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DREAM TOWN DEVELOPMENT

TRAFFIC IMPACT ANALYSIS

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1. INTRODUCTION

HSL Developers is proposing the development of Dream Town, a multi-use site containing a restaurant and a mixed-use office/retail building. This section contains information on the purpose of the report and the study level of the proposed development.

PURPOSE

This report presents a Traffic Impact Analysis (TIA) for the proposed Dream Town development. This TIA was completed to fulfill requirements from Provo City and the Utah Department of Transportation (UDOT) for new development, and was done in accordance with UDOT guidelines to apply for an access permit (Utah Department of Transportation 2019). The TIA identifies current traffic patterns—including traffic volume counts—in the study area, and projects traffic volumes to opening day and 5 years after opening (2024 and 2029, respectively).

STUDY LEVEL

UDOT Administrative Rule R930-6-8(4) (Utah Department of Transportation 2019) sets forth general requirements for an access permit, including determining the application level. The proposed development will contain an 8,000 ft^2 high-turnover restaurant (Institute of Transportation Engineers (ITE) land use code 932) and a 32,000 ft^2 office/retail building, half of which (18,000 ft^2) will be a general office location (ITE land use code 710), and the remaining half (18,000 ft^2) will be a hardware store (ITE land use code 816). From the ITE Trip Generation Manual (Institute of Transportation Engineers 2021), these locations are expected to generate 1264 weekday trips and 166 PM peak trips (see Appendix C). These trip rates, along with the land use and respective development sizes, necessitate a Level II application.

A Level II application requires the following:

- Analysis area includes intersection of site access drives with state highways and any signalized and un-signalized intersection within access category distance of property line

- Design years are opening day and 5 years after project completion
- Traffic is identified for weekday AM and PM peak, and special peak hour as necessary
  - **N.B. This analysis only includes PM peak traffic due to the scope of the assignment**
- Data collection includes:
- Daily and turning movement counts
- Site and adjacent roadway/intersection geometrics
- Information on traffic control devices
  - Crash data
  - Trip generation following the ITE Trip Generation Manual or other ITE procedures
  - Trip distributions and assignment (existing, site, background, and future volumes in analysis area)
  - Conflict and capacity analysis
- Traffic signal impacts
- Right-of-way identified, including no- and limited-access control lines
- Includes safe operational design needs and concerns with accompanying mitigation measures

2. PROPOSED DEVELOPMENT

This section describes the plans for the development, including site location, land use, zoning information, and the the site plan itself.

SITE DESCRIPTION

The proposed Dream Town development is located in southern Provo, in Utah. The site is located on the west side of University Ave. (US-189), between 1200 South and Towne Centre Dr. This location is just east of the Provo Towne Centre mall, which is a large retail and dining center and a large generator of trips. The location is also near (to the north of) the intersection of I-15 and University Ave. Figure 2.1 shows a map of the site area. Four intersections are included in the analysis:

- 1200 South / Towne Centre Blvd. (all-way–stop-controlled (AWSC))
- 1200 South / University Ave. (signalized)
- Towne Centre Dr. / Towne Centre Blvd. (signalized)
- Towne Centre Dr. / University Ave. (signalized)

LAND USE AND ZONING INFORMATION

The proposed Dream Town development has two development pads, with one pad containing a sit-down restaurant and the other pad containing a mixed-use office/retail location. The retail location is intended to be a hardware/paint store. Table 1 shows these land uses along with their ITE land use codes and respective square footage.

The site area is zoned as SC3, which is a Regional Shopping Center. The primary use of this land is for commercial and service uses to serve needs of people living in an entire region. It is located close to freeways & major arterials for easy access (Provo City Utah 2022a chap. 14.20). Figure 2.2 provides a section of the Provo zoning map (Provo City Utah 2022b) with the site location marked.

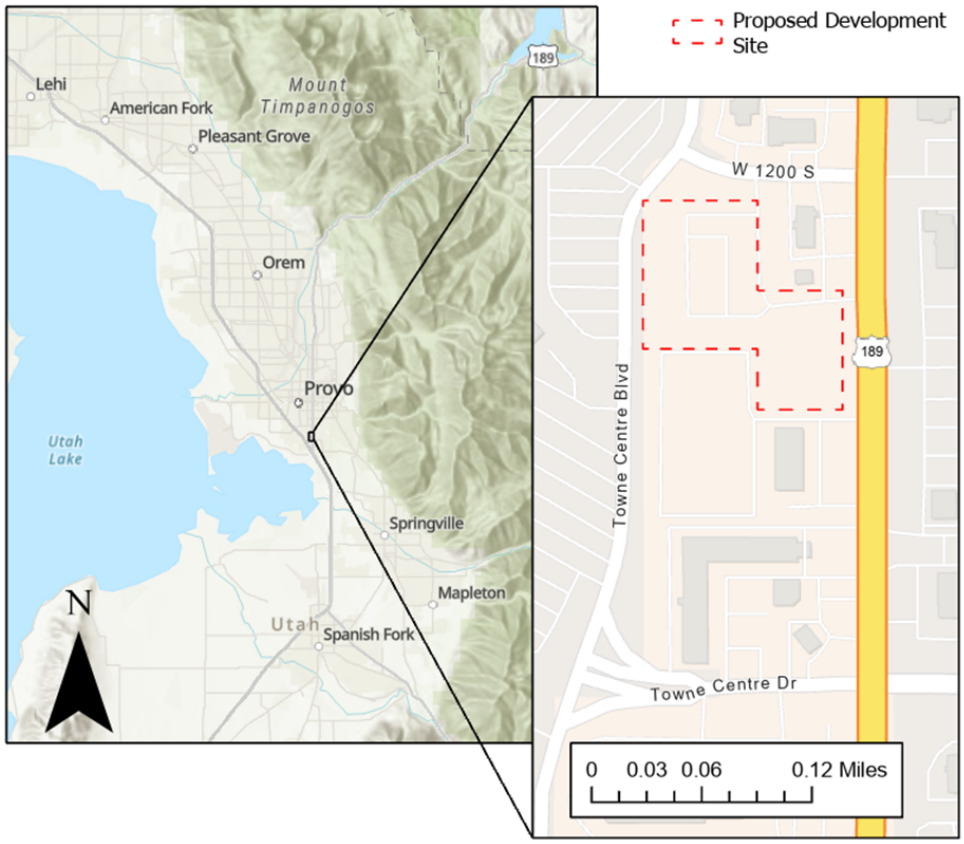

Figure 2.1: Map of the site location.

Table 1: *Proposed Land Use and Sizes*

Development Pad	Proposed Land Use	ITE Land Use Code	Area (ft^2)
A	Sit-Down Restaurant	932	8,000
B	General Office Building	710	18,000
B	Hardware/Paint Store	816	18,000

Site Plan

Figure 2.3 shows a site plan for the proposed development. Note that Pad C is not included in this analysis, as it has already been developed. This site map is still preliminary, and will be updated in a future draft of this report.

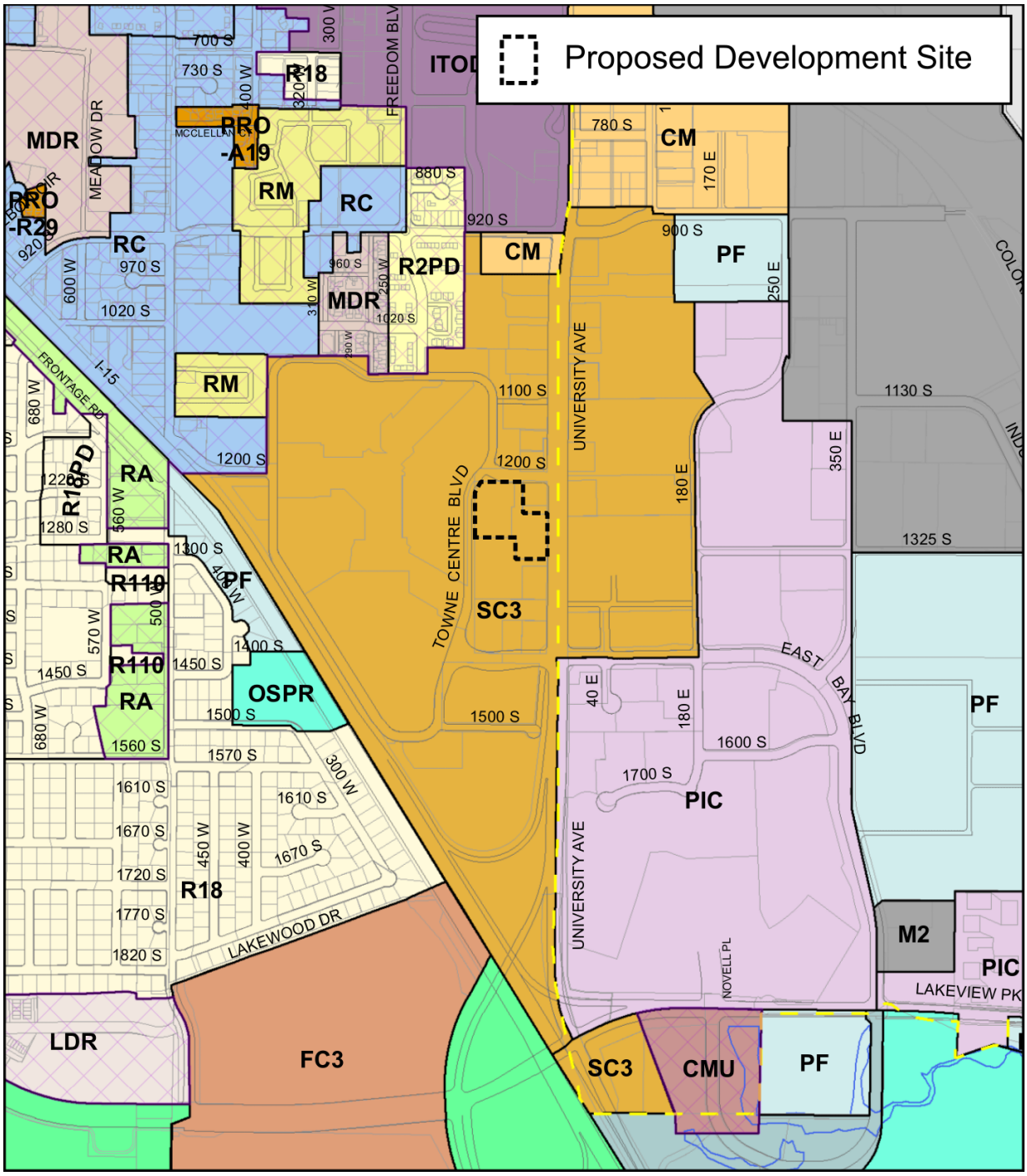

Figure 2.2: Provo zoning map, cropped to vicinity of the site.

