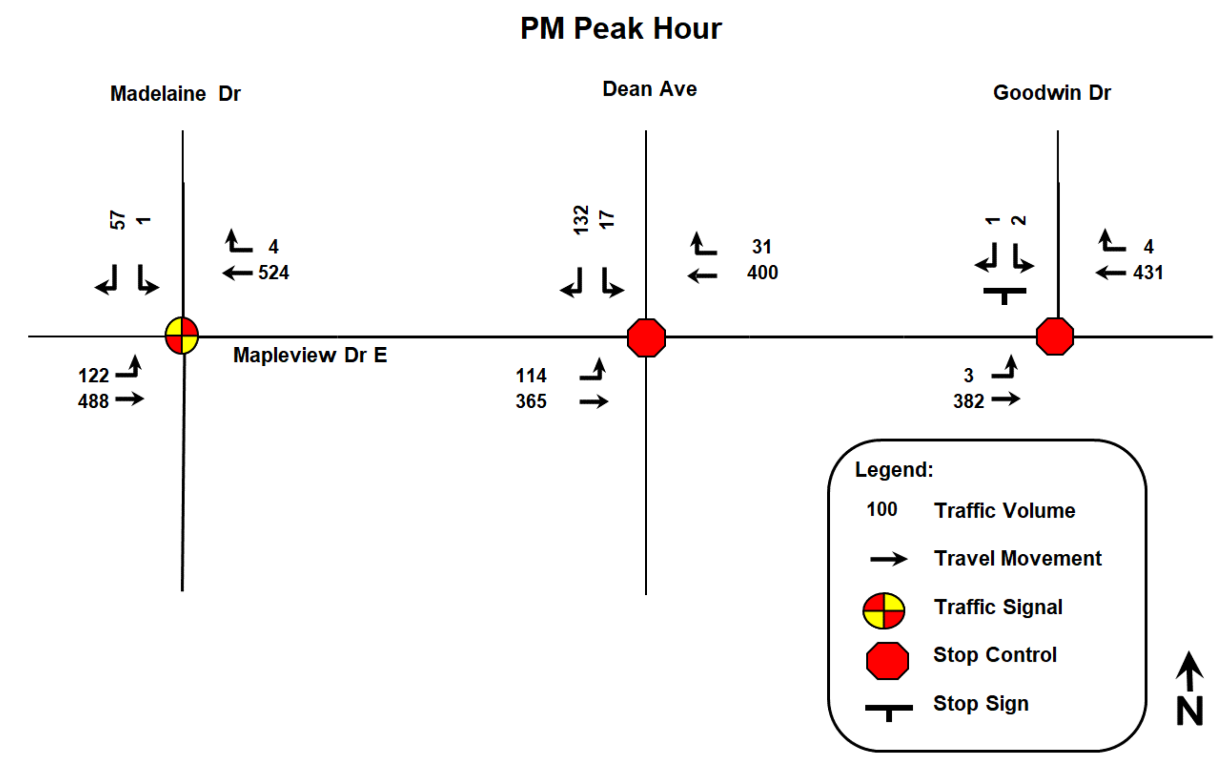
3.1 Existing Traffic Volumes

To estimate existing traffic volumes at the study intersections, LMM referenced interim horizon conditions (2020) from the Master Transportation Study report. It is noted that these traffic conditions are based on 2017 base traffic volumes, factored up to 2020 using background growth rates and a further addition of 20% build-out traffic volumes of the Hewitt Subdivision Lands located south of the study area. The Master Transportation Study report does not account for the proposed school development traffic volumes.

The existing road network configuration is different from the assumptions in the Master Transportation Study, most notably the inclusion of the raised concrete median along Mapleview Drive at Dean Avenue. Traffic volumes are therefore redistributed to account for the changed road geometry.

The interim horizon condition traffic volumes were used as the existing condition volumes in this report and are shown in **Figure 3-2**.

Figure 3-2 Existing Peak Hour Traffic Volumes



3.1.1 Existing Intersection Capacity Analysis

The projected interim volumes from the Master Transportation Study were overlaid onto existing lane configurations to estimate existing conditions in the study area. The analysis results are the are summarized in Table 3-1 below. Detailed intersection capacity analysis output for this condition are included in *Appendix B*.

Table 3-1 Existing Condition Intersection Capacity Analysis Summary

Intersection	Overall / Critical Movement	Traffic Operations (LOS, Delay (sec's), V/C Ratio)
		P.M. Peak Hour
Madelaine Drive / Mapleview Drive E	Intersection	A, 1.5s, 0.19
	Critical Movement	-
Dean Ave / Mapleview Drive E	Intersection	A, 2.9s, 0.21
	Critical Movement	-
Goodwin Drive / Mapleview Drive E	Intersection	A, 0.1s, 0.18
	Critical Movement	-

* For unsignalized intersections, the overall intersection operations are stated for the approach or movement with the worst level of service and highest delay.

The results of the existing condition intersection capacity analysis indicate that all the existing study intersections operate at acceptable levels of service "LOS".

4.0 Future Total Condition

4.1 Proposed Development

The proposed development consists of a three-storey school building with associated recreational space, parking lots and drop-off areas. The total gross floor area of the school is 5,857 sqm. The subject site is currently vacant. Given school classroom capacities as well as building design constraints, the number of students expected to use the facility is 662 students as well as 12 portables providing additional capacity for up to 276 students. Thus, the total anticipated number of students at full build-out is 938. The site location map is shown in Figure 1-1 and the proposed site plan is shown in Figure 1-2.

4.2 Total Traffic Conditions

Estimates for total traffic conditions were taken from the Master Transportation Study using the Ultimate Horizon of 2031. It is noted that the traffic model used in that report considered site generated traffic from the full build out of the Hewitt Secondary Plan area as well as projected background traffic within the vicinity such as schools and employment centres. As noted in the modelling rationale in that report, school trips are not expected to significantly contribute to impacts along Mapleview Drive due to a large percentage of school traffic being part of pass-by trips for other land uses. An excerpt of traffic volumes is provided in *Appendix C*.

The Master Transportation Study indicates that the weekday PM peak period is expected to be the critical analysis period for the surrounding road network due to the amount of background vehicular traffic for a variety of land uses in the study area. Thus, only the weekday PM peak period is analyzed for total traffic conditions.

4.3 Walking Trips

The catchment area for the proposed school is shown in *Appendix D*. Given the geographic context of the school as well as a significant number of students originating from residential areas south of Mapleview Drive E, it is anticipated that many students will be crossing Mapleview Drive E at Dean Avenue. In order to estimate the new walking trips that would be generated by the proposed school, trip generation rates in the Institute of Transportation Engineers (ITE) Trip Generation Manual for the following uses:

- The ITE Land Use Code 520 – Elementary School

The resultant total walking trip generation for the primary and pass-by related traffic associated with the proposed development uses is summarized in **Table 4-1**.

Table 4-1 Pedestrian Trip Generation Summary

Land Use	Size of Development	Walking Trips (pedestrians per hour)					
		AM Peak Hour			PM Peak Hour		
		IN	OUT	TOTAL	IN	OUT	TOTAL
ITE Land Use Code 520 – Elementary School	938 Students	-	-	169	-	-	122
Total		-	-	169	-	-	122

The trip generation estimates shown Table 5-1 based on the ITE data indicate that the development would generate around 122 to 169 pedestrians during the weekday AM and PM peak periods. A significant number of these would cross Mapleview Drive E at Dean Avenue in order to access the school.

4.4 Option 1 – Proposed Signalization of Intersection

As noted in the Existing Conditions section, the intersection of Mapleview Drive E at Dean Avenue is currently configured as a two-way stop-controlled intersection with a raised concreted median on Mapleview Drive E. Thus, left-turns are prohibited for all approaches. Following a review of the intersection configuration as well as future demands, LMM Engineering proposes the conversion of this intersection into a full movement, signalized intersection. A rationale for this proposal is detailed in the subsections that follow.

4.4.1 Rationale For Signalization

4.4.1.1 <u>Alignment with Master Transportation Study / Hewitt Secondary Plan</u>

The Master Transportation Study / Secondary Plan for the Hewitt Lands located south of the study area project a large increase in development and subsequently vehicular traffic from the south. Dean Avenue is a minor collector in the north-south direction and northbound traffic from these lands would capacity constraints at the eastbound left turn on Mapleview Drive E at Yonge Street. In alignment with the Master Transportation Study, the removal of the raised median is recommended in order to facilitate all movements at the key intersection.

4.4.1.2 <u>Safety Concerns for Vehicular Traffic</u>

With the existing configuration of the intersection of Mapleview Drive at Dean Avenue, it is noted that eastbound left-turns are prohibited, restricting access to the proposed school at Dean Avenue. As a result, auto drivers are more likely to proceed eastbound along Mapleview Drive and make a left turn into the proposed right-in entrance along Mapleview Drive East. Given that there are no traffic controls for east/west traffic along this segment of the road, the likelihood of left-hook collisions and head-on collisions is elevated. As this segment of Mapleview Drive E is posted as 50 km/hr and is a multilane, undivided road with no notable horizontal curves or traffic calming measures, the severity of any resultant collisions will be high.

Removal of the raised median and the subsequent conversion of this intersection into a full-movement, signalized intersection would permit eastbound left turns, easing access to the proposed school from Dean Avenue. Left-in movements further east along Mapleview Drive are less likely to occur, however moving the raised concrete median further east along this segment may further ensure that these manoeuvres do not occur.

4.4.1.3 Safety Concerns for Pedestrians

With the existing configuration of the intersection of Mapleview Drive at Dean Avenue, it is noted that pedestrians intending to cross Mapleview Drive in order to access the school must detour approximately 530m west to the intersection with Madelaine Drive in order to use a protected pedestrian crossing facility, before returning 530m east for a total detour distance of 1.06km. As stated in Section 4.3, the proposed school is anticipated to generate pedestrian volumes of approximately 122-169 during the peak periods. A significant number of these are vulnerable, being elementary school children.

The walking detour distance of 1.06km is likely to contribute to pedestrians choosing to cross Mapleview Drive at Dean avenue in the absence of controls, in a piecemeal fashion (looking out for gaps one direction at a time) with the 1.5m wide raised concrete median used as a pedestrian refuge. Providing a controlled crossing facility at this location is essential for pedestrian safety, especially given the context of vulnerable road users.

Although provision for a pedestrian crossing may be provided by way of a PXO (marked pedestrian crossing), the recommendation for signalization is preferred due to historic driver compliance rates, road user vulnerability and alignment with the goal of enhanced safety and flow for all road users.