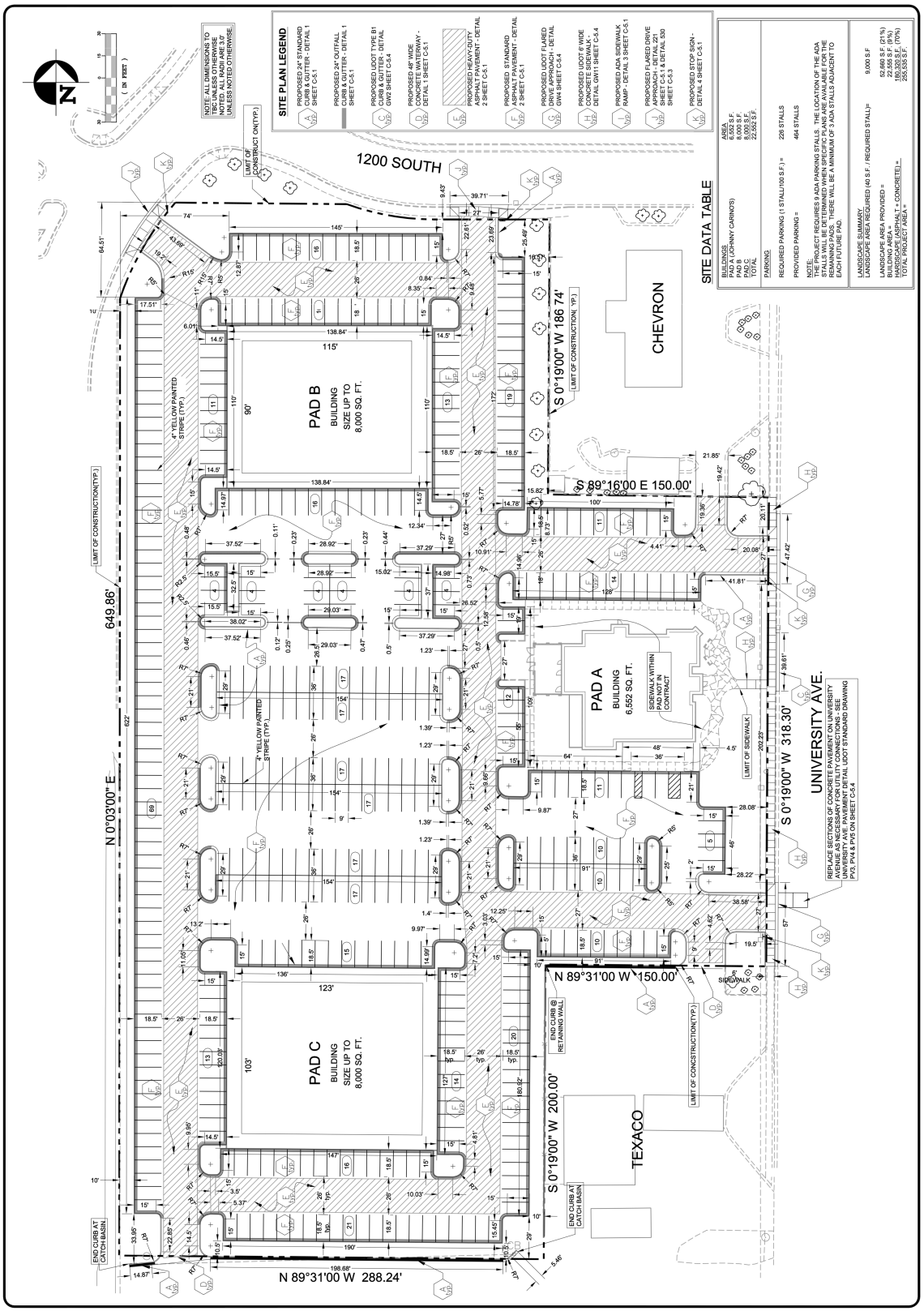

Figure 2.3: Site plan for proposed Dream Town development.

3. STUDY AREA CONDITIONS

The study area for this analysis includes the streets adjacent to the site as well as their intersections. These streets are 1200 South on the north, University Ave. (US-189) on the east, Towne Centre Blvd. on the west, and Towne Centre Dr. on the south. Note that there is existing development between Dream Town and Towne Centre Dr. All of the intersections in the TIA are signalized with the exception of the Town Centre Blvd. / 1200 South intersection, which is stop-controlled. This is shown in Figure 3.1.

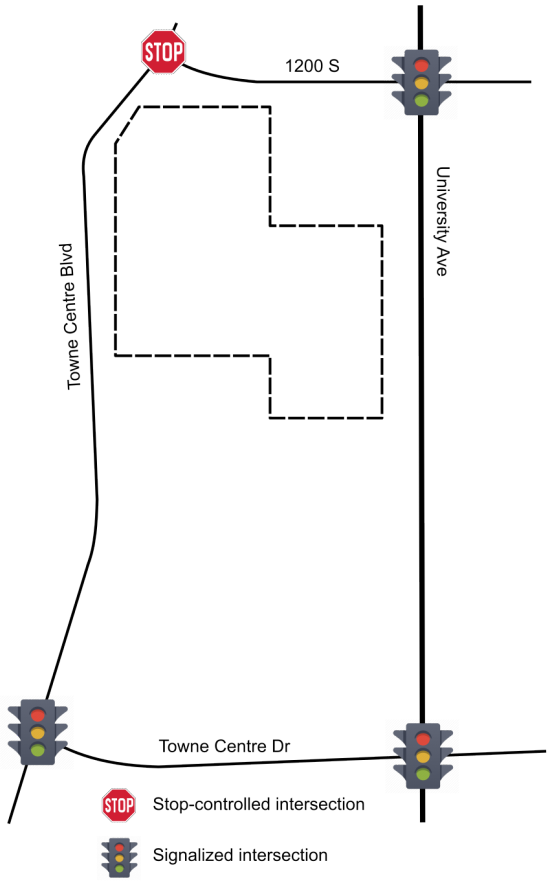

Figure 3.1: Map of basic intersection characteristics in TIA area.

STREET CONDITIONS

Detailed information on each street in the study area is provided in the following sections. This information is also summarized in Table 2.

Table 2: *Summary of Adjacent Street Configuration*

Road Name	Direction	NB/EB Lanes	SB/WB Lanes	Speed Limit (mph)	Meridian/ TWLTL[1]
1200 South	E/W	2	2	---	Median
University Ave. (US-189)	N/S	3	3	35	TWLTL
Towne Centre Blvd.	N/S	1	2	25	TWLTL
Towne Centre Dr.	E/W	2	2	30	Median

[1]Two-way--left-turn lane

1200 South

1200 South is an east-west (E/W) local road which, at least in the vicinity of the study area, exists only to offer a connection between Towne Centre Bvld. and University Ave. It runs about 350 feet along the north edge of the proposed Dream Town, and has 3 eastbound (EB) lanes and 2 westbound (WB) lanes. The 2 WB lanes offer a left- and right-turning movement, respectively (onto Towne Centre Blvd.), and there is no through movement since 1200 South and Towne Centre Bvld. form a T-intersection. The 3 EB lanes offer a left, through, and right-turn movement, either onto University Ave. for the turning movements or into a parking lot for the through movement. There is no posted speed limit, and a raised median prevents left turns onto or off of this road.

University Avenue (US-189)

University Ave. is also designated as US-189, and is a minor arterial running north-south (N/S) through Provo (Utah Department of Transportation 2023a). It is assigned an access category of 5, which is described as "Regional priority—urban importance" (Utah Department of Transportation 2023b). University Ave. connects with I-15 roughly 3000 feet south of the study area and has connections to many commercial developments on either side of the road. There are 3 through lanes in both directions in the study area, with 2

additional left- and 1 additional right-turn lanes/bays in both directions at the Towne Centre Dr. intersection and 1 additional left- and right-turn lane/bay each in both directions at the 1200 South intersection. The posted speed limit is 35 mph, and there is a two-way–left-turn lane (TWLTL) on this road for the length of the study area. There is also a 10-foot shoulder on the west side of the road, and there are sidewalks on both sides, though the sidewalk on the east is separated from the road by a grassy curb strip, and the sidewalk on the west is not.

Towne Centre Boulevard

Towne Centre Blvd. is a local street that circumnavigates the Provo Towne Centre mall. Within the study area this road runs N/S, and has 1 northbound (NB) and 2 southbound (SB) through lanes. A TWLTL runs the length of this road, though it is broken up by a bus stop located in the center of the road near the southern end of Dream Town. This bus stop serves the UVX/830X route, which is a BRT route running from the Towne Centre mall to Utah Valley University (Utah Department of Transportation n.d.). The TWLTL becomes two SB left turn bays at the Towne Centre Dr. intersection, and there is a NB free-flow right-turn bay at the same. This road has a posted speed limit of 25 mph.

Towne Centre Drive

Towne Centre Dr. is an E/W road that runs between Towne Centre Blvd. and University Ave., and becomes East Bay Blvd. east of University Ave. There is a posted speed limit of 30 mph, and there is a raised median between Towne Centre Blvd. and University Ave., though the median breaks about midway, allowing a WB left turn to access the development to the south. There are 2 left- and 1 right-turn bays at the University Ave. intersection in both directions, and 2 left-turn lanes and a right-turn free-flow lane at the Towne Centre Blvd. intersection.

Additional Information

There are several bus stops in the study area, as shown in Figure 3.2. Towne Centre Blvd. is elevated relative to Dream Town, but there is a staircase next to the bus stop that allows for pedestrian access.

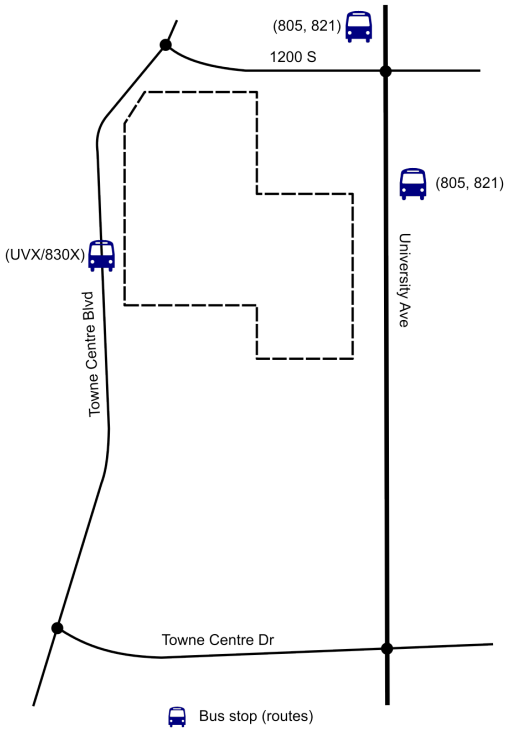

Figure 3.2: Map of bus stops on the studied streets.

ADJACENT LAND USES

As can be seen in Figure 2.2, much of the nearby land is in the same zoning category as Dream Town (SC3). This is borne out by the actual land use: there are several suburban commercial sites in this area. The most notable of these is the Provo Towne Centre mall to the east, but other locations include a Sam's Club, several fast-food and sit-down restaurants, and multiple small hotels and gas stations.

SITE ACCESSIBILITY

University Ave. provides direct access to Dream Town from I-15, as well as from downtown Provo. This is the most major road with direct access to the site, but several other roads provide access as well. Towne Centre Blvd. has connections to a neighborhood northwest of the site, and East Bay Blvd. provides access to University Ave. near the site from southeast Provo, and has a connection to Springville via Lakeview Pkwy. and I-89.

4. ANALYSIS OF EXISTING CONDITIONS

The study area contains four intersections: 1200 South / Towne Centre Blvd., a 3-way stop-controlled intersection; 1200 South / University Ave., a 4-way signalized intersection; Towne Centre Dr. / Towne Centre Blvd., a 3-way signalized intersection; and Towne Centre Dr. / University Ave., a 4-way signalized intersection (see Figure 3.1).

Much of the information regarding lane striping and turning movements for these intersections is given in Table 2. It is additionally worth noting that the east leg of the 1200 South / University Ave. intersection is a parking lot access, and the WB lane has a designated left-turn lane and a shared right/through lane. For reference, a lane configuration diagram of these intersections is provided in Figure 4.1, and timings for the signalized intersections are provided in Appendix B.