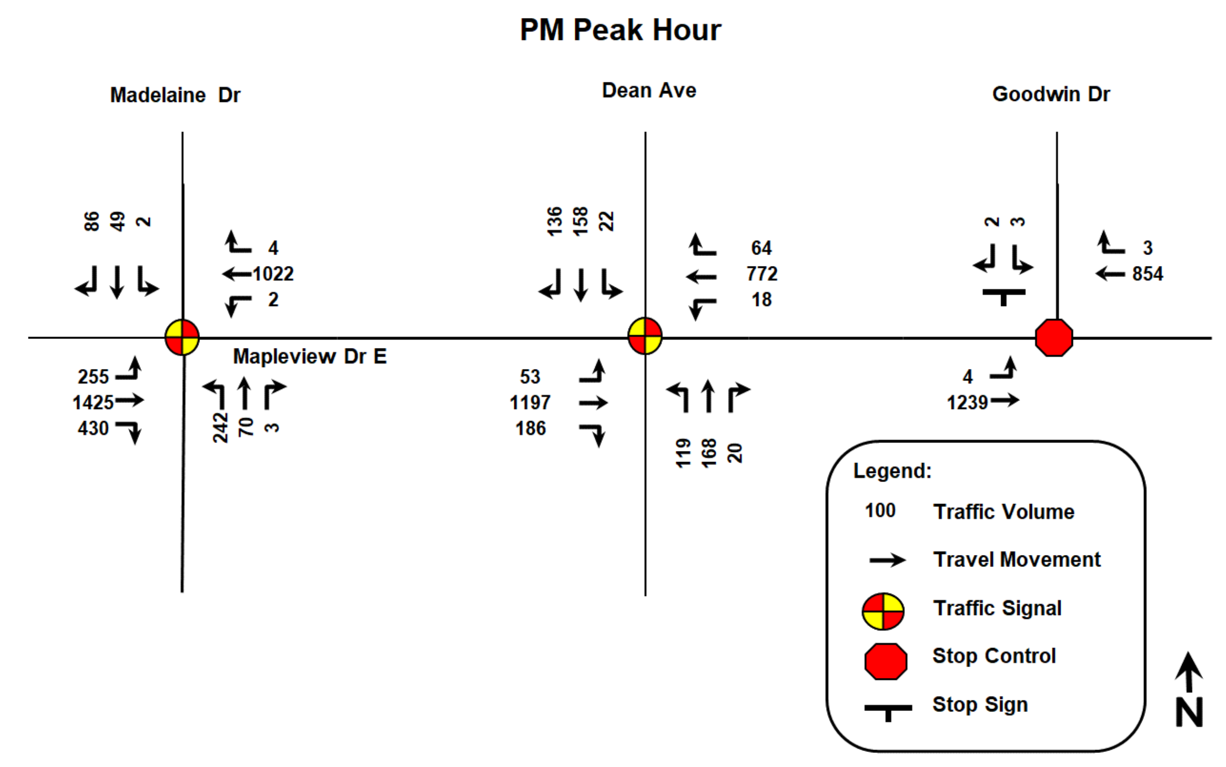
4.4.1.4 Minimum Spacing Compliance

The proposed signalization at Dean Avenue is approximately 530m east of the nearest upstream signalized intersection of Mapleview Drive at Madelaine Drive. As per the Ontario Traffic Manual (OTM) Book 12, this is greater than the minimum recommended spacing of 215m for roads posted at 60 km/hr or less. Additionally, it benefits from being within the range of 415 – 625m that Book 12 states as providing for good progression efficiency for roads posted at 50 km/hr.

4.4.2 Site Trip Distribution / Assignment

In order to analyze the future total conditions, the estimated new peak hour vehicular trips summarized in *Appendix C* were assigned to the study intersections, using the anticipated 2031 road network configuration (with the intersection at Dean Avenue signalized). In order to be consistent, the directional route distribution of site generated traffic was based on the distribution at the study intersections. The total assigned peak hour site generated traffic volumes are shown in **Figure 4-1**.

Figure 4-1 Total Peak Hour Proposed Site Development Related Trips



4.4.3 Future Total Intersection Capacity Analysis

The future total peak hour traffic volumes shown in Figure 5-1 were used to analyze the study intersections according to the methodology outlined in Section 2.4 *Intersection Capacity Evaluation* for signalized and unsignalized intersections.

The future total intersection capacity analysis results for the study intersections are summarized in **Table 4-2** below. Detailed future total intersection capacity analysis output is included in *Appendix E*.

Table 4-2 Future Condition Intersection Capacity Analysis Summary

Intersection	Overall / Critical Movement	Traffic Operations (LOS, Delay (sec's), V/C Ratio) P.M. Peak Hour
Madelaine Drive / Mapleview Drive E	Intersection	B, 18.3s, 0.99
	Critical Movement	EBL, NBL
Dean Ave / Mapleview Drive E	Intersection	B, 16.9s, 0.88
	Critical Movement	NBL
Goodwin Drive / Mapleview Drive E	Intersection	A, 0.1s, 0.40
	Critical Movement	-

* For unsignalized intersections, the overall intersection operations are stated for the approach or movement with the worst level of service and highest delay.

The results of the future total condition intersection capacity analysis indicate that the study intersection will continue to operate acceptably, with overall LOS at B or better. At the intersection of Madelaine Drive at Mapleview Drive, the eastbound left and northbound left movements are critical, but are not expected to exceed capacity. At the intersection of Dean Avenue at Mapleview Drive, the northbound left movement is critical with a v/c of 0.88 but is not expected to exceed capacity. As a result of the City's direction to retain the raised concrete median, this solution is not viable.

4.5 Option 2 – Proposed Pedestrian Signal

Following a review of the initial traffic study dated June 4th, 2024, City staff provided feedback that the City's preferred option was to retain the raised concrete median along Mapleview Drive East and implement a midblock pedestrian crossing at Mapleview Drive East to facilitate north-south pedestrian movements.

4.5.1 Rationale

LMM requested traffic modeling data to evaluate traffic operations with the raised concrete median but was informed that no traffic modeling data is available and that the decision to maintain the concrete median was based on political direction.

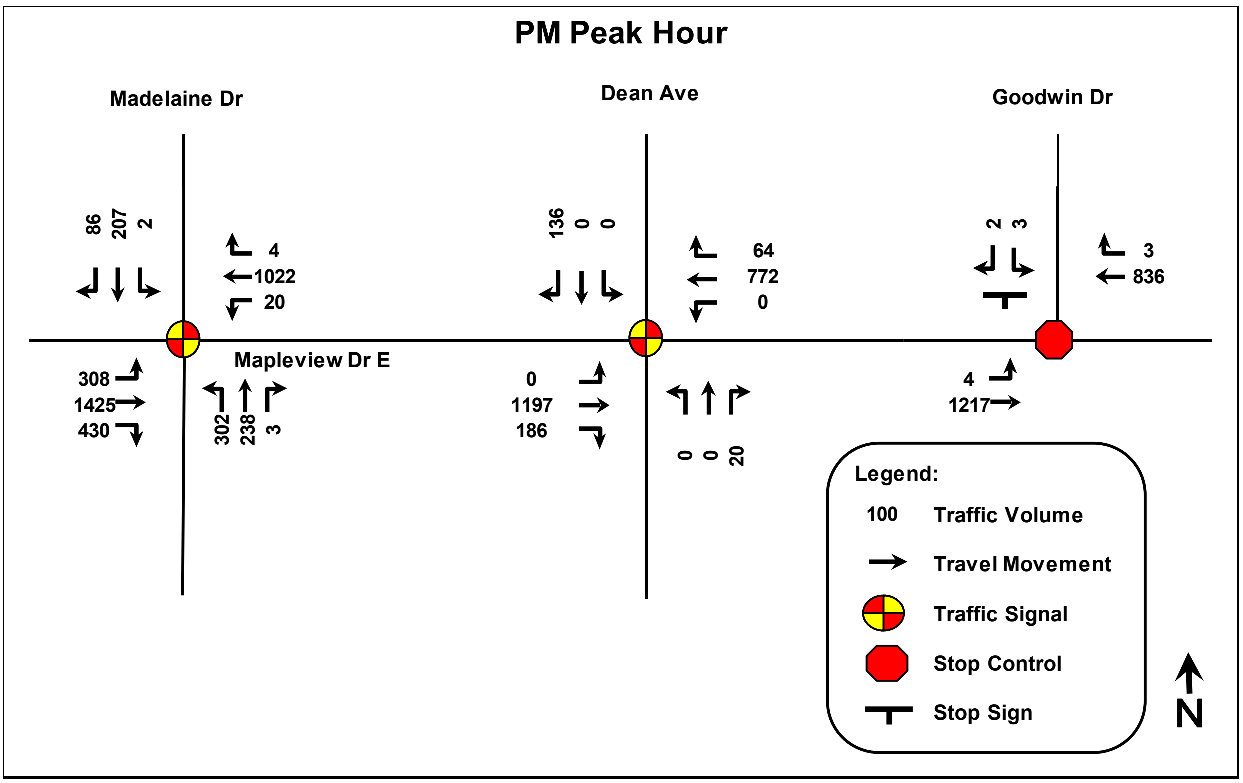
With limited traffic data available, LMM conducted a review of the impacts of maintaining the intersection in its current configuration with the addition of a midblock pedestrian crossing, detailed as follows.

4.5.2 Site Trip Distribution / Assignment

A design retaining the existing raised concrete median along Mapleview Drive East would restrict the eastbound and westbound traffic to through and right turn movements only. Northbound and southbound traffic would be restricted to a right turn only.

As a result, it is anticipated that vehicular traffic originating from west and south of the site would access the site via Country Lane. Given that there is no data available for northbound and southbound cut-through traffic along Dean Avenue, estimates for traffic following the alternative route were derived from future total conditions estimates from the Hewitt Lands study. The future total traffic volumes given the alternative route is shown in Figure 4-2.

Figure 4-2 Future Total Traffic Volume – Option 2



4.5.3 Future Total Intersection Capacity Analysis

The future total intersection capacity analysis results for the study intersections are summarized in Table 4-3 below. Detailed future total intersection capacity analysis output is included in *Appendix F.*

Table 4-3 Future Alternative Condition Intersection Capacity Analysis Summary

Intersection	Overall / Critical Movement	Traffic Operations (LOS, Delay (sec's), V/C Ratio) P.M. Peak Hour
Madelaine Drive / Mapleview Drive E	Intersection	C, 31.6s, 0.99
	Critical Movement	NBL
Dean Ave / Mapleview Drive E	Intersection	A, 0.9s, 0.51
	Critical Movement	-

With the alternative routes for traffic shown, it is noted that the performance of the intersection of Dean Avenue at Mapleview Drive East improves significantly to LOS A. The performance of the intersection of Madelaine Drive at Mapleview Drive East deteriorates due to additional traffic making eastbound left turn and northbound left turn movements. It is recommended that the eastbound left turn and northbound left turn movements have protected phases during the weekday PM peak period.

Additionally, it is recommended that the following geometric changes be made at the study intersections to reflect lane recommendations in the Hewitt Study, so as to accommodate alternative route traffic patterns:

- Dual northbound left-turn lanes at the intersection of Madelaine Drive and Mapleview Drive East.
- A dedicated left-turn lane and a shared through-right turn lane at the intersection of Goodwin Drive and Mapleview Drive East.

Given the configuration detailed above, as well as signal timing changes, it is expected that the surrounding road network can accommodate future total traffic volumes with the raised concrete median at Mapleview Drive and Dean Avenue maintained.

4.5.4 Active Mode Design Considerations

4.5.4.1 Pedestrian Considerations

As noted in Section 4.4.1.3, under existing conditions, there is no provision for pedestrians crossing Mapleview Drive East at Dean Avenue. In order to enhance pedestrian safety and connectivity while retaining the raised concrete median, a signalized pedestrian crossing is recommended. As per the Ontario Traffic Manual (OTM) Book 12 (Traffic Signals) an intersection or midblock pedestrian signal (IPS or MPS) is required in lieu of a pedestrian crossover due to the road having a 4-lane cross-section, design speeds in excess of 60km/hr and the volume of assisted pedestrians anticipated given the proximity to a school.

The recommended location for the pedestrian signal is on Mapleview Drive between Dean Avenue and the east property line of the proposed school, in order to minimize the detour distance for all pedestrian users at the intersection of Mapleview Drive and Dean Avenue. There are geometric constrains due to the shared left turn lane for properties located on the south side of Mapleview Avenue, the intersection with Dean Avenue and the raised concrete median. Therefore, an intersection pedestrian signal (IPS) is recommended for this location, placed on the east approach of the intersection. Curb letdowns are to be made to the raised concrete median for accessibility. The location of the IPS allows for the raised concrete median to be retained – thereby preventing cut-through vehicle traffic – as well as protect pedestrians crossing through the provision of a dedicated phase and avoids conflicts from left turning vehicles accessing the properties on the south side of the road.

The signal head is to be clearly visible up to 165m from each approach, assuming an 85th percentile speed of 80km/hr on Mapleview Drive. The pavement markings for an IPS are similar to those for normal signalized intersections with the vehicle stop lines set back a minimum of 12 m from the primary signal head (15 m recommended practice). The raised concrete median is to be retained on either side of the crossing to provide further traffic calming features. The components for an IPS are detailed in Table 4-4. A schematic of an IPS is shown in Figure 4-3.

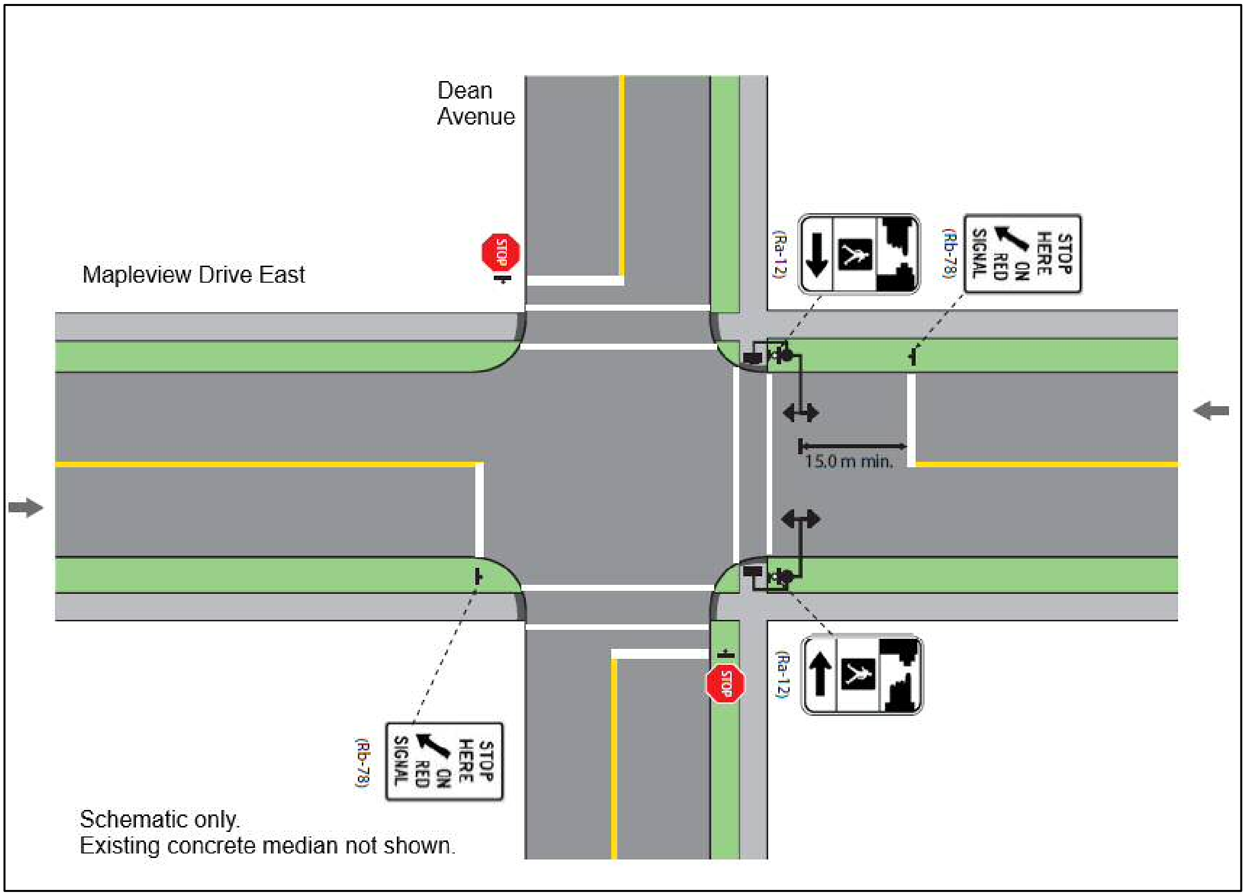
Table 4-4 Components for Intersection Pedestrian Signal

Required Components	Desirable Components	Optional Components

• Traffic Signal Heads as required
• Approach Markings (Stop Line, No-Passing zone, and Turn Lanes markings, as required)
• Crosswalk Markings
• Advanced Stop Bar at Crosswalk with mandatory Stop Here on Red Signal Sign (Rb-78)
• Stop Here On Red sign (Rb-78) on the near side of an IPS with vehicle and pedestrian heads installed on the far side
• Pedestrian Control Indications with AODA compliant Pedestrian Signal Pushbuttons and Pedestrian Pushbutton Symbol Sign (Ra-12)
• Stop sign (Ra-1) on the cross street for IPS
	• Raised refuge island (for road cross-sections with more than two lanes and two-directional traffic) with mandatory:
	  • Pavement markings on approaches to obstructions
	  • Keep Right Sign (Rb-25, Rb-125)
	  • Object Marker Sign (Wa-33L)
	• Stopping prohibition for a minimum of 30 m on each approach to the crossing, and 15 m following the crossing
	• Parking and other sight obstructions prohibition within at least 30 m of crossings
		• School Crossing Guard
		• Pedestrian Count Down Signals
		• Pedestrian Countdown Signal Information Sign
		• Auxiliary Signal Heads
		• Type 12 Signal Head (300 mm red / amber / green lens)
		• Ladder Crosswalk Markings
		• Textured Crosswalk
		• Raised Crosswalk
		• Cross on Walk Signal Only Sign (RA-7)
		• Cross Other Side Sign (Ra-9)
		• Do Not Cross Here Sign (Ra-9a)
		• No Right Turn on Red sign (Rb-79)
		• Pedestrian Must Push Button to Receive Walk Signal (Ra-13)
		• Safety elements including Barricades, Pedestrian Fencing, Gates, Walls, Bollards, and Barriers

Source: OTM Book 15 – Pedestrian Crossing Treatments

Figure 4-3 IPS Schematic



Source: OTM Book 15 (Pedestrian Crossing Treatments)

4.5.4.2 Cyclist Considerations

Under existing conditions, with the raised concrete median, there is no permeability for bicycles travelling north and south across Mapleview Drive at Dean Avenue. Maintaining this raised concrete median, there are two options considered by LMM for the permeability of bicycles across Mapleview Drive.

Uncontrolled Crossing

Without dedications available to accommodate a multi-use path north and south of Mapleview Drive, an uncontrolled crossing may be considered at the intersection of Mapleview Drive and Dean Avenue. The crossing would require a break in the raised concrete median of 1.8-2.0m to allow for the permeability of bicycles but prevent cut-through vehicular traffic from travelling north-south. Turn restriction signs for vehicular traffic would require “bicycles excepted” signage for cut-through bicycle traffic only.

Given that the average annual daily traffic (AADT) for this segment of Mapleview Drive exceeds 9,000 vehicles with a design speed in excess of 60 km/hr, a median refuge is recommended with a minimum width of 2m and an ideal width of 3m to allow a two-stage crossing for cyclists. For a 4-lane cross-section, an east-west sight distance of at least 300m is recommended for approaching cyclists, with stop controls on Dean Avenue.

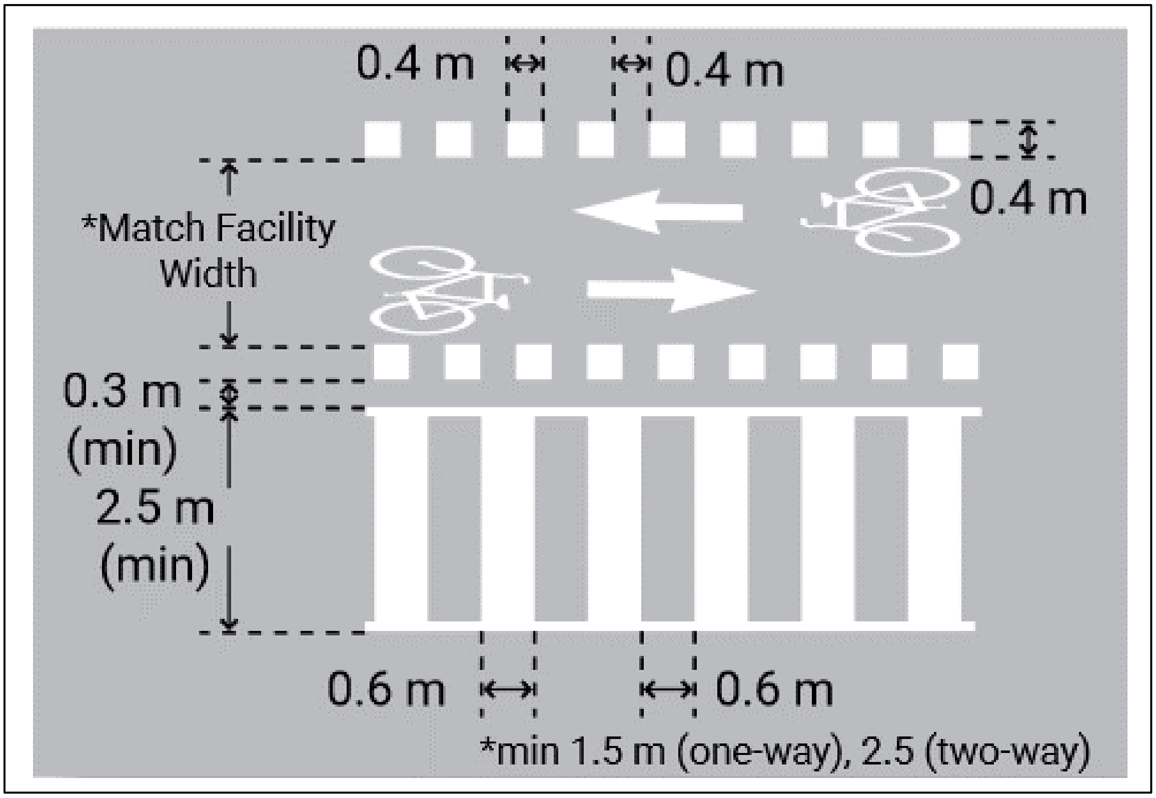
It is noted that the presence of this uncontrolled crossing for cyclists may compete with pedestrian demand for the proposed IPS on the east leg of the crossing and is likely to encourage uncontrolled pedestrian crossings.

The proposed uncontrolled crossing is at the limits of applicability for lane width (a maximum of 4 lanes), posted speed limits (60 km/hr) and daily traffic (9,000 vehicles per day). The existing concrete median has a width less than 2m which is the minimum to safely function as a refuge island for a two-stage crossing. Additionally, the crossing would encourage uncontrolled pedestrian crossings. Given this, it is not recommended that an uncontrolled bicycle crossing be implemented.

Protected Phase with IPS

Crossride markings may be added to the proposed intersection pedestrian signal (IPS) to accommodate bicycles to cross with pedestrians when a sufficient gap is available, protecting bicycle and pedestrian movements with a dedicated phase. The raised concrete median may be maintained on either side of the crossing, with stop lines set back at least 12m from the edge of the crossride markings as per OTM Book 18. A schematic of this is shown in Figure 4-4.

Figure 4-4 Crossride with IPS



(Source: OTM Book 18)

It is noted that although this configuration serves two-way bicycle traffic, there is no counterflow lane at either end of the crossing to permit southbound cyclists to continue cycling opposing the flow of traffic. As a result, this configuration would require a bi-directional multiuse path on the east side of Dean Avenue.