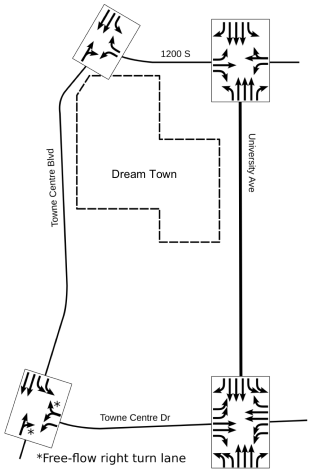

Figure 4.1: Intersection lane diagram of the study area.

TRAFFIC VOLUMES

Traffic turning volume counts were done at the Towne Centre Dr. / University Ave. and Towne Centre Dr. / Towne Center Blvd. intersections from 4:15–6:00 PM on January 24 (Tue)

and 25 (Wed), 2023, respectively. Scheduling conflicts and limited personnel necessitated that the counts be performed for only two intersections and on different days. However, for the purposes of this report this is not a significant limitation, due to the limited scope of the assignment. The two days are considered similar enough, and volumes were estimated for the other two intersections based on these counts and historical data.

From these traffic volumes, 4:30–5:30 PM was determined to be the peak hour, with a peak hour factor (PHF) of 0.92. The volumes for this peak hour are presented in Figure 4.2.

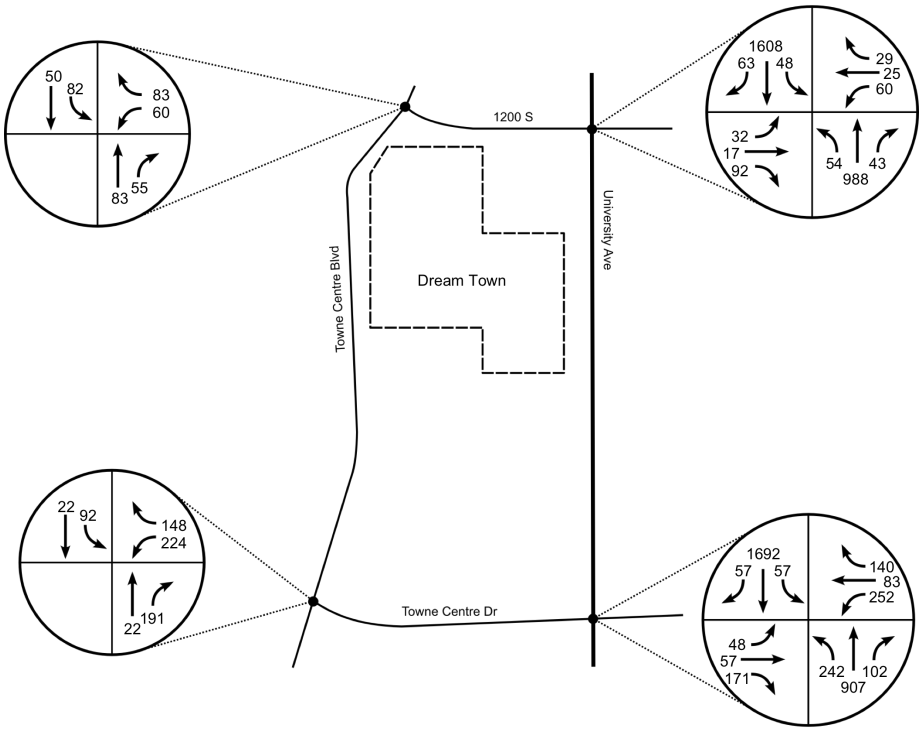

Figure 4.2: Peak hour turning volumes on the studied intersections.

LEVEL OF SERVICE

A level of service (LOS) measure was used to determine the traffic performance of each intersection (and each movement within each intersection). This LOS is a measurement of average control delay per vehicle, and bins the amount of delay into groups designated by

the letters A–F. These designations differ between signalized and unsignalized intersections, and are provided in the Highway Capacity Manual (HCM) (National Academies of Sciences, Engineering, and Medicine 2022). Table 3 summarizes these criteria. This analysis assumes a LOS of D or better represents acceptable conditions.

Table 3: *LOS Criteria for Intersections*

LOS Designation	Average Control Delay (sec/veh)	
	Signalized	Unsignalized
A	≤ 10	≤ 10
B	10–20	10–15
C	20–35	15–25
D	35–55	25–35
E	55–80	35–50
F	> 80	> 50

The existing level of service was determined based on current traffic volumes Section 4.1 and signal timings (obtained from UDOT), using the Synchro software package (Trafficware 2019). Synchro performs a LOS analysis using these and other inputs, including roadway and lane configuration. Figure 4.3 shows the average control delay for each movement and intersection, and Figure 4.4 shows the LOS of the same. The full results of the Synchro analysis are available in Appendix A.

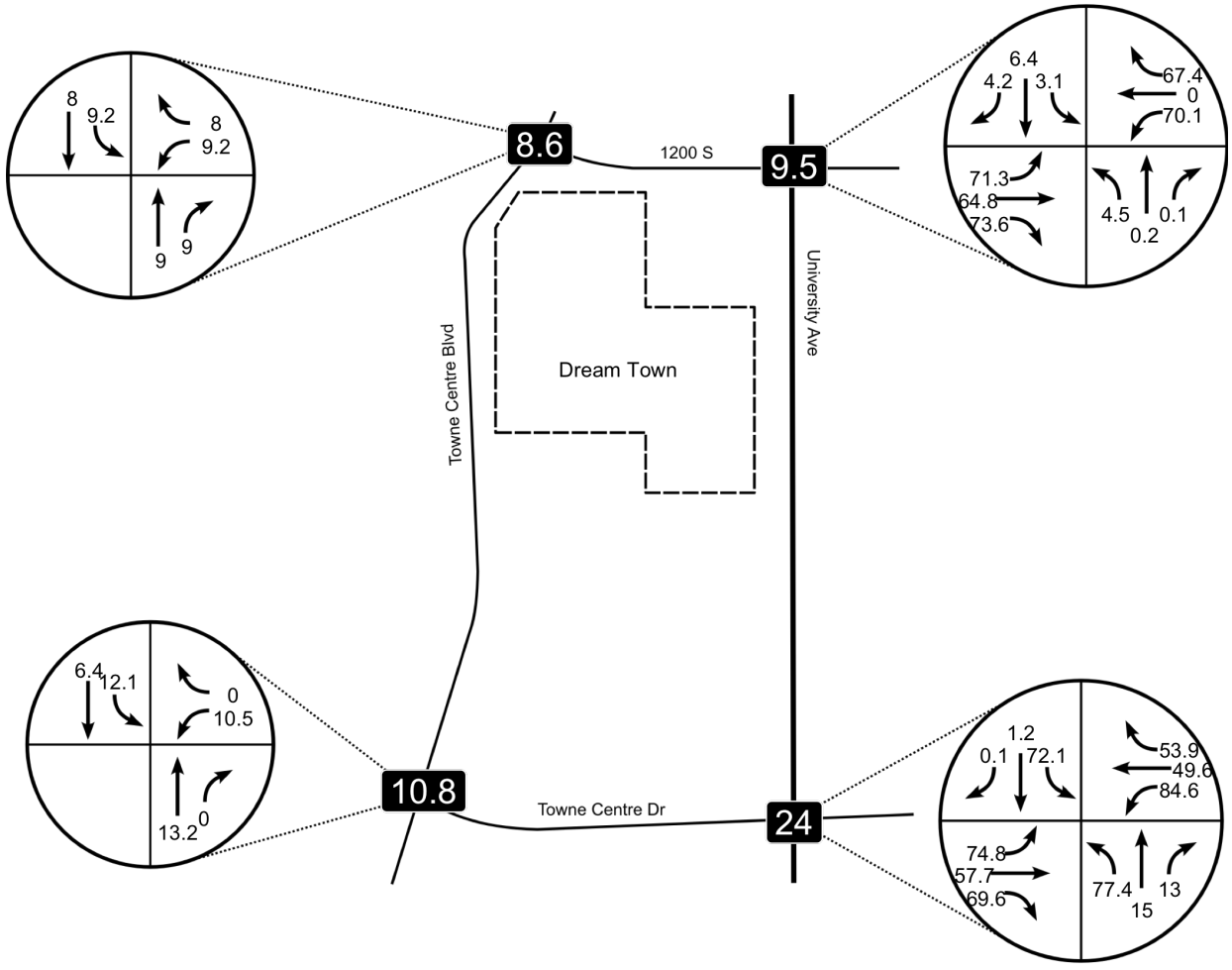

Figure 4.3: Results of existing Synchro LOS analysis for each movement and intersection studied: control delay values.

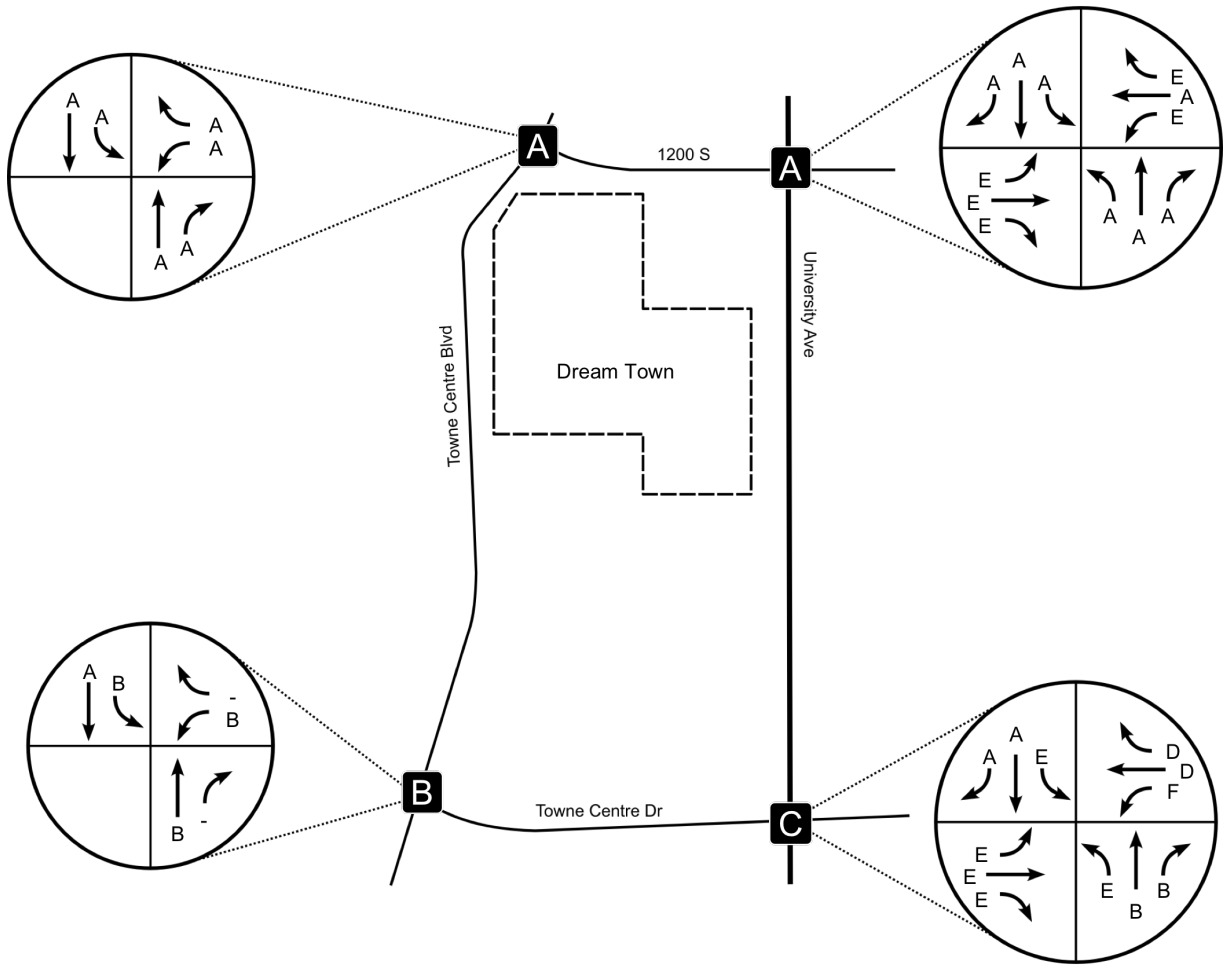

Figure 4.4: Results of existing Synchro LOS analysis for each movement and intersection studied: level of service.