Alternatively, southbound cyclists may be forced to dismount and perform a three-stage crossing east across Dean Avenue, south across the IPS and west across Dean Avenue. This configuration is less preferable than a full-movement, signalized intersection as suggested in Section 4.4.

5.0 Traffic Circulation Review

LMM conducted a review of the proposed site plan with respect to traffic circulation for all transportation modes. The proposed development consists of 3 accesses as detailed below:

- A full movement, unsignalized access on Dean Avenue, approximately 110m north of Mapleview Drive East.
- A right-out (egress) on Mapleview Drive East, approximately 40m east of Dean Avenue
- A right-in (ingress) on Mapleview Drive East, approximately 120m east of Dean Avenue

All proposed accesses are anticipated to have a stop-control applied to site traffic, with the public road remaining free of traffic controls.

5.1 Dean Avenue Access

The proposed access on Dean Avenue would serve north-south traffic along Dean Avenue, as well as vehicle trips originating west of the site.
The proposed throat width of 7.0m and drive aisle widths of 6.5m are expected to accommodate typical vehicular traffic (passenger vehicles) as well as emergency vehicles circulating the site. The parking lot connected to this access provides for parking spaces as well as a drop-off area. A fire route is demarcated on the site plan, with the drop-off lay-by located outside of the path of the fire route.

There is one Type A and one Type B accessible parking space located in the southwest corner of the parking lot closest to the concrete sidewalk that connects to the school building. A sidewalk connection from the site to the existing east-side sidewalk on Dean Avenue is provided at 2 locations. The proposed drop-off area is also located adjacent to a sidewalk providing connections to the school building. Marked crosswalks are recommended where the sidewalks cross the driveway throat.

The proposed access at Dean Avenue is expected to sufficiently accommodate multimodal traffic entering / exiting the site as well as facilitate drop-offs. Emergency vehicles are expected to manoeuvre the site safely.

5.2 Mapleview Drive Accesses

The proposed accesses onto Mapleview drive would serve vehicle trips originating east of the site and exiting westbound and are configured as right-in, right-out (RIRO) accesses.

To discourage eastbound left turns into the site at Mapleview Avenue an extension of the raised median and/or the angling of the site access away from eastbound traffic is recommended.

The proposed throat widths of 6.5m and the one-way drive aisle width of 4.4m are expected to sufficiently accommodate site traffic. A lay-by for bus drop-offs is provided, with a width of 3.0m. The location of the bus lay-by lane is not expected to impact parking manoeuvres in this parking lot.

As indicated on the site plan, pedestrian access is provided by maintaining the existing sidewalk along Mapleview Drive East. The internal sidewalk network connects to the existing sidewalk network at the ingress and egress points for vehicular traffic. Marked crosswalks are recommended where the sidewalks cross the driveway throat.

5.3 Jessica Drive Trail Access

In addition to vehicular accesses, there is a proposed pedestrian and bicycle connection to Jessica Drive via D & J Fralick Park. This proposed connection provides a walkway for trips originating northeast of the site through the park and connects to the internal walkway network of the school. No conflicts with vehicular traffic are anticipated for this pedestrian connection.

6.0 Conclusions and Recommendations

LMM Engineering Inc. was retained by Simcoe County District School Board (SCDSB) to undertake a traffic impact study to evaluate the traffic impacts of the proposed elementary school on the site located at 210 Dean Avenue in the City of Barrie, ON. The site is located at the northeast quadrant of the intersection of Dean Avenue and Mapleview Drive East.

The proposed development consists of a three-storey school building with associated recreational space, parking lots and drop-off areas. The total gross floor area is 5,857sqm. The proposed school is designed for a capacity of 938 students. The subject site is currently vacant.

The results of the existing condition intersection capacity analysis indicate that all the existing study intersections currently operate at acceptable levels of service.

The study area is expected to undergo a significant increase in background traffic due to the development of the Hewitt Lands south of the subject site as well as the general growth rate from new employment centres, institutions and residential areas throughout southeast Barrie, as informed by the Hewitt Lands Master Transportation Study and Functional Design Review.

A previous submission of the traffic study by LMM identified a full-movement, signalized intersection at Mapleview Drive East and Dean Avenue as the ideal solution for improved mobility for all road users. Following a review of the study with City staff, staff stated that the raised concrete median is to be retained. Upon further discussion and in order to accurately model future conditions, City staff were asked by SCDSB staff for additional traffic information and data including previous studies which may assist in the understanding as to how the need or requirement for the raised median came to be. Further correspondence from City staff advised there was no additional traffic information or data available and the decision for the construction of the raised concrete median was based solely on political direction to deter north/south traffic from lands on the south side of Mapleview Drive East north onto Dean Avenue. As a result, an alternative solution was required by City staff for the intersection of Mapleview Drive East and Dean Avenue.

An alternative analysis was conducted (option 2) which retains the raised concrete median with left-turn restrictions in place for eastbound and westbound vehicular traffic as well as left-through restrictions in place for northbound and southbound vehicular traffic. The results of the future intersection capacity analysis indicate that all future study intersections are expected to operate at acceptable levels of service (LOS) given the following changes to the road network:

- Madelaine Drive & Mapleview Drive East:
  - Dual northbound left-turn lanes, as per the Hewitt Lands Secondary Plan.
  - Protected phases for the eastbound left turn and northbound left turn.
- Goodwin Drive & Mapleview Drive East
  - Dedicated left-turn lane and a shared through-right turn lane, as per the Hewitt Lands Secondary Plan.

An intersection pedestrian crossing (IPS) is recommended on the east approach of the intersection of Mapleview Drive East and Dean Avenue to facilitate a protected north-south crossing for pedestrians. The IPS is to be in standard two-phase operation and constructed as per minimum standards defined in the Ontario Traffic Manual (OTM). Retention of the raised concrete median on either end of the IPS is anticipated to provide further traffic calming. This proposed solution would maintain turning restrictions at the intersection and maintain westbound access to properties on the south side of Mapleview Drive East.

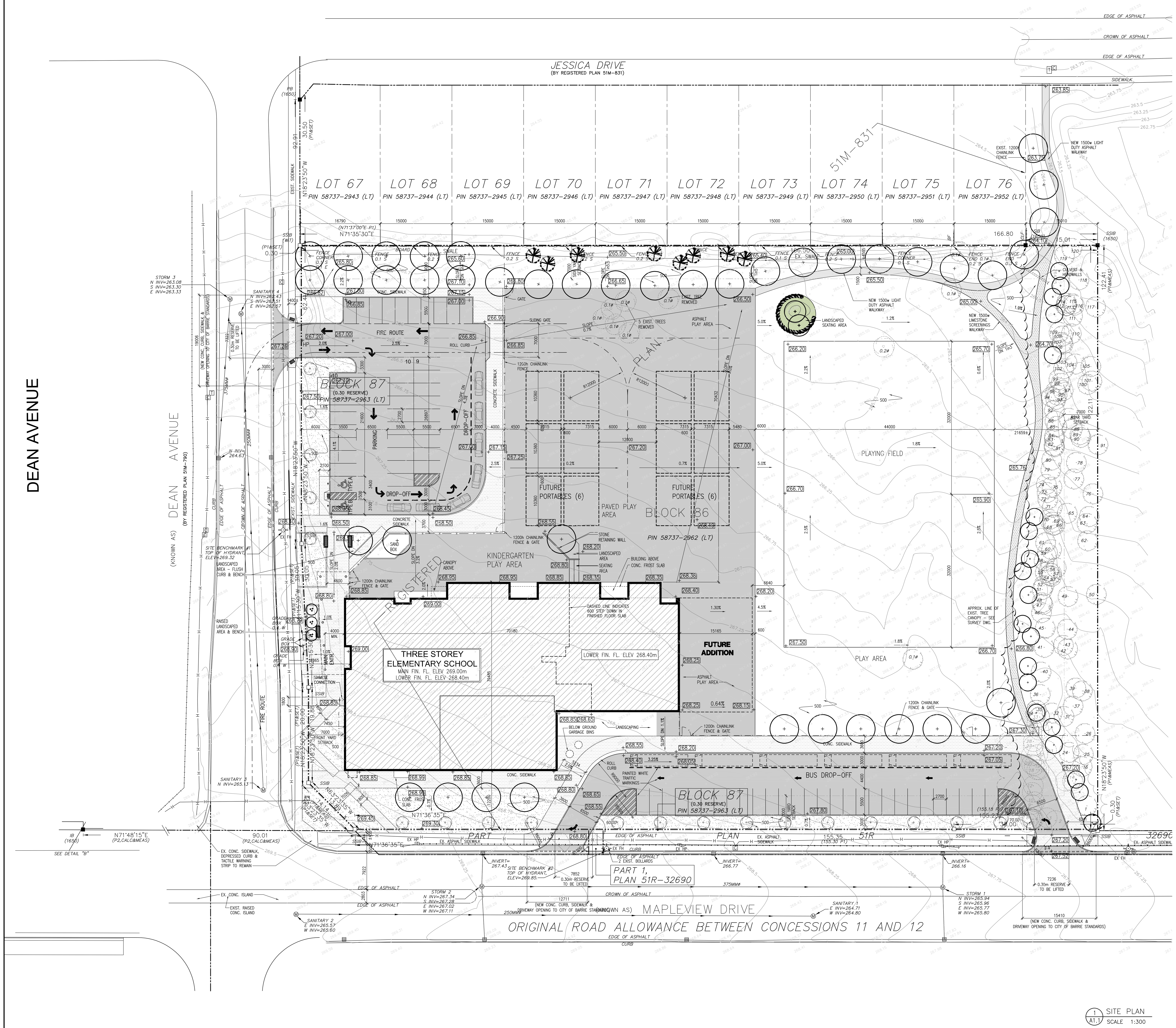
An uncontrolled crossing for bicycles is not feasible due to the width of the concrete median being insufficient as well as its potential to encourage uncontrolled pedestrian crossings. A crossride is recommended adjacent to the IPS. It is noted that southbound bicycle trips along Dean Avenue would require a counterflow bicycle lane along Dean Avenue by way of a multi-use path, or have cyclists dismount and use the IPS as pedestrians in order to maintain permeability.

The subject proposed development will generally not have a significant impact on the road network.

The proposed site accesses and drop-off areas are able to safely accommodate site traffic including the ingress, circulation and egress for passenger vehicles, waste collection, emergency vehicles and school buses. Lay-by areas for drop-offs are located outside of drive aisles, limiting the likelihood of drive aisle obstruction. Pedestrian connections are provided at all proposed accesses. It is recommended that marked crosswalks be used where existing sidewalks cross driveway entrances.

Appendix A

Proposed Site Plan



1 SITE PLAN A1.1 SCALE 1:300

KEY PLAN N.T.S.

MUNICIPAL ADDRESS

__ DEAN AVENUE, BARRIE, ONTARIO

SURVEY

SURVEY INFORMATION TAKEN FROM PLAN OF SURVEY OF BLOCK 86, REGISTERED PLAN 51M-831 CITY OF BARRIE, COUNTY OF SIMCOE AS PREPARED BY RUDY MAK SURVEYINGI LTD. OLS DATED FEBRUARY 5, 2024

EASEMENTS

NONE

1' RESERVE ALONG MAPLEVIEW DRIVE & DEAN AVENUE.

ELEVATION NOTE

ELEVATION ARE GEODETIC IN ORIGIN AND WERE DERIVED FROM SPECIFIED PROVINCIAL COSINE CONTROL POINTS 0011931U460S (ELEVATION=263.48, CGVD-1928:1978) AND 03120080008 (ELEVATION=247.827, CGVD-1928:1978).