It is unsurprising that the Towne Centre Blvd. intersections perform so well, since the traffic volumes at these intersections are relatively low (see Figure 4.2). It also makes sense that the through and right-turn movements on University Ave. perform well, as this is a major arterial and these movements have relatively long phase lengths (see Appendix B). The EB and WB movements at the University Ave. intersections, however, generally perform much worse. This is for the same reason: the phase lengths on these movements are significantly

shorter compared to the conflicting phase lengths. However, these movements do not have an enormous impact on the overall intersection delay(s), as the volumes are much lower than those *on* University Ave.

The overall LOS is generally good, though there are several movements with worse than LOS D (generally acceptable conditions) performance. However, these are relatively minor movements, and the intersections themselves perform much better, so this is not too much cause for concern. It is possible that something as simple as a change in signal timings may do much to improve these under-performing movements. However, further analysis into the coordination of signals on University Ave. is warranted before implementing any signal timing changes.

TRANSPORTATION SAFETY

As part of the analysis for this proposed development, a crash analysis was performed for University Ave. near the site. Recent crash data (obtained from \acr{UDOT} for 2019--2022) was used, and crashes between mileposts 0.5 and 1.0 were included in the analysis. This includes the intersections of University Ave. with both Towne Center Dr. and 1200 South, as well as the roadway segments between and either side of the intersections. Crashes occurring within 0.05 miles of an intersection are considered as "belonging" to that intersection, and all other crashes are counted on their respective roadway segments.

Equation 4.1 gives the crash rate of an intersection, and Equation 4.2 gives the crash rate of a segment, where AADT is average annual daily traffic, MEV is million entering vehicles, and MVMT is million vehicle-miles traveled. The AADT was provided by UDOT for this stretch of University Ave., and is given as 30550 vehicles per day.

$$Crash\ Rate_{Intersection} = \frac{1000000 \times crashes}{365.25 \times years \times AADT}\ \text{crashes per MEV} \qquad (4.1)$$

$$Crash\ Rate_{Segment} = \frac{1000000 \times crashes}{365.25 \times years \times AADT \times length}\ \text{crashes per MVMT} \qquad (4.2)$$

Table 4 provides the two intersections and their crash rates, and Table 5 provides the crash rates on each segment of the roadway (as calculated by Equations 4.1 and 4.2).

Table 4: *University Ave. Crash Rates (Intersections)*

Intersection	Milepost	Number of Crashes	Crash Rate (per MEV)
Towne Center Blvd.	0.66	11	0.33
1200 South	0.82	24	0.72

Table 5: *University Ave. Crash Rates (Segments)*

Intersection	Milepost Start	Milepost End	Segment Length	Number of Crashes	Crash Rate (per MVMT)
–Towne Center Blvd.	0.50	0.61	0.11	21	5.70
Towne Center Blvd.–1200 South	0.71	0.77	0.06	6	2.99
1200 South–	0.87	1.00	0.13	19	4.37

For the purposes of this assignment, the critical crash rate for intersection analysis is assumed to be 2 crashes per MEV, and the critical crash rate for the roadway segment analysis is assumed to be 8 crashes per MVMT. None of the crash rates approach these values, and so this roadway segment is deemed to be safe. However, it is also important to examine the severity of the crashes. Table 6 provides the UDOT crash severity scale, and Table 7 gives the number and proportion of crashes at each level. From Table 7 it is clear that the vast majority of crashes are Property damage only (PDO) crashes, with around 10% with confirmed injury and none fatal. This gives further evidence for the safety of the roadway corridor.

Table 6: *Crash Severity Levels (UDOT)*

Crash Severity Level	Description
5	Fatal
4	Incapacitating Injury
3	Non-incapacitating Injury
2	Possible Injury
1	Property Damage Only

Table 7: *Crashes by Severity*

Crash Severity Level	Description	Number of Crashes	Proportion of Crashes
5	Fatal	0	0.000
4	Incapacitating Injury	2	0.025
3	Non-incapacitating Injury	7	0.086
2	Possible Injury	14	0.173
1	Property Damage Only	58	0.716

5. PROJECTED TRAFFIC

Part of this analysis involves determining the number of trips that are expected to visit the site. This includes trips made with the express purpose of visiting a location in the development, as well as "pass-by" trips where the ultimate destination is somewhere else. Each of the separate land uses has distinct baseline trip rates, as well as different parameters for adjusting these rates to obtain final projected trips.

TRIP GENERATION

ITE provides several resources for estimating trip generation of new developments. The primary of these is the Trip Generation Manual (Institute of Transportation Engineers 2021), but these baseline trip generation rates do not account for trips internal to the site, or for "pass-by" trips. The following section discusses the baseline ITE-provided trip rates along with the trip reductions due to these factors. Table 11 provides the trip rates after the reductions.

Base Trip Generation

The initial estimates for trip generation are given from the ITE Trip Generation Manual. For this report, only an analysis of the weekday PM peak period is performed, due to the scope of the assignment. Table 8 gives the baseline trip generation as provided by the Manual. These initial estimates seem reasonable, noting that these are the trips in the peak hour. See Appendix C for the Manual references used in these calculations.

Table 8: *ITE Trip Generation (Baseline)*

Description	Pad	ITE Land Use Code	Variable Value	Variable Unit	Trip Generation Equation	Entering Trips	Exiting Trips	Total Trips
Sit-Down Restaurant	A	932	8	1000 ft²	$T = 9.05 \times x$	44	28	72
General Office Building	B	710	18	1000 ft²	$\ln(T) = 0.83 \times \ln(x) + 1.29$	7	33	40
Hardware/ Paint Store	B	816	18	1000 ft²	$T = 2.98 \times x$	25	29	54

Multi-use Reduction

Since these land uses share a development site, a multi-use trip reduction may be applied. This accounts for trips that are made from one location in the development site to another, and thus do not use the adjacent roadways. Information on the multi-use capture rates was obtained from the ITE Trip Generation Handbook (Institute of Transportation Engineers 2014), and the relevant information is summarized in Table 9. The reduced trip rates are given in Table 10, and Appendix C contains more detailed calculations.

Table 9: ITE Multi-use Capture Rates

Trip Movement	Capture Rate
From Office to Retail	23%
From Retail to Office	3%
To Office from Retail	31%
To Retail from Office	2%

Table 10: Trips After Multi-use Reduction

Description	ITE Land Use Code	Entering Trips	Exiting Trips	Total Trips
Sit-Down Restaurant	932	44	28	72
General Office Building	710	6	32	38
Hardware/Paint Store	816	24	28	52

Pass-by Reduction

The ITE Trip Generation Manual also provides information on pass-by trip reductions. These indicate trips that otherwise would have passed the development, and are "stopping by" the site rather than making a dedicated "new" trip to the site. Two of the proposed land uses—the restaurant and the hardware store—are given pass-by reduction rates of 43% and 26%, respectively. The office land use has no pass-by reduction, which is logical as most office trips would be for employment and not leisure. Table 11 presents the new calculated trips after these reductions.

Table 11: Trips After Pass-by Reduction

Description	ITE Land Use Code	Pass-by Reduction Rate	Entering Trips	Exiting Trips	Total Trips
Sit-Down Restaurant	932	0.43	25	16	41
General Office Building	710	0.00	6	32	38
Hardware/Paint Store	816	0.26	18	21	39

MODAL SPLIT

This analysis will be done in full in the future, but a mode split reduction of 1% is assumed. This will be used to calculate trip numbers for trip assignment on new trips, but not pass-by trips as those are assumed (almost by definition) to all be vehicular.

TRIP DISTRIBUTION

This section presents two methods for determining the distribution of trips to Dream Town. The first is the analogy method, which assumes that the trip distribution will largely match the distribution of the existing background traffic. The second is the gravity method, which uses a gravity model to estimate trips based on population and travel time. These two analyses are presented and compared below.

Analogy Method

The analogy method uses existing traffic volumes to determine the distribution of site trips. Figure 4.2 shows the volumes used. Each of the intersection movements were assigned a direction and were designated as either an entering or exiting movement. Several movements, however, enter an intersection in the study area but leave on a road that doesn't ultimately pass the site, or are "internal" movements that do not show a clear direction away from the site. These movements are disregarded in this part of the analysis. Additionally, the eastern side of the University Ave. / 1200 South intersection leads directly to a parking lot, and these movements are disregarded as well. Figure 5.1 shows a map of each movement's classification.

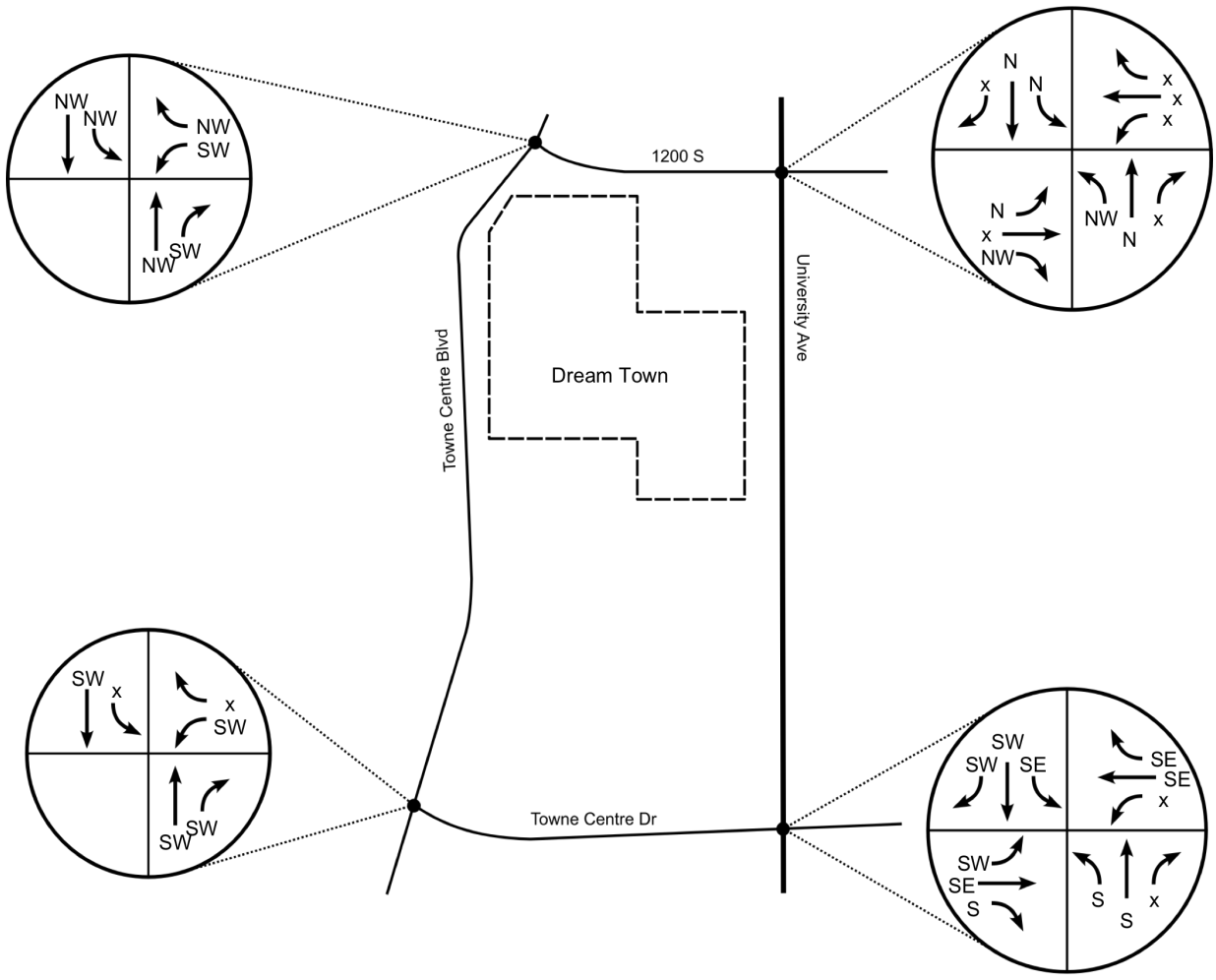

Figure 5.1: Assumed access directions for the analogy method analysis.