BENCH MARK

SITE BENCHMARK #1 IS THE TOP OF THE HYDRANT ON DEAN AVENUE, HAVING AN ELEVATION OF 269.32.

SITE BENCHMARK #2 IS THE TOP OF THE HYDRANT ON MAPLEVIEW DRIVE, HAVING AN ELEVATION OF 269.85.

BEARING NOTE

BEARINGS ARE UTM GRID, DERIVED FROM SPECIFIED CONTROL POINTS 0011931U460S AND 03120080008, UTM ZONE 17, NAD 83 (ORIGINAL)

NOTE: LOCATION OF CURBS SHOWN ON THIS DRAWING ALONG NORTH SIDE OF SPRING FARM ROAD, EAST SIDE OF CONOVER AVE. & SOUTH SIDE OF BOREALIS AVE. ARE APPROXIMATE AND ARE DERIVED FROM MUNICIPAL ENGINEERING DRAWINGS 401, 403, 404 & G-3 BY STANTEC CONSULTING LTD. DATED FEBRUARY 2003 PROJECT No. 20267

GROSS FLOOR AREA:

GROUND FLOOR = $2330.9m^2$
SECOND FLOOR = $1854.2m^2$
THIRD FLOOR = $1672.4m^2$
TOTAL = $5857.0m^2$

PARKING REQUIREMENTS:

REQUIRED BOARD PARKING SPACES: 60
INCLUSIVE OF 2 BF SPACES(1.5 SPACES PER TEACHING SPACE PLUS 12 PORTABLES)

REQUIRED ZONING BY-LAW PARKING SPACES: 43
(1.0 SPACES PER CLASSROOM PLUS 3 OFFICES PLUS 12 PORTABLES.)

TOTAL PARKING SPACES PROVIDED:

- STANDARD PARKING 65
- BARRIER FREE TYPE 'A' 1
- BARRIER FREE TYPE 'B' 1

TOTAL PARKING SPACES PROVIDED: 67

LEGEND

EXISTING

- IRON BAR
- SURVEY MONUMENT SET
- LS LIGHT STANDARD (MUNICIPAL)
- CATCH BASIN
- MH MANHOLE
- HYD FIRE HYDRANT
- 135.5 EXISTING CONTOURS
- +172.18 EXISTING ELEVATION
- BH7 BORE HOLE

PROPOSED

- 132.44+ NEW GRADES
- MH MANHOLE
- CBMH CATCH BASIN MANHOLE
- CB CATCH BASIN (REFER TO AD/107)
- FH FIRE HYDRANT
- LS LIGHT STANDARD (SCHOOL)
- B BOLLARDS
- CHAIN LINK FENCE
- W WEEPING TILE
- CONCRETE CURBS
- PAINTED ARROWS
- PAINTED BARRIER FREE SYMBOL
- FRS FIRE ROUTE SIGN INSTALLED VERTICALLY AT A HEIGHT OF 1700MM TO 2000MM FROM THE GROUND TO BOTTOM OF THE SIGN
- FRS 2 FIRE ROUTE SIGN WITH TAB 'NO PARKING BOTH SIDES OF DRIVEWAY' INSTALLED VERTICALLY OR WALL MOUNTED AT A HEIGHT OF 1700MM TO 2000MM FROM THE GROUND TO BOTTOM OF THE SIGN
- WR WASTE RECEPTACLE
- MAIN SECONDARY SERVICE ENTRANCES INTO BUILDING - INCLUDING MECHANICAL & STORAGE ROOM DOORS AND OVERHEAD DOORS
- BF 'B.F.' INDICATES A BARRIER FREE ENTRANCE - FLUSH THRESHOLD -BF OPERATOR AND DOORS
- CONCRETE A
- CONCRETE B
- CONCRETE PAVING
- HEAVY DUTY CONCRETE PAVING
- CONCRETE SLAB ON GRADE OR FROST SLAB
- PAINTED LINES
- HEAVY DUTY ASPHALT PAVING
- MEDIUM DUTY ASPHALT PAVING
- LIGHT DUTY ASPHALT PAVING
- PAVER
- SOD

NOTES

1) REFER TO DRAWING GR-1 FOR GRADING INFORMATION AND SS-1 FOR SITE SERVICES.
2) ALL WORK UNDERTAKEN WITHIN THE ROAD RIGHT-OF-WAY SHALL BE DONE TO APPLICABLE TOWN OF AURORA STANDARDS.
3) REMOVE AND REPLACE SIDEWALK AT DRIVEWAYS. NEW SIDEWALK TO CONFORM WITH TOWN OF AURORA STANDARDS.
4) REMOVE AND REPLACE CURB AND GUTTER AT DRIVEWAYS. CAST MONOLITHIC TRANSITIONS AT DRIVEWAY CURB RETURNS AND CONTINUE STREET PROFILE TO EDGE OF SIDEWALK TO TOWN OF AURORA STANDARDS.
5) REFER TO LANDSCAPE DRAWINGS FOR PLANTING, SODDING, PLANTING BED, AND LANDSCAPE INFORMATION
6) PROVIDE SILT FENCING AROUND ENTIRE SITE IN ACCORDANCE WITH DETAILS.
7) CONFORM TO TOWN OF AURORA STANDARDS FOR DRIVEWAY ENTRANCES.
8) REFER TO GEOTECHNICAL REPORT PREPARED BY FORWARD ENGINEERING & ASSOCIATES INC. DATED NOVEMBER 14, 2019 FOR BOREHOLE INFORMATION.
9) CHAINLINK FENCES ALONG PROPERTY LINES TO BE LOCATED ENTIRELY ON THE PROPERTY INCLUDING CONCRETE POST ENCASEMENTS.
10) ROOF DESIGNED FOR FREE FLOW DRAINAGE. REFER TO ROOF PLAN FOR OVERFLOW SCUPPER LOCATIONS
11) WHERE CHAINLINK FENCES ARE INDICATED AT THE TOP OF RETAINING WALLS THEY ARE TO EXTEND ALONG THE FULL LENGTH OF THE RETAINING WALL.
12) DISTURBED AREAS WITHIN THE TOWN OF AURORA RIGHT OF WAY SHALL BE REINSTATED TO TOWN OF AURORA STANDARD 208 INCLUDING A MINIMUM UNIFORM DEPTH OF A 150mm OF TOPSOIL AND No. 1 NURSERY SOD.
13) FOR TRAFFIC SIGNAGE REFER TO DRAWING A1.3.

WORK IN PROGRESS

MAY 6, 2024

DATE		REVISIONS

Barrie Southeast Elementary School #1

210 DEAN AVE. BARRIE, ONTARIO

Simcoe County District School Board

DRAWING: SITE PLAN

TRUE NORTH | DWG. NORTH

JOB NO.: 2410

SCALE: 1:300

DATE: March 2024

PRINTED: May 9, 2024

DWG. NO.: A1.1

MOFFET & DUNCAN architects inc.
5052 DUNDAS ST. WEST
ISLINGTON, ONT. M9A 1B9
TEL: (416) 239-2775
FAX: (416) 239-6729
EMAIL: mdarch@mdarch.ca

Appendix B

Intersection Capacity Analysis Output
Existing Conditions

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	114	365	0	0	400	31	0	0	0	17	0	132
Future Volume (vph)	114	365	0	0	400	31	0	0	0	17	0	132
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Util. Factor	0.95	0.95	0.95	0.95	0.95	0.95	1.00	1.00	1.00	0.95	0.95	1.00
Frt					0.989						0.852	
Flt Protected		0.988								0.950	0.999	
Satd. Flow (prot)	0	3536	0	0	3539	0	1883	1883	0	1700	1523	0
Flt Permitted		0.988								0.950	0.999	
Satd. Flow (perm)	0	3536	0	0	3539	0	1883	1883	0	1700	1523	0
Link Speed (k/h)		48			48			48			48	
Link Distance (m)		545.0			452.2			185.6			267.4	
Travel Time (s)		40.9			33.9			13.9			20.1	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	124	397	0	0	435	34	0	0	0	18	0	143
Shared Lane Traffic (%)										10%		
Lane Group Flow (vph)	0	521	0	0	469	0	0	0	0	16	145	0
Enter Blocked Intersection	No	No	No	No	No	No	No	No	No	No	No	No
Lane Alignment	Left	Left	Right	Left	Left	Right	Left	Left	Right	Left	Left	Right
Median Width(m)		3.7			3.7			3.7			3.7	
Link Offset(m)		0.0			0.0			0.0			0.0	
Crosswalk Width(m)		4.9			4.9			4.9			4.9	
Two way Left Turn Lane												
Headway Factor	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Turning Speed (k/h)	24		14	24		14	24		14	24		14
Sign Control		Free			Free			Stop			Stop	

Intersection Summary

Area Type: Other

Control Type: Unsignalized

Intersection Capacity Utilization 40.0% ICU Level of Service A

Analysis Period (min) 15

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	114	365	0	0	400	31	0	0	0	17	0	132
Future Volume (Veh/h)	114	365	0	0	400	31	0	0	0	17	0	132
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	124	397	0	0	435	34	0	0	0	18	0	143
Pedestrians												
Lane Width (m)												
Walking Speed (m/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (m)												
pX, platoon unblocked												
vC, conflicting volume	469			397			1006	1114	199	899	1097	235
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	469			397			1006	1114	199	899	1097	235
tC, single (s)	4.1			4.1			7.5	6.5	6.9	7.5	6.5	6.9
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	89			100			100	100	100	92	100	81
cM capacity (veh/h)	1089			1158			145	183	809	214	188	767

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	NB 1	NB 2	SB 1	SB 2
Volume Total	323	199	218	252	0	0	12	149
Volume Left	124	0	0	0	0	0	12	6
Volume Right	0	0	0	34	0	0	0	143
cSH	1089	1700	1158	1700	1700	1700	214	695
Volume to Capacity	0.11	0.12	0.00	0.15	0.00	0.00	0.06	0.21
Queue Length 95th (m)	2.9	0.0	0.0	0.0	0.0	0.0	1.3	6.2
Control Delay (s/veh)	4.0	0.0	0.0	0.0	0.0	0.0	22.8	11.6
Lane LOS	A				A	A	C	B
Approach Delay (s/veh)	2.5		0.0		0.0		12.4	
Approach LOS					A		B	

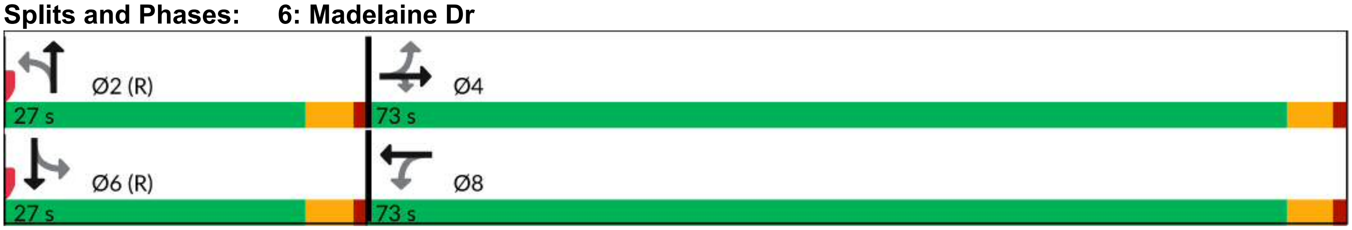
Intersection Summary			
Average Delay	2.9		
Intersection Capacity Utilization	40.0%	ICU Level of Service	A
Analysis Period (min)	15		