Based on these classifications and the movement volumes from Figure 4.2, the proportion of trips from each site access point is determined. Entering and exiting trips from each direction are averaged, since the primary trips this analysis applies to are assumed to return from where they came. These results are shown in Table 12.

Table 12: *Trip Distribution Based on the Analogy Method*

Direction	Base Traffic Volume	Proportion of Trips
North	2,676	37%
Northeast	0	0%
Northwest	444	6%
South	3,012	42%
Southeast	337	5%
Southwest	679	9%

Note however that these results only apply to "new" trips, i.e. trips made with the express purpose of visiting the site. "Pass-by" trips, or trips made as a stop-in en route to another destination, are determined based on the directional distribution of traffic on University Ave. alone. The movements considered here are the six that turn on to or off of University Ave. at each of its two intersections (including in this case the eastern side of the University Ave. / 1200 South intersection), and are categorized by travel direction (NB or SB). The NB and SB volumes differ slightly between entering and exiting due to the inherent inaccuracies of traffic counts, but are similar enough to not be a concern. The two volumes in each direction are averaged for this analysis. The pass-by distribution is given in Table 13.

Table 13: *Distribution of Pass-By Trips*

Direction	Volume (mean)	Proportion of Trips
Northbound	1,090	38%
Southbound	1,783	62%

Gravity Method

The gravity method uses a modified gravity model to determine trip distribution. This model estimates the proportion of trips from each Transportation Analysis Zone (TAZ) in the vicinity of the study area based on the population of each TAZ and the travel time from each TAZ to the site. The model is of the form

$$PT_i = \frac{A_i \times \frac{1}{tt_i^2}}{\sum A_i \times \frac{1}{tt_i^2}},$$

where PT_i is the proportion of trips from TAZ i, A_i is the population of TAZ i, and tt_i is the travel time to the site from TAZ i. Note that this differs from a typical gravity model in two main ways: we are calculating a *proportion* of trips since all trips are going to the same TAZ, and population, which is typically used in determining TAZ trip productions, is instead used as a trip attraction value.

It is not reasonable to analyze every TAZ in Utah, and so a maximum travel time of 20 minutes is enforced. Based on the land uses of the site and the proximity of similar/competing land uses, this threshold provides a good estimate of the travel behavior. Though there are other residential, retail, and office land uses within a smaller travel window than this, the retail is specific, and the office will attract its employees with little regard for other nearby offices.

For reference, Figure 5.2 shows travel time contours from the site for 10-, 15-, and 20-minute thresholds to each TAZ. Note that there are several TAZs "missing": these had a travel time of 0 and so were removed from the analysis.

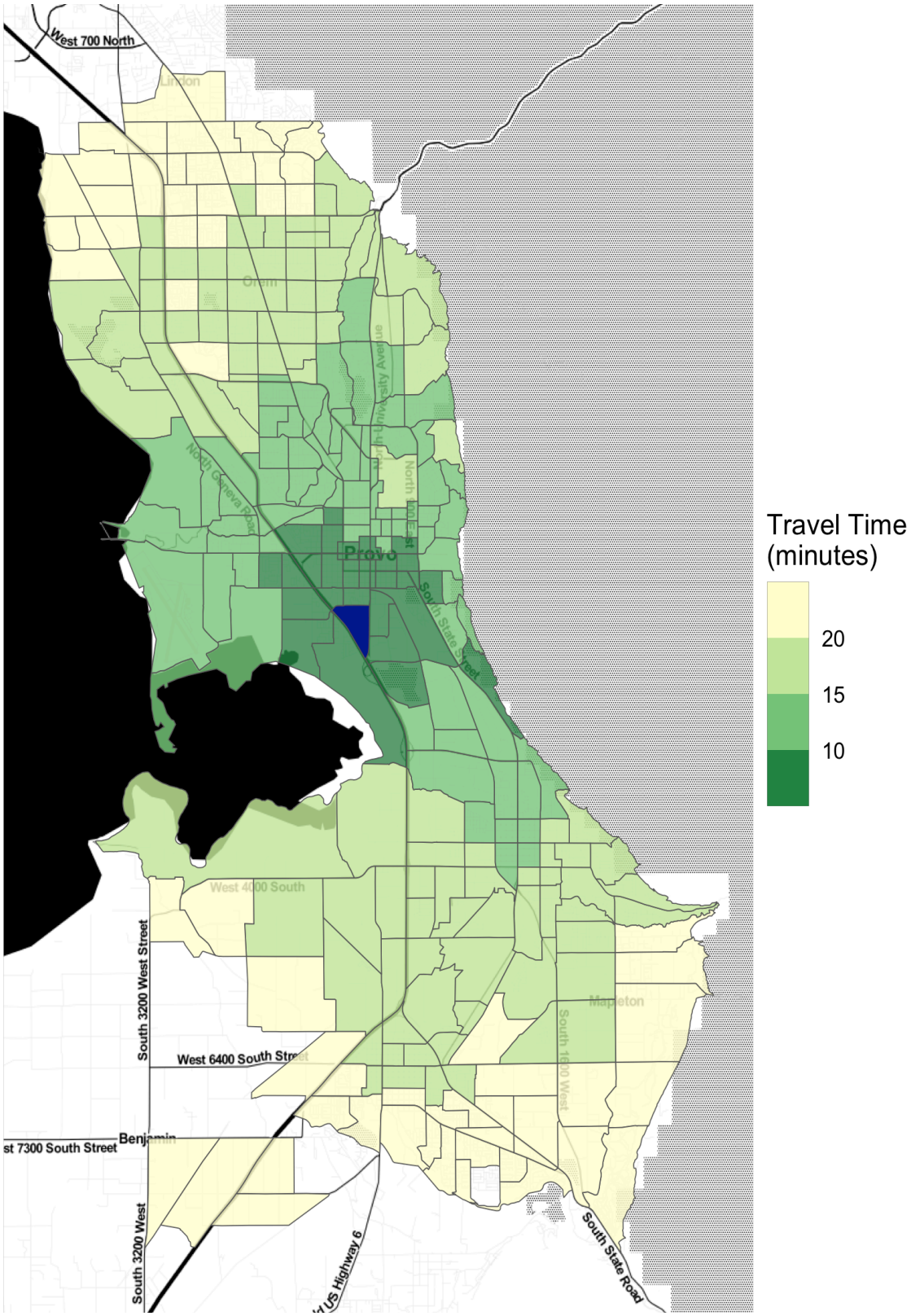

Figure 5.2: Isochrone map of travel times to the study site (in dark blue).

Figure 5.3 shows the site access direction for each TAZ. Several assumptions were made in this regard. The most significant of these is who accesses the site via I-15 north of Provo. This analysis assumes that west of Orem State Street (500W) and north of University Parkway, I-15 would be used, and east of Orem State Street and/or south of University Parkway, a combination of University Parkway, University Avenue, and Orem Center Street would be used to access the site from the north. North of Orem 800 N is assumed to have access via I-15 as well. Note that I-15 has an interchange directly south of the site on University Avenue, so it is assumed everyone using I-15 will access the site from the south. There are a handful of TAZs on the west of the site that are assumed to have western access due to I-15 under- and overpasses, but no access from the east is assumed due to the presence of the railroad.

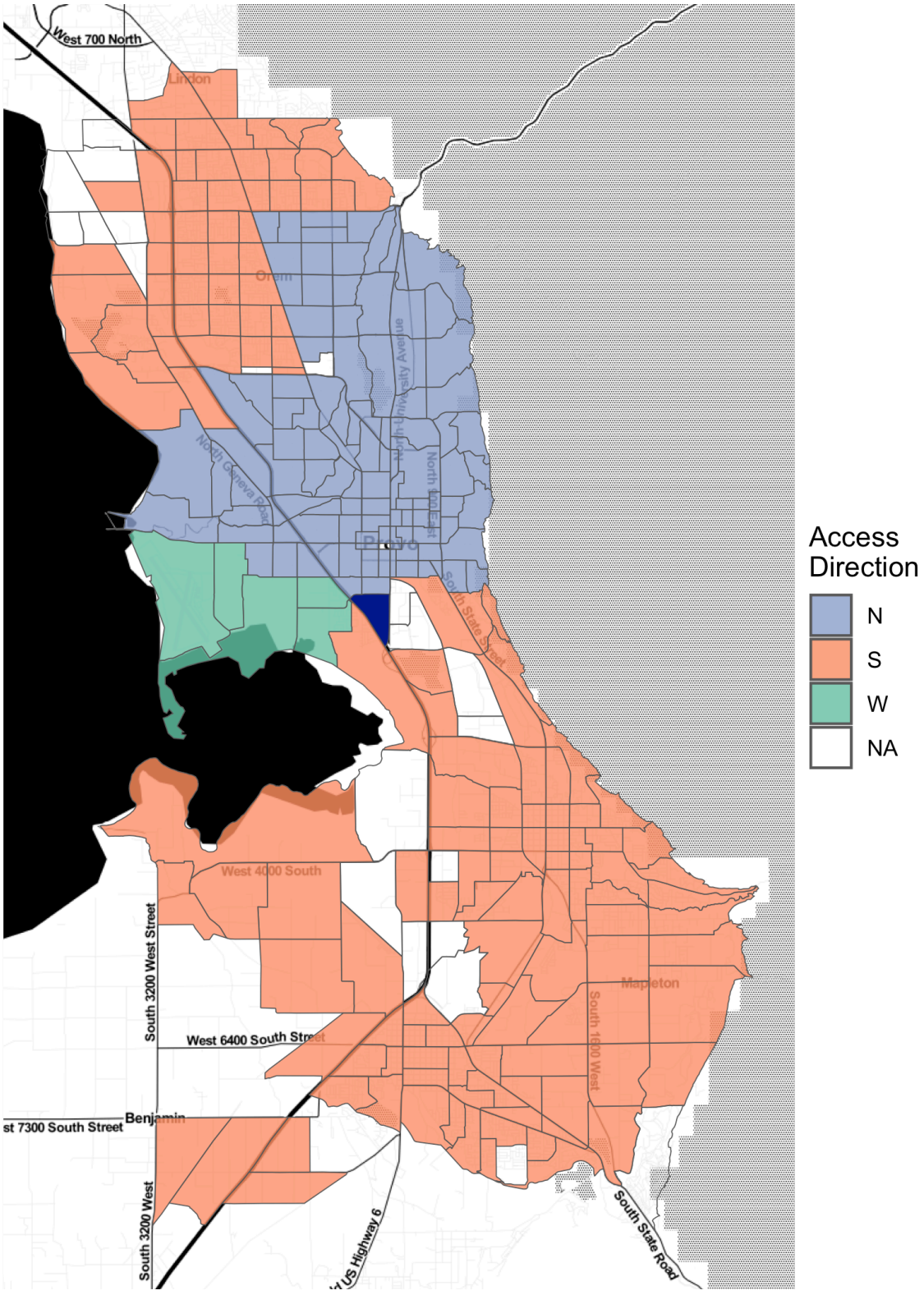

Figure 5.3: Access to site (in dark blue) by direction.