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	122	488	0	0	524	4	0	0	0	1	0	57
Future Volume (vph)	122	488	0	0	524	4	0	0	0	1	0	57
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	150.0		63.0	150.0		45.0	40.0		0.0	30.0		30.0
Storage Lanes	1		1	1		0	1		0	1		0
Taper Length (m)	7.6			7.6			7.6			7.6		
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	0.95	1.00	0.95	0.95	1.00	0.95	0.95
Frt					0.999						0.850	
Flt Protected	0.950									0.950		
Satd. Flow (prot)	1789	3579	1883	1883	3575	0	1883	3579	0	1789	3042	0
Flt Permitted	0.436									0.757		
Satd. Flow (perm)	821	3579	1883	1883	3575	0	1883	3579	0	1426	3042	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)					2						372	
Link Speed (k/h)		48			48			48			48	
Link Distance (m)		529.5			545.0			261.3			299.3	
Travel Time (s)		39.7			40.9			19.6			22.4	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	133	530	0	0	570	4	0	0	0	1	0	62
Shared Lane Traffic (%)												
Lane Group Flow (vph)	133	530	0	0	574	0	0	0	0	1	62	0
Enter Blocked Intersection	No	No	No	No	No	No	No	No	No	No	No	No
Lane Alignment	Left	Left	Right	Left	Left	Right	Left	Left	Right	Left	Left	Right
Median Width(m)		3.7			3.7			3.7			3.7	
Link Offset(m)		0.0			0.0			0.0			0.0	
Crosswalk Width(m)		4.9			4.9			4.9			4.9	
Two way Left Turn Lane												
Headway Factor	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Turning Speed (k/h)	24		14	24		14	24		14	24		14
Number of Detectors	1	2	1	1	2		1	2		1	2	
Detector Template	Left	Thru	Right	Left	Thru		Left	Thru		Left	Thru	
Leading Detector (m)	6.1	30.5	6.1	6.1	30.5		6.1	30.5		6.1	30.5	
Trailing Detector (m)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Detector 1 Position(m)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Detector 1 Size(m)	6.1	1.8	6.1	6.1	1.8		6.1	1.8		6.1	1.8	
Detector 1 Type	Cl+Ex	Cl+Ex	Cl+Ex	Cl+Ex	Cl+Ex		Cl+Ex	Cl+Ex		Cl+Ex	Cl+Ex	
Detector 1 Channel												
Detector 1 Extend (s)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Detector 1 Queue (s)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Detector 1 Delay (s)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Detector 2 Position(m)		28.7			28.7			28.7			28.7	
Detector 2 Size(m)		1.8			1.8			1.8			1.8	
Detector 2 Type		Cl+Ex			Cl+Ex			Cl+Ex			Cl+Ex	
Detector 2 Channel												
Detector 2 Extend (s)		0.0			0.0			0.0			0.0	
Turn Type	Perm	NA	Perm	Perm	NA		Perm			Perm	NA	
Protected Phases		4			8			2			6	
Permitted Phases	4		4	8			2			6		

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Detector Phase	4	4	4	8	8		2	2		6	6	
Switch Phase												
Minimum Initial (s)	5.0	5.0	5.0	5.0	5.0		5.0	5.0		5.0	5.0	
Minimum Split (s)	22.5	22.5	22.5	22.5	22.5		22.5	22.5		22.5	22.5	
Total Split (s)	73.0	73.0	73.0	73.0	73.0		27.0	27.0		27.0	27.0	
Total Split (%)	73.0%	73.0%	73.0%	73.0%	73.0%		27.0%	27.0%		27.0%	27.0%	
Maximum Green (s)	68.5	68.5	68.5	68.5	68.5		22.5	22.5		22.5	22.5	
Yellow Time (s)	3.5	3.5	3.5	3.5	3.5		3.5	3.5		3.5	3.5	
All-Red Time (s)	1.0	1.0	1.0	1.0	1.0		1.0	1.0		1.0	1.0	
Lost Time Adjust (s)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.5	4.5	4.5	4.5	4.5		4.5	4.5		4.5	4.5	
Lead/Lag												
Lead-Lag Optimize?												
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0		3.0	3.0		3.0	3.0	
Recall Mode	Max	Max	Max	Max	Max		C-Min	C-Min		C-Min	C-Min	
Walk Time (s)	7.0	7.0	7.0	7.0	7.0		7.0	7.0		7.0	7.0	
Flash Dont Walk (s)	11.0	11.0	11.0	11.0	11.0		11.0	11.0		11.0	11.0	
Pedestrian Calls (#/hr)	0	0	0	0	0		0	0		0	0	
Act Effct Green (s)	85.3	85.3			85.3					5.7	5.7	
Actuated g/C Ratio	0.85	0.85			0.85					0.06	0.06	
v/c Ratio	0.19	0.17			0.19					0.01	0.12	
Control Delay (s/veh)	2.0	1.4			1.4					45.0	0.5	
Queue Delay	0.0	0.0			0.0					0.0	0.0	
Total Delay (s/veh)	2.0	1.4			1.4					45.0	0.5	
LOS	A	A			A					D	A	
Approach Delay (s/veh)		1.5			1.4						1.2	
Approach LOS		A			A						A	

Intersection Summary	
Area Type: Other	
Cycle Length: 100	
Actuated Cycle Length: 100	
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green	
Natural Cycle: 45	
Control Type: Actuated-Coordinated	
Maximum v/c Ratio: 0.19	
Intersection Signal Delay (s/veh): 1.5	Intersection LOS: A
Intersection Capacity Utilization 36.8%	ICU Level of Service A
Analysis Period (min) 15	

Splits and Phases: 6: Madelaine Dr



Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	122	488	0	0	524	4	0	0	0	1	0	57
Future Volume (vph)	122	488	0	0	524	4	0	0	0	1	0	57
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.5	4.5			4.5					4.5	4.5	
Lane Util. Factor	1.00	0.95			0.95					1.00	0.95	
Frt	1.00	1.00			1.00					1.00	0.85	
Flt Protected	0.95	1.00			1.00					0.95	1.00	
Satd. Flow (prot)	1789	3579			3575					1789	3042	
Flt Permitted	0.44	1.00			1.00					0.76	1.00	
Satd. Flow (perm)	821	3579			3575					1426	3042	
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	133	530	0	0	570	4	0	0	0	1	0	62
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	58	0
Lane Group Flow (vph)	133	530	0	0	574	0	0	0	0	1	4	0
Turn Type	Perm	NA	Perm	Perm	NA		Perm			Perm	NA	
Protected Phases		4			8			2			6	
Permitted Phases	4		4	8			2			6		
Actuated Green, G (s)	85.3	85.3			85.3					5.7	5.7	
Effective Green, g (s)	85.3	85.3			85.3					5.7	5.7	
Actuated g/C Ratio	0.85	0.85			0.85					0.06	0.06	
Clearance Time (s)	4.5	4.5			4.5					4.5	4.5	
Vehicle Extension (s)	3.0	3.0			3.0					3.0	3.0	
Lane Grp Cap (vph)	700	3052			3049					81	173	
v/s Ratio Prot		0.15			0.16						c0.00	
v/s Ratio Perm	c0.16									0.00		
v/c Ratio	0.19	0.17			0.19					0.01	0.02	
Uniform Delay, d1	1.3	1.3			1.3					44.5	44.5	
Progression Factor	1.00	1.00			1.00					1.00	1.00	
Incremental Delay, d2	0.6	0.1			0.1					0.3	0.2	
Delay (s)	1.9	1.4			1.4					44.8	44.7	
Level of Service	A	A			A					D	D	
Approach Delay (s/veh)		1.5			1.4			0.0			44.7	
Approach LOS		A			A			A			D	

Intersection Summary			
HCM 2000 Control Delay (s/veh)	3.6	HCM 2000 Level of Service	A
HCM 2000 Volume to Capacity ratio	0.18		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	9.0
Intersection Capacity Utilization	36.8%	ICU Level of Service	A
Analysis Period (min)	15		

c Critical Lane Group

Lane Group	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (vph)	3	382	431	4	2	1
Future Volume (vph)	3	382	431	4	2	1
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Storage Length (m)	200.0			0.0	0.0	0.0
Storage Lanes	1			0	1	1
Taper Length (m)	7.6				7.6	
Lane Util. Factor	1.00	0.95	0.95	0.95	1.00	1.00
Frt			0.999			0.850
Flt Protected	0.950				0.950	
Satd. Flow (prot)	1789	3579	3575	0	1789	1601
Flt Permitted	0.950				0.950	
Satd. Flow (perm)	1789	3579	3575	0	1789	1601
Link Speed (k/h)		48	48		48	
Link Distance (m)		452.2	355.3		358.6	
Travel Time (s)		33.9	26.6		26.9	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	3	415	468	4	2	1
Shared Lane Traffic (%)						
Lane Group Flow (vph)	3	415	472	0	2	1
Enter Blocked Intersection	No	No	No	No	No	No
Lane Alignment	Left	Left	Left	Right	Left	Right
Median Width(m)		3.7	3.7		3.7	
Link Offset(m)		0.0	0.0		0.0	
Crosswalk Width(m)		4.9	4.9		4.9	
Two way Left Turn Lane						
Headway Factor	0.99	0.99	0.99	0.99	0.99	0.99
Turning Speed (k/h)	24			14	24	14
Sign Control		Free	Free		Stop	

Intersection Summary	
Area Type: Other	
Control Type: Unsignalized	
Intersection Capacity Utilization 22.0%	ICU Level of Service A
Analysis Period (min) 15	

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	3	382	431	4	2	1
Future Volume (Veh/h)	3	382	431	4	2	1
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	3	415	468	4	2	1
Pedestrians						
Lane Width (m)						
Walking Speed (m/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (m)						
pX, platoon unblocked						
vC, conflicting volume	472				684	236
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	472				684	236
tC, single (s)	4.1				6.8	6.9
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	100				99	100
cM capacity (veh/h)	1086				382	766

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	SB 1	SB 2
Volume Total	3	208	208	312	160	2	1
Volume Left	3	0	0	0	0	2	0
Volume Right	0	0	0	0	4	0	1
cSH	1086	1700	1700	1700	1700	382	766
Volume to Capacity	0.00	0.12	0.12	0.18	0.09	0.01	0.00
Queue Length 95th (m)	0.1	0.0	0.0	0.0	0.0	0.1	0.0
Control Delay (s/veh)	8.3	0.0	0.0	0.0	0.0	14.5	9.7
Lane LOS	A					B	A
Approach Delay (s/veh)	0.1			0.0		12.9	
Approach LOS						B	

Intersection Summary			
Average Delay	0.1		
Intersection Capacity Utilization	22.0%	ICU Level of Service	A
Analysis Period (min)	15		