Combining this model with the information about access direction gives a proportion of trips from each direction. Table 14 presents a summary of this information. Note that this distribution is only for "new" trips; the distribution of pass-by trips is the same as in the analogy method (see Table 13).

Table 14: *Proportion of Trips by Access Direction (Gravity Model)*

Access Direction	Number of TAZs	Proportion of Trips
N	106	69.6%
S	85	27.5%
W	5	2.9%

Method Comparison

The first point of comparison between the two methods is that the analogy method includes 6 directional distributions, and the gravity model only includes 3. This is due to the assumptions made in the gravity method analysis, one of which is the difficulty of assigning TAZs so granularly. Especially since Towne Centre Blvd. and University Ave. are so close to each other, it is hard to determine for example which of these roads a trip from the north would utilize. This is much easier when analyzing intersection movements, and so the analogy method includes this distinction. For comparison's sake, however, the analogy method's results are further summarized into the 3 directions used in the gravity method analysis (northwest and southwest are classified as "west", and southeast is classified as "south", with the remaining movements unchanged). This comparison is shown in Table 15.

Table 15: *Comparison of Trip Distribution Methods*

Direction	Analogy Method	Gravity Method
N	37.4%	69.6%
S	46.9%	27.5%
W	15.7%	2.9%

Due to the higher granularity and more reasonable distribution of the analogy method, this analysis is preferred. The gravity method seems to under-predict trips from the south, especially since these will be coming from I-15 and are likely to be many. Figure 5.4 shows the final trip distribution for both new and pass-by trips that will be used. This is the same result as given by the analogy method (Tables and 12 and 13).

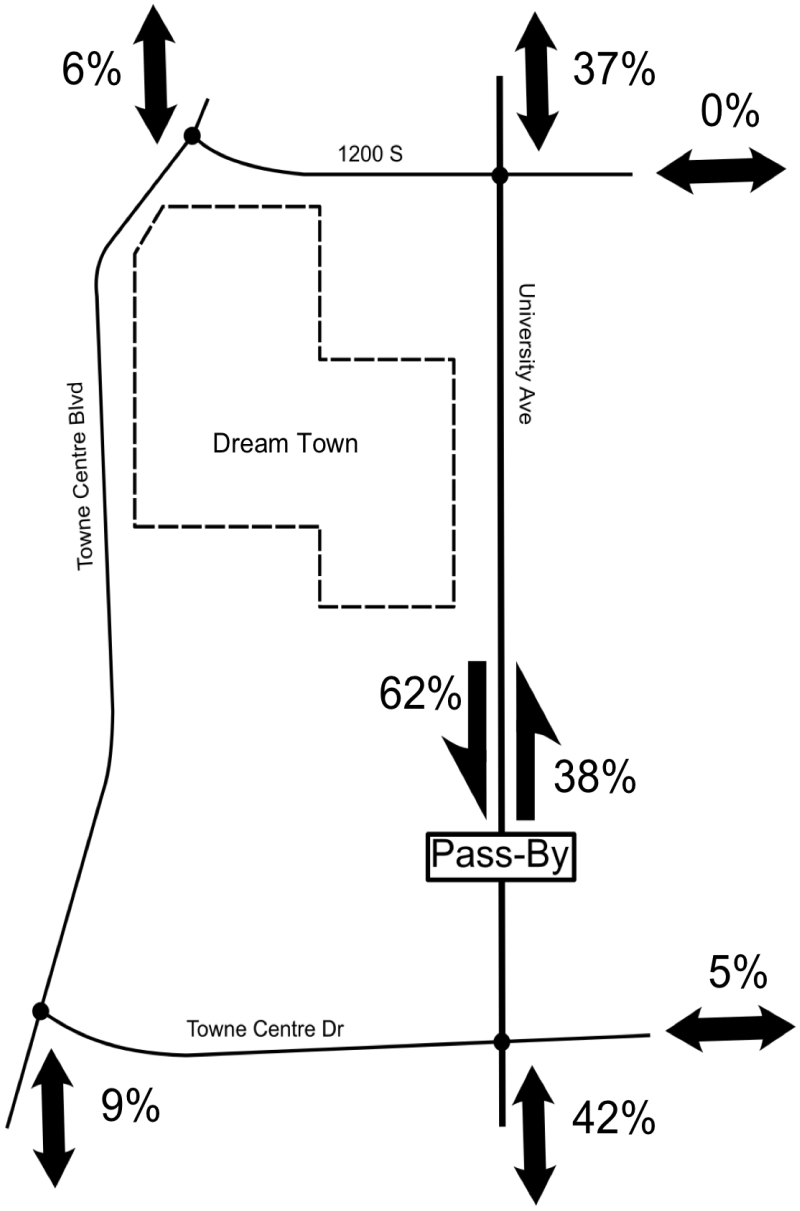

Figure 5.4: Trip distribution as given by the analogy method.

TRIP ASSIGNMENT

Non-site Traffic

The background or non-site traffic was originally determined with traffic counts and estimations for the uncounted movements Section 4.1. In addition to this, opening-day and 5-year projections are needed to assess the impact of the development over time. Note that opening day is planned for Spring of 2024, and so the 5-year horizon date is 2029.

The growth was determined by using an average rate of the form $vol = vol_0 \times (1 + rate)^{year}$, where vol is the projected volume, vol_0 is the original volume, $rate$ is the growth rate, and $year$ is the number of years since the original volume was measured. This equation applies to all movements equally. The growth in this case is modeled as exponential; this is a reasonable assumption considering the relatively short horizons, but would need to be scrutinized further for longer horizons.

In the pre-application meeting, a growth rate of 2% was determined, and so this rate is used here. Figure 5.5 shows the projected volumes at opening, and Figure 5.6 shows the projected volumes 5 years after opening.

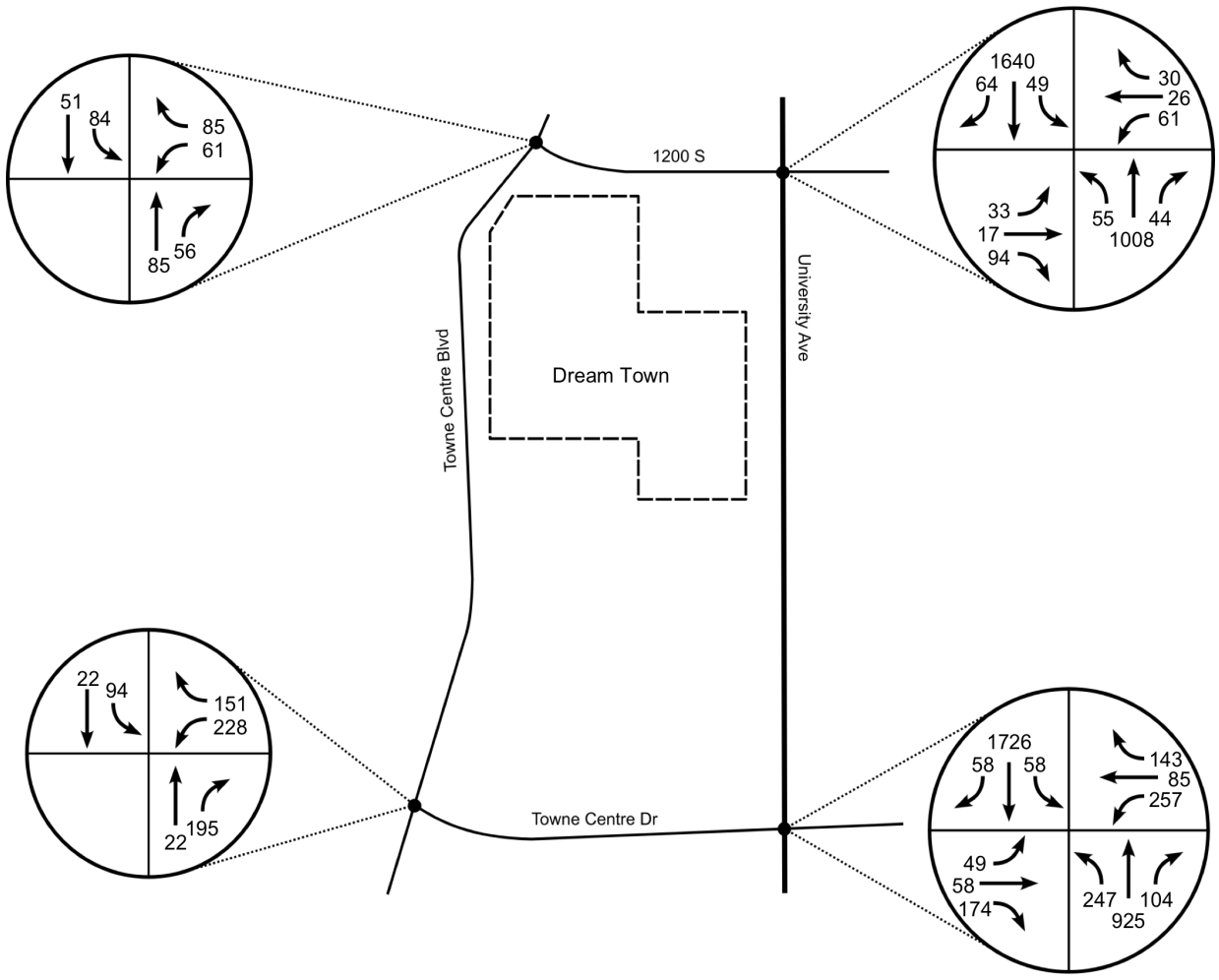

Figure 5.5: Projected background traffic at opening (1 year).