Appendix C

Total Traffic Volumes

Master Transportation Study Excerpt

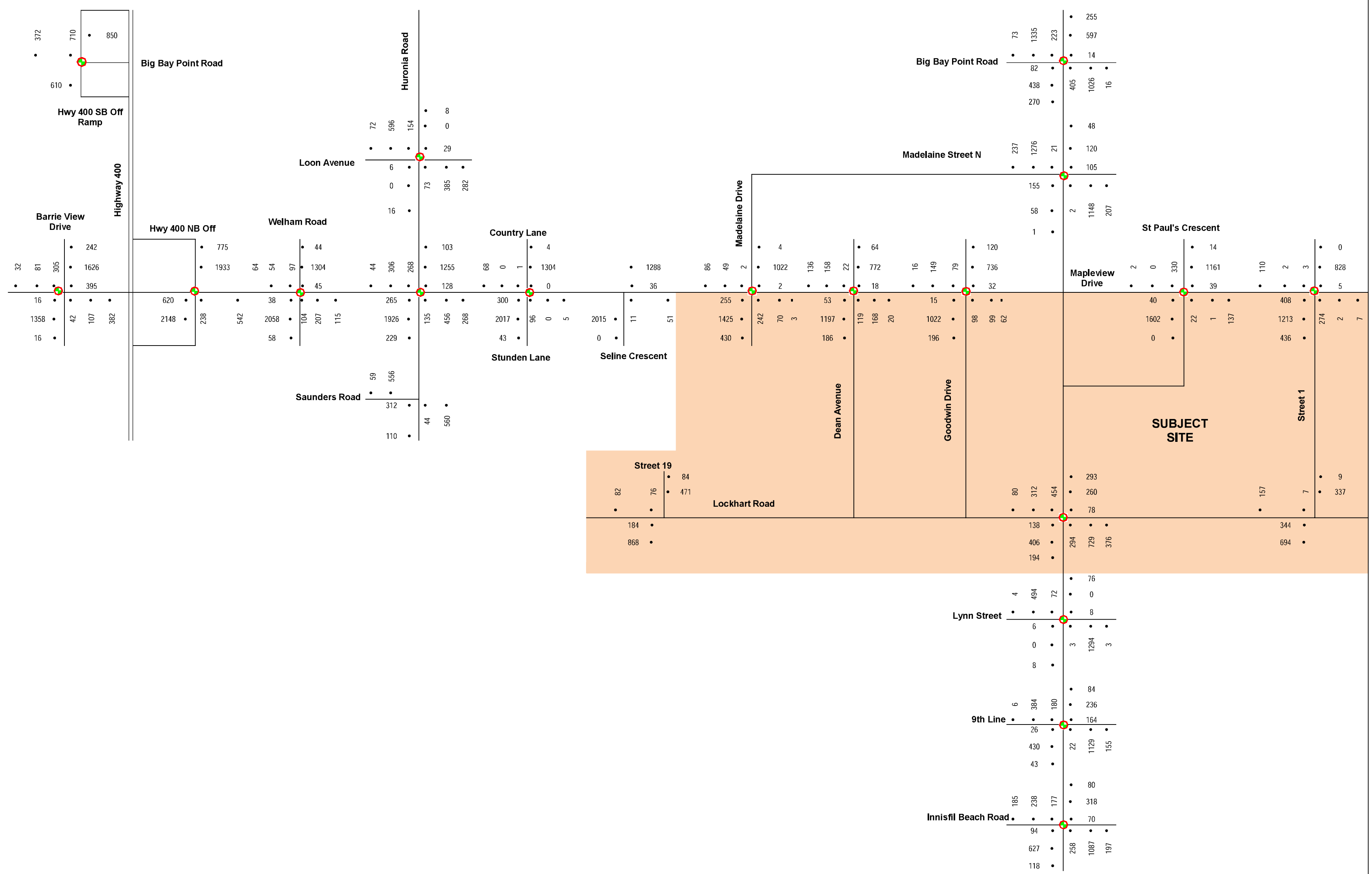


FIGURE 4-3
Future Total Traffic Volumes- Ultimate Horizon (2031)
Weekday PM Peak Hour
Constraint Ateas

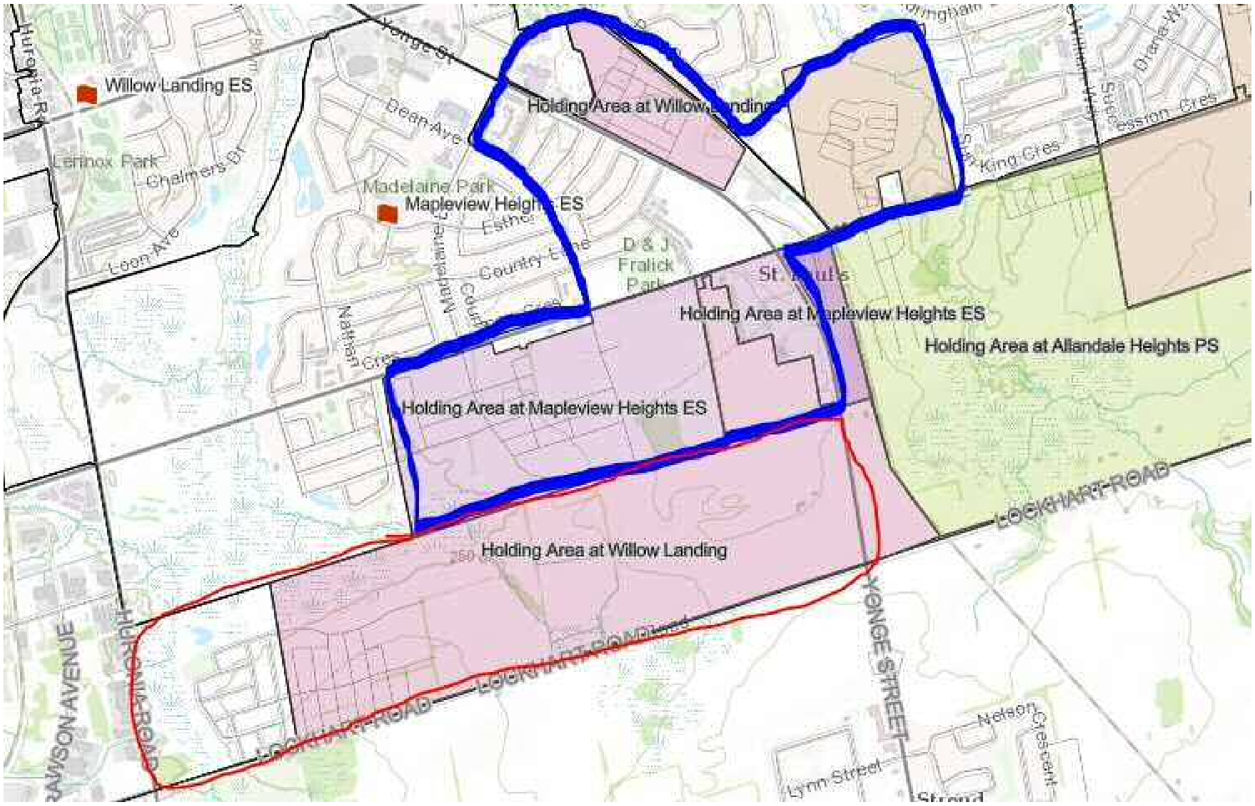
LEGEND
XX Peak Hour Traffic Volumes
Signalized Intersection

NOT TO SCALE

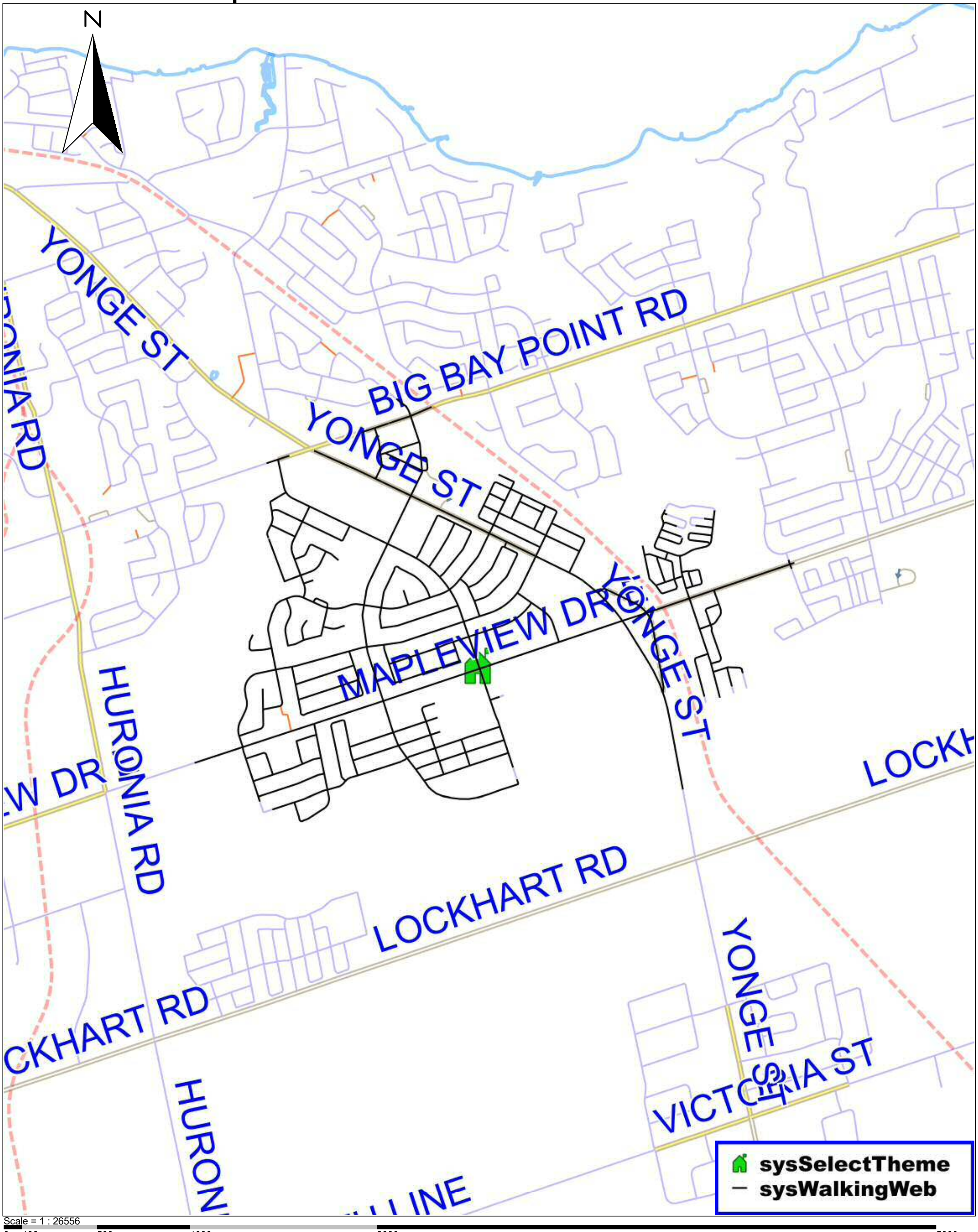
Appendix D

Catchment Area for Proposed School

Proposed School Catchment Area – Provided by Simcoe Country District School Board



Mapleview & Dean walk zone 1.6kms



Appendix E

Intersection Capacity Analysis Output
Future Total Condition – Option 1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	53	1197	186	18	772	64	119	168	20	22	158	136
Future Volume (vph)	53	1197	186	18	772	64	119	168	20	22	158	136
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.5			4.5		4.5	4.5		4.5	4.5	
Lane Util. Factor		0.95			0.95		1.00	1.00		0.95	0.95	
Frt		0.98			0.99		1.00	0.98		1.00	0.93	
Flt Protected		1.00			1.00		0.95	1.00		0.95	1.00	
Satd. Flow (prot)		3503			3534		1789	1853		1700	1665	
Flt Permitted		0.88			0.90		0.32	1.00		0.57	1.00	
Satd. Flow (perm)		3097			3194		611	1853		1021	1663	
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	58	1301	202	20	839	70	129	183	22	24	172	148
RTOR Reduction (vph)	0	17	0	0	9	0	0	8	0	0	56	0
Lane Group Flow (vph)	0	1544	0	0	920	0	129	197	0	22	266	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		4			8			2			6	
Permitted Phases	4			8			2			6		
Actuated Green, G (s)		36.5			36.5		14.5	14.5		14.5	14.5	
Effective Green, g (s)		36.5			36.5		14.5	14.5		14.5	14.5	
Actuated g/C Ratio		0.61			0.61		0.24	0.24		0.24	0.24	
Clearance Time (s)		4.5			4.5		4.5	4.5		4.5	4.5	
Vehicle Extension (s)		3.0			3.0		3.0	3.0		3.0	3.0	
Lane Grp Cap (vph)		1884			1943		147	447		246	401	
v/s Ratio Prot								0.11				
v/s Ratio Perm		c0.50			0.29		c0.21			0.02	0.16	
v/c Ratio		0.82			0.47		0.88	0.44		0.09	0.66	
Uniform Delay, d1		9.2			6.5		21.9	19.3		17.6	20.5	
Progression Factor		1.00			1.00		1.00	1.00		1.00	1.00	
Incremental Delay, d2		4.1			0.8		47.2	3.1		0.7	8.4	
Delay (s)		13.3			7.3		69.1	22.5		18.4	28.9	
Level of Service		B			A		E	C		B	C	
Approach Delay (s/veh)		13.3			7.3			40.5			28.2	
Approach LOS		B			A			D			C	