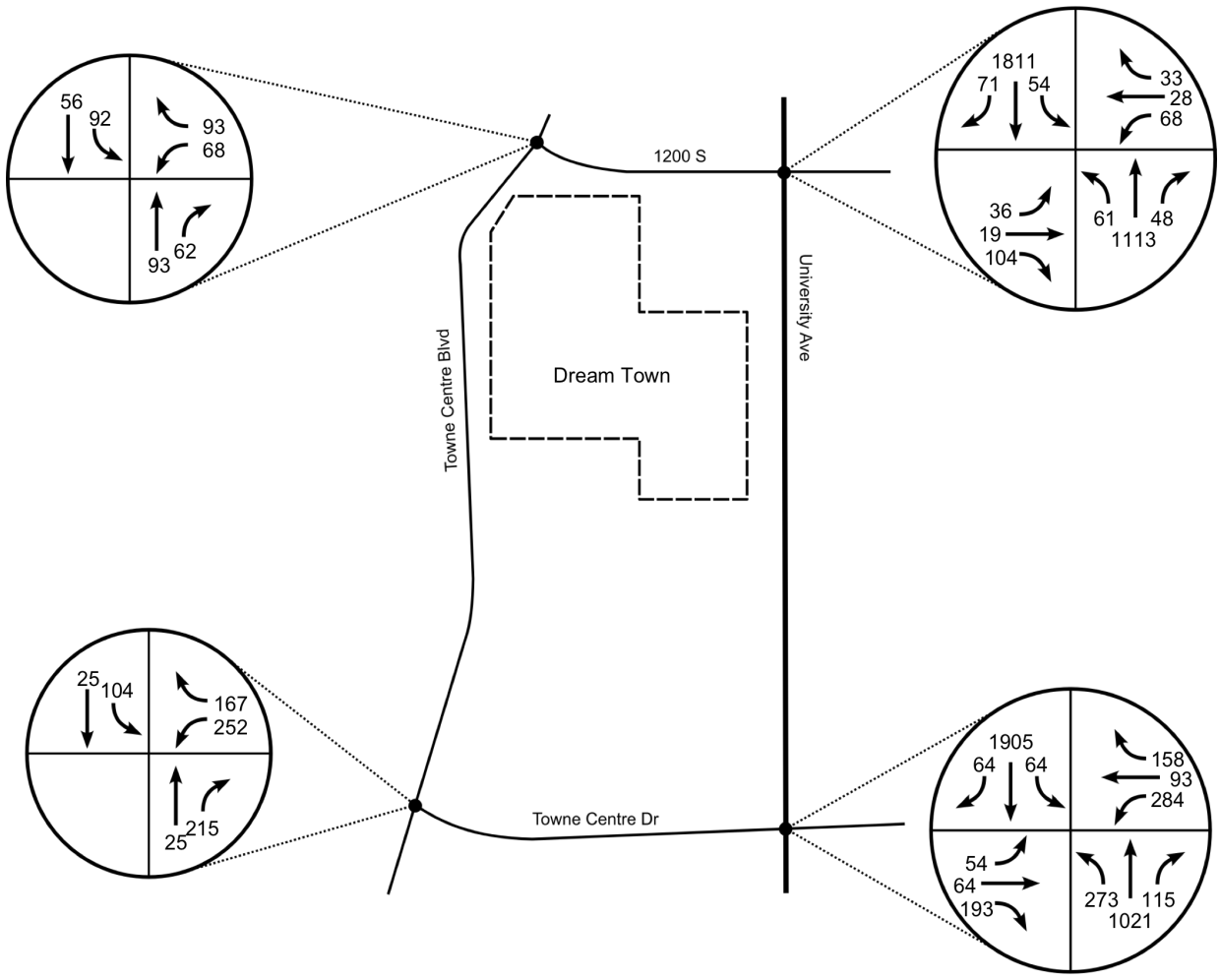

Figure 5.6: Projected background traffic 5 years after opening.

Site Traffic

Figure 5.4 shows the distribution of site trips by entry/exit direction, but these trips need to be assigned to specific intersection movements. This depends substantially on the access layout of the site. There are two accesses to the site: one on 1200 South and one on University Ave.[1]—for a full discussion of access and site layout see Chapter 6.

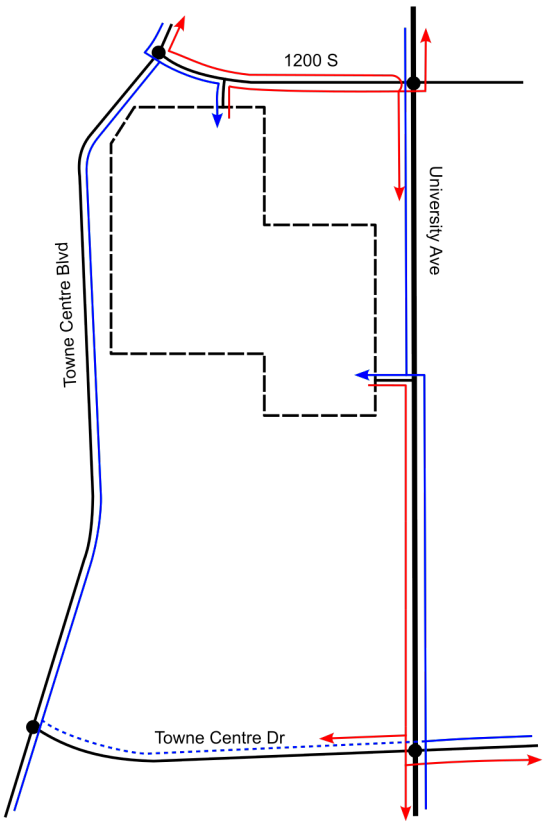

Figure 5.7: Assumed site access routes. Blue indicates entering trips and red indicates exiting trips.

Figure 5.7 shows the assumed site access routes from each access road. Several assumptions are made regarding this:

[1] This access is shared with the development to the south.

- Because of the raised median on 1200 South, no left turns are possible into or out of this driveway. This primarily affects drivers entering the site from the northeast (SB University Avenue) and exiting to the northwest (NB Towne Centre Blvd.). The former are assumed to continue past 1200 South and use the University Ave. driveway, and the latter are assumed to exit on 1200 South EB, then U-turn at the University Ave. intersection.
- It is assumed that *all* vehicles exiting to the northwest (on Towne Centre Blvd.) use the above route, rather than using the University Ave. exit and looping around the site.
- No vehicles for primary trips are assumed to turn left (NB) from the University Ave. driveway. This is due to the difficulty of doing so combined with the ability to use the 1200 South driveway and turn left at the University Ave. intersection.
- All pass-by trips are assumed to use the University Ave. driveway for both entering and exiting (this includes left turns onto University Ave. exiting the site). Note that pass-by trips are considered to come from the existing background traffic, rather than new trips to the study area. These trips are therefore added to the intersection movements into or out of the University Ave. driveway, and only the *difference* between exiting and entering trips is added to the downstream intersection movements. Since there are so few pass-by trips, these differences are placed only on intersection through movements for simplicity.
- All vehicles exiting to the southwest (SB Towne Centre Blvd.) are assumed to use the University Ave. driveway, rather than making the required U-turn and left turn via the 1200 South driveway.
- Where multiple routes exist to enter/exit the site from the same direction, the assignment is split between them. This primarily affects the trips exiting to SB University Ave. and the trips entering from NB University Ave. and WB Towne Centre Drive.
    - The trips exiting to SB University Ave. can exit the site from either driveway. Most drivers will likely use the University Ave. driveway, but the 1200 South driveway may be more convenient for vehicles near the northwest corner of

the site. A 2/3-1/3 split is used for the University Ave. driveway and the 1200 South driveway, respectively.

- The trips entering from NB University Ave. and WB Towne Centre Drive can either turn left into the University Ave. driveway or loop around the site to use the 1200 South driveway. The latter option would generally be disregarded due to its complexity; however, since left turns off of University Ave. are difficult in this corridor (it is required to cross 3 lanes of opposing traffic), some drivers may elect to loop around the site. These trips are split proportionally according to background traffic distribution (see Figure 4.2), but since the left turn off of University Ave. is assumed to still be the more prevalent route the proportion of vehicles using WB Towne Centre Dr. is halved.

Tables 8–11 provide information on the total number of site trips generated (both primary and pass-by). Using this information and the above assumptions, the primary site trips are assigned to the intersection movements as shown in Figure 5.8, and the pass-by trips as shown in Figure 5.9. These trips are added together to give an expected site impact as shown in Figure 5.10.

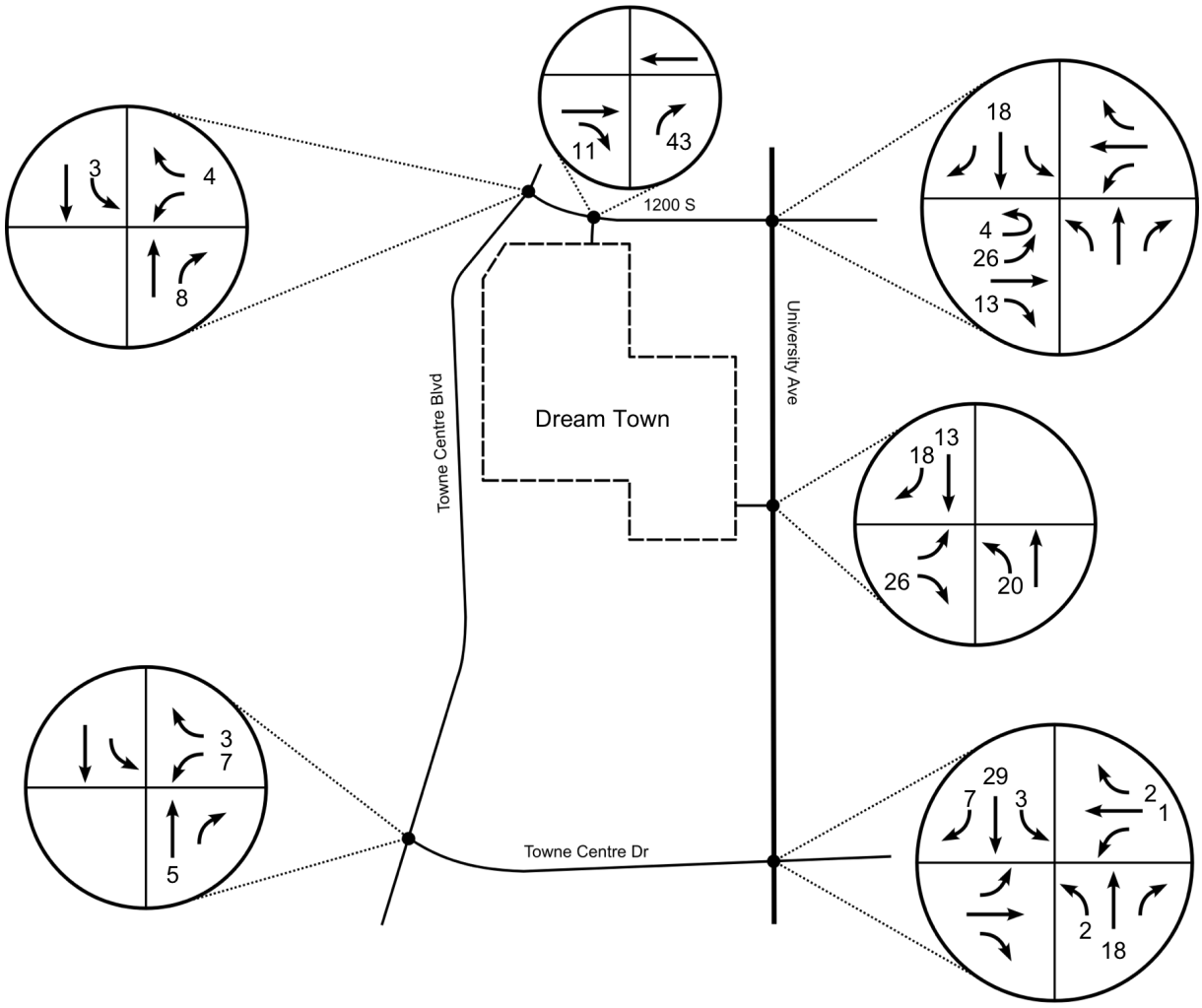

Figure 5.8: New site trips assigned to intersection movements.