Intersection Summary			
HCM 2000 Control Delay (s/veh)	16.0	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.83		
Actuated Cycle Length (s)	60.0	Sum of lost time (s)	9.0
Intersection Capacity Utilization	98.4%	ICU Level of Service	F
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	255	1425	430	2	1022	4	242	70	3	2	49	86
Future Volume (vph)	255	1425	430	2	1022	4	242	70	3	2	49	86
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.5	4.5	4.5	4.5	4.5		4.5	4.5		4.5	4.5	
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95		1.00	0.95		1.00	0.95	
Frt	1.00	1.00	0.85	1.00	1.00		1.00	0.99		1.00	0.90	
Flt Protected	0.95	1.00	1.00	0.95	1.00		0.95	1.00		0.95	1.00	
Satd. Flow (prot)	1789	3579	1601	1789	3577		1789	3558		1789	3237	
Flt Permitted	0.22	1.00	1.00	0.11	1.00		0.66	1.00		0.70	1.00	
Satd. Flow (perm)	407	3579	1601	214	3577		1242	3558		1324	3237	
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	277	1549	467	2	1111	4	263	76	3	2	53	93
RTOR Reduction (vph)	0	0	146	0	0	0	0	2	0	0	72	0
Lane Group Flow (vph)	277	1549	321	2	1115	0	263	77	0	2	74	0
Turn Type	Perm	NA	Perm	Perm	NA		Perm	NA		Perm	NA	
Protected Phases		4			8			2			6	
Permitted Phases	4		4	8			2			6		
Actuated Green, G (s)	68.7	68.7	68.7	68.7	68.7		22.3	22.3		22.3	22.3	
Effective Green, g (s)	68.7	68.7	68.7	68.7	68.7		22.3	22.3		22.3	22.3	
Actuated g/C Ratio	0.69	0.69	0.69	0.69	0.69		0.22	0.22		0.22	0.22	
Clearance Time (s)	4.5	4.5	4.5	4.5	4.5		4.5	4.5		4.5	4.5	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0		3.0	3.0		3.0	3.0	
Lane Grp Cap (vph)	279	2458	1099	147	2457		276	793		295	721	
v/s Ratio Prot		0.43			0.31			0.02			0.02	
v/s Ratio Perm	c0.68		0.20	0.01			c0.21			0.00		
v/c Ratio	0.99	0.63	0.29	0.01	0.45		0.95	0.10		0.01	0.10	
Uniform Delay, d1	15.4	8.6	6.1	4.9	7.1		38.3	30.9		30.2	30.9	
Progression Factor	1.00	1.00	1.00	1.00	1.00		1.00	1.00		1.00	1.00	
Incremental Delay, d2	52.1	1.2	0.7	0.2	0.6		43.3	0.2		0.0	0.3	
Delay (s)	67.5	9.9	6.8	5.1	7.7		81.7	31.1		30.3	31.2	
Level of Service	E	A	A	A	A		F	C		C	C	
Approach Delay (s/veh)		16.2			7.7			70.0			31.2	
Approach LOS		B			A			E			C	

Intersection Summary			
HCM 2000 Control Delay (s/veh)	19.1	HCM 2000 Level of Service	B
HCM 2000 Volume to Capacity ratio	0.98		
Actuated Cycle Length (s)	100.0	Sum of lost time (s)	9.0
Intersection Capacity Utilization	76.1%	ICU Level of Service	D
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Volume (veh/h)	4	1239	854	3	3	2
Future Volume (Veh/h)	4	1239	854	3	3	2
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	4	1347	928	3	3	2
Pedestrians						
Lane Width (m)						
Walking Speed (m/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (m)						
pX, platoon unblocked						
vC, conflicting volume	931				1611	466
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	931				1611	466
tC, single (s)	4.1				6.8	6.9
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	99				97	100
cM capacity (veh/h)	731				95	544

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	SB 1	SB 2
Volume Total	4	674	674	619	312	3	2
Volume Left	4	0	0	0	0	3	0
Volume Right	0	0	0	0	3	0	2
cSH	731	1700	1700	1700	1700	95	544
Volume to Capacity	0.01	0.40	0.40	0.36	0.18	0.03	0.00
Queue Length 95th (m)	0.1	0.0	0.0	0.0	0.0	0.7	0.1
Control Delay (s/veh)	10.0	0.0	0.0	0.0	0.0	44.3	11.6
Lane LOS	A					E	B
Approach Delay (s/veh)	0.0			0.0		31.2	
Approach LOS						D	

Intersection Summary			
Average Delay	0.1		
Intersection Capacity Utilization	44.2%	ICU Level of Service	A
Analysis Period (min)	15		

Appendix F

Intersection Capacity Analysis Output
Future Total Condition – Option 2

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	0	1197	186	0	772	64	0	0	20	0	0	136
Future Volume (vph)	0	1197	186	0	772	64	0	0	20	0	0	136
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Util. Factor	1.00	0.95	0.95	1.00	0.95	0.95	1.00	1.00	1.00	1.00	1.00	1.00
Frt		0.980			0.988				0.865			0.865
Flt Protected												
Satd. Flow (prot)	0	3507	0	0	3536	0	0	0	1629	0	0	1629
Flt Permitted												
Satd. Flow (perm)	0	3507	0	0	3536	0	0	0	1629	0	0	1629
Link Speed (k/h)		48			48			48			48	
Link Distance (m)		545.0			452.2			185.6			267.4	
Travel Time (s)		40.9			33.9			13.9			20.1	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	1301	202	0	839	70	0	0	22	0	0	148
Shared Lane Traffic (%)												
Lane Group Flow (vph)	0	1503	0	0	909	0	0	0	22	0	0	148
Sign Control		Free			Free			Stop			Stop	

Intersection Summary		
Area Type:	Other	
Control Type: Unsignalized		
Intersection Capacity Utilization 49.0%		ICU Level of Service A
Analysis Period (min) 15		

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	308	1425	430	20	1022	4	302	238	3	2	207	86
Future Volume (vph)	308	1425	430	20	1022	4	302	238	3	2	207	86
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Storage Length (m)	150.0		63.0	150.0		45.0	40.0		0.0	30.0		30.0
Storage Lanes	1		1	1		0	1		0	1		0
Taper Length (m)	7.6			7.6			7.6			7.6		
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	0.95	1.00	0.95	0.95	1.00	0.95	0.95
Frt			0.850		0.999			0.998			0.956	
Flt Protected	0.950			0.950			0.950			0.950		
Satd. Flow (prot)	1789	3579	1601	1789	3575	0	1789	3571	0	1789	3421	0
Flt Permitted	0.084			0.141			0.253			0.590		
Satd. Flow (perm)	158	3579	1601	266	3575	0	477	3571	0	1111	3421	0
Right Turn on Red			Yes			Yes			Yes			Yes
Satd. Flow (RTOR)			376					1			47	
Link Speed (k/h)		48			48			48			48	
Link Distance (m)		529.5			545.0			261.3			299.3	
Travel Time (s)		39.7			40.9			19.6			22.4	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	335	1549	467	22	1111	4	328	259	3	2	225	93
Shared Lane Traffic (%)												
Lane Group Flow (vph)	335	1549	467	22	1115	0	328	262	0	2	318	0
Turn Type	pm+pt	NA	Perm	Perm	NA		pm+pt	NA		Perm	NA	
Protected Phases	7	4			8		5	2			6	
Permitted Phases	4		4	8			2			6		
Detector Phase	7	4	4	8	8		5	2		6	6	
Switch Phase												
Minimum Initial (s)	5.0	5.0	5.0	5.0	5.0		5.0	5.0		5.0	5.0	
Minimum Split (s)	22.5	22.5	22.5	22.5	22.5		9.5	22.5		22.5	22.5	
Total Split (s)	25.0	70.0	70.0	45.0	45.0		21.0	46.5		25.5	25.5	
Total Split (%)	21.5%	60.1%	60.1%	38.6%	38.6%		18.0%	39.9%		21.9%	21.9%	
Yellow Time (s)	3.5	3.5	3.5	3.5	3.5		3.5	3.5		3.5	3.5	
All-Red Time (s)	1.0	1.0	1.0	1.0	1.0		1.0	1.0		1.0	1.0	
Lost Time Adjust (s)	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Total Lost Time (s)	4.5	4.5	4.5	4.5	4.5		4.5	4.5		4.5	4.5	
Lead/Lag	Lead			Lag	Lag		Lead			Lag	Lag	
Lead-Lag Optimize?	Yes			Yes	Yes		Yes			Yes	Yes	
Recall Mode	Min	Max	Max	Max	Max		None	C-Min		C-Min	C-Min	
Act Effct Green (s)	71.9	71.9	71.9	46.1	46.1		35.6	35.6		14.6	14.6	
Actuated g/C Ratio	0.62	0.62	0.62	0.40	0.40		0.31	0.31		0.13	0.13	
v/c Ratio	0.85	0.70	0.41	0.21	0.79		0.99	0.24		0.01	0.68	
Control Delay (s/veh)	49.5	17.8	3.6	33.1	37.1		83.6	30.2		42.5	48.5	
Queue Delay	0.0	0.0	0.0	0.0	0.0		0.0	0.0		0.0	0.0	
Total Delay (s/veh)	49.5	17.8	3.6	33.1	37.1		83.6	30.2		42.5	48.5	
LOS	D	B	A	C	D		F	C		D	D	
Approach Delay (s/veh)		19.5			37.1			59.9			48.4	
Approach LOS		B			D			E			D	
Queue Length 50th (m)	55.6	117.3	7.9	3.4	120.8		63.5	23.4		0.4	31.4	
Queue Length 95th (m)	#103.2	157.5	24.7	11.2	#164.2		#104.8	32.2		2.7	44.6	

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Internal Link Dist (m)		505.5			521.0			237.3			275.3	
Turn Bay Length (m)	150.0		63.0	150.0			40.0			30.0		
Base Capacity (vph)	413	2208	1131	105	1414		331	1288		200	655	
Starvation Cap Reductn	0	0	0	0	0		0	0		0	0	
Spillback Cap Reductn	0	0	0	0	0		0	0		0	0	
Storage Cap Reductn	0	0	0	0	0		0	0		0	0	
Reduced v/c Ratio	0.81	0.70	0.41	0.21	0.79		0.99	0.20		0.01	0.49	

Intersection Summary

Area Type: Other

Cycle Length: 116.5

Actuated Cycle Length: 116.5

Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green

Natural Cycle: 90

Control Type: Actuated-Coordinated

Maximum v/c Ratio: 0.99

Intersection Signal Delay (s/veh): 31.6 Intersection LOS: C

Intersection Capacity Utilization 85.6% ICU Level of Service E

Analysis Period (min) 15

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 6: Madelaine Dr