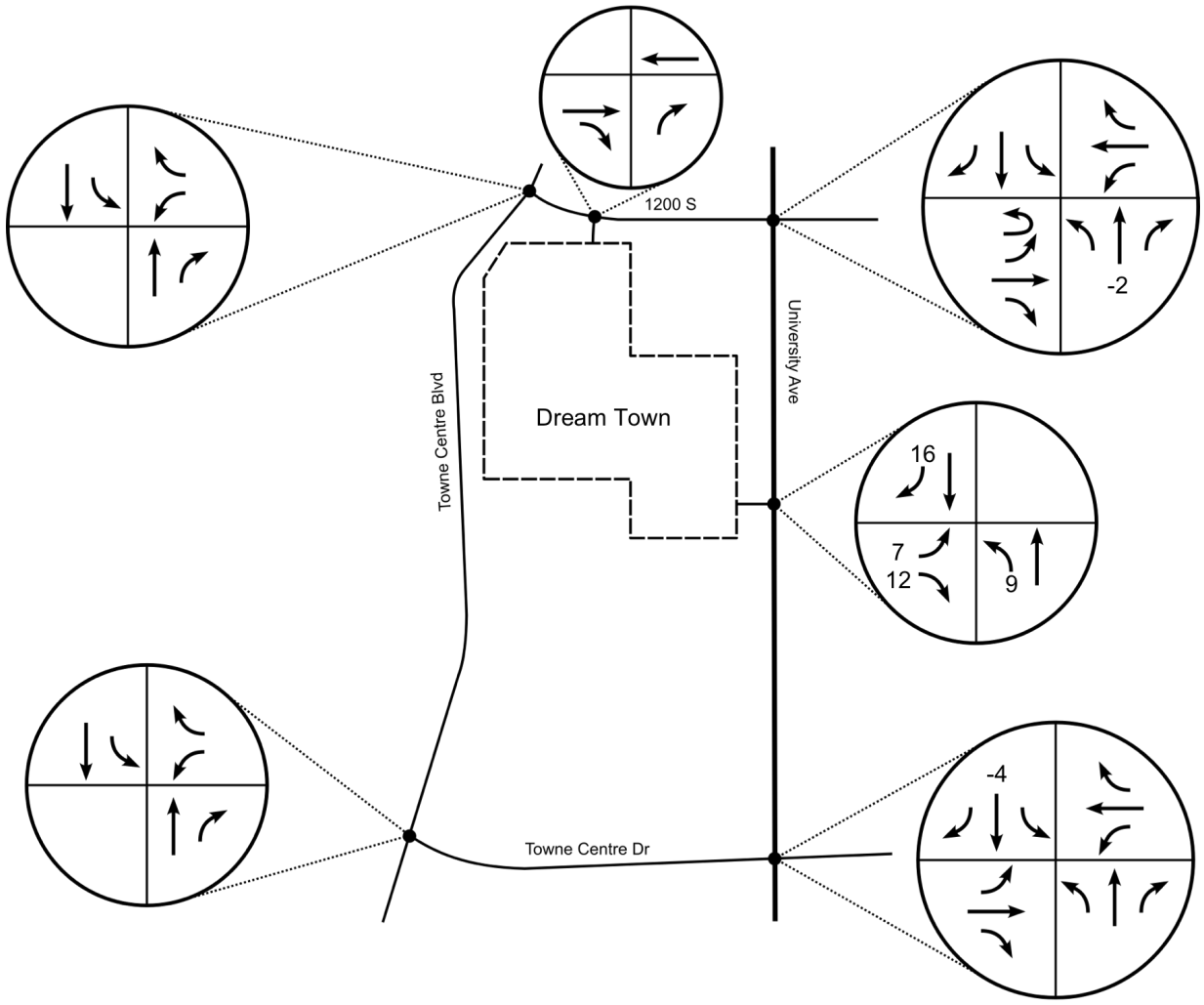

Figure 5.9: Pass-by site trips assigned to intersection movements.

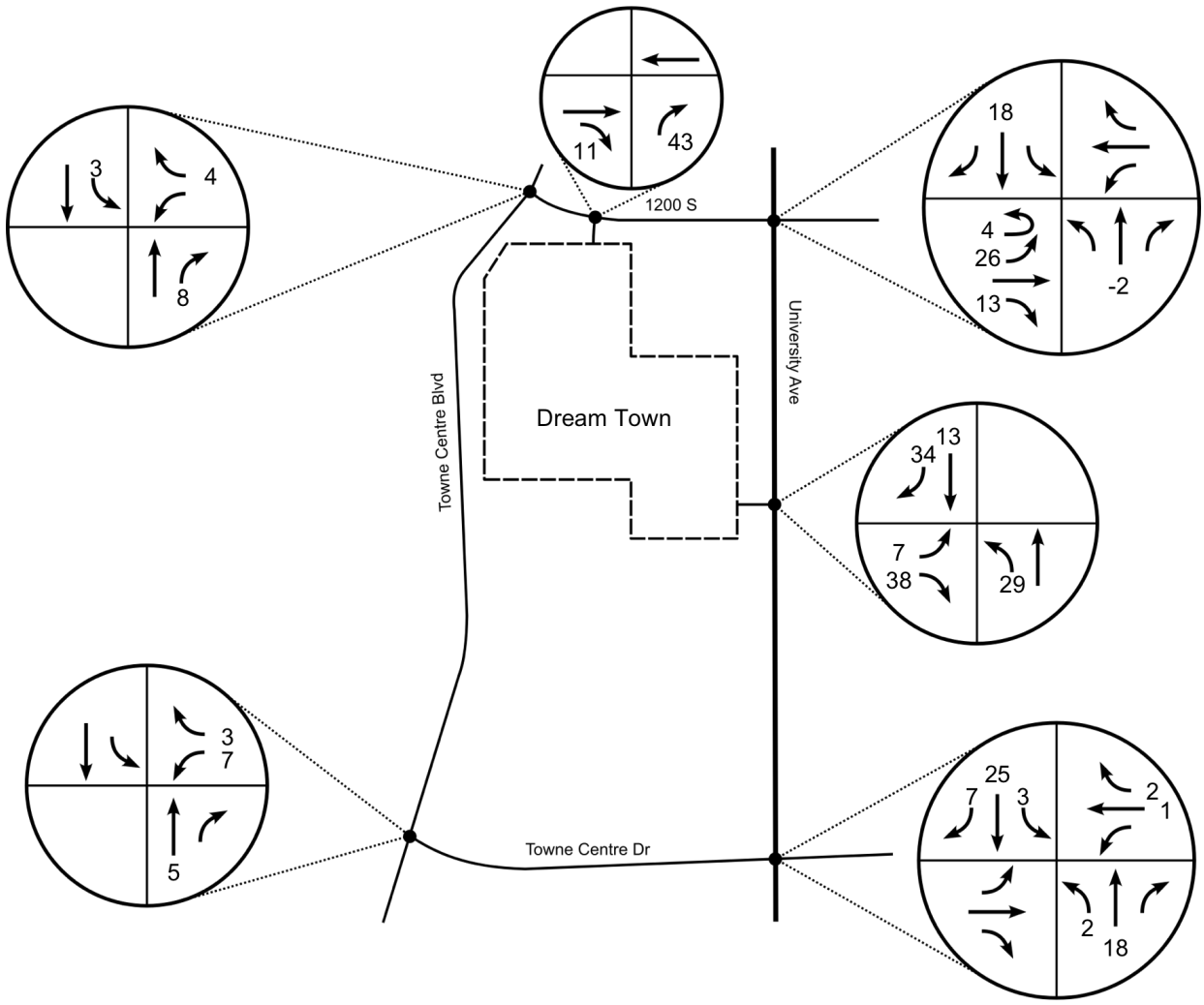

Figure 5.10: Total site trips assigned to intersection movements.

Total Traffic

The two horizon years of interest in this analysis are opening day (2024) and 5 years after opening (2029). The background traffic is expected to grow as determined in Section 5.4.1; however, the site traffic is not assumed to change at any of the horizon times. Figures 5.11 and 5.12 show the total vehicles (background traffic plus site trips) at each intersection turning movement for the two horizon years.

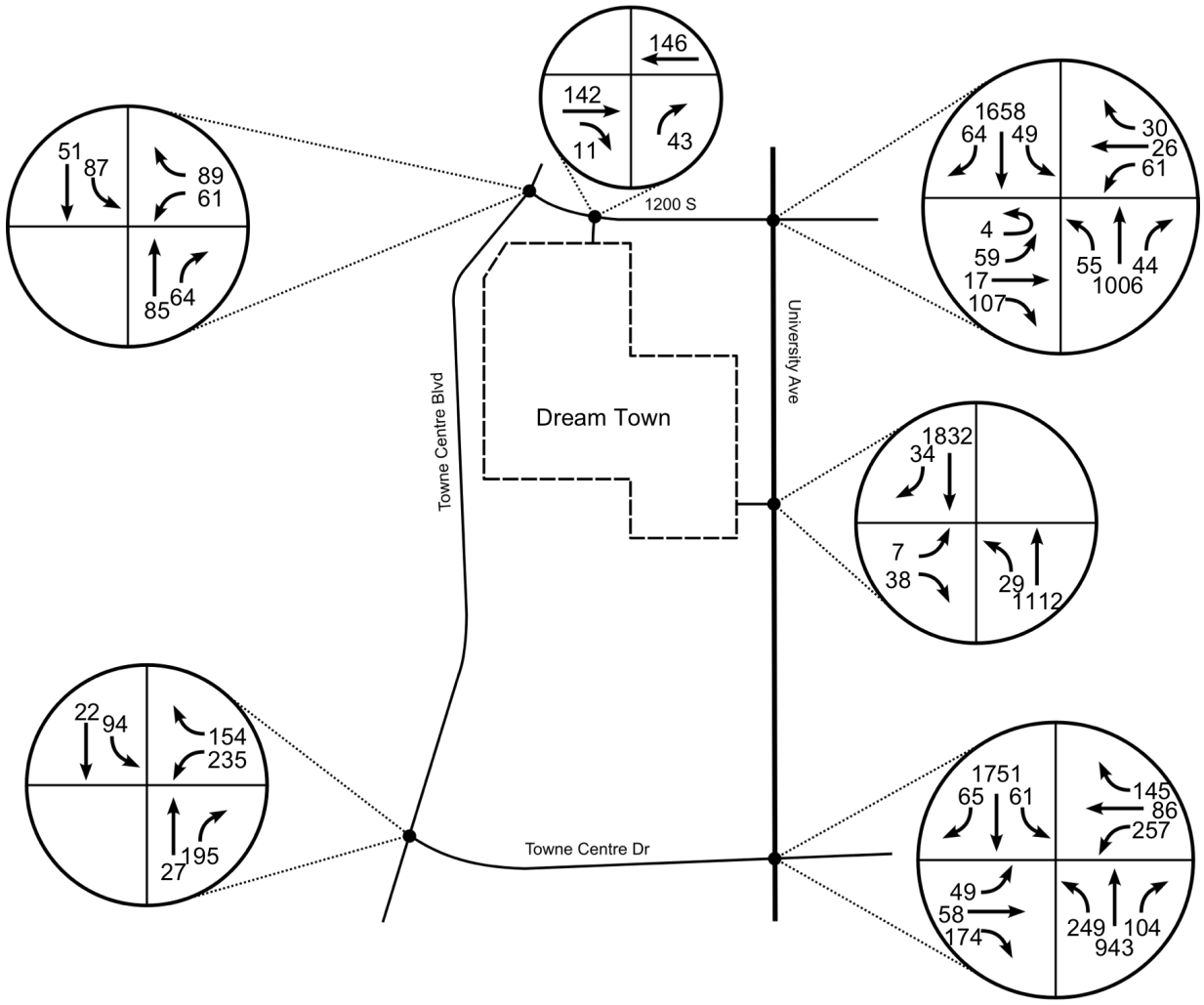

Figure 5.11: Total expected volumes including site and non-site traffic, opening day (2024).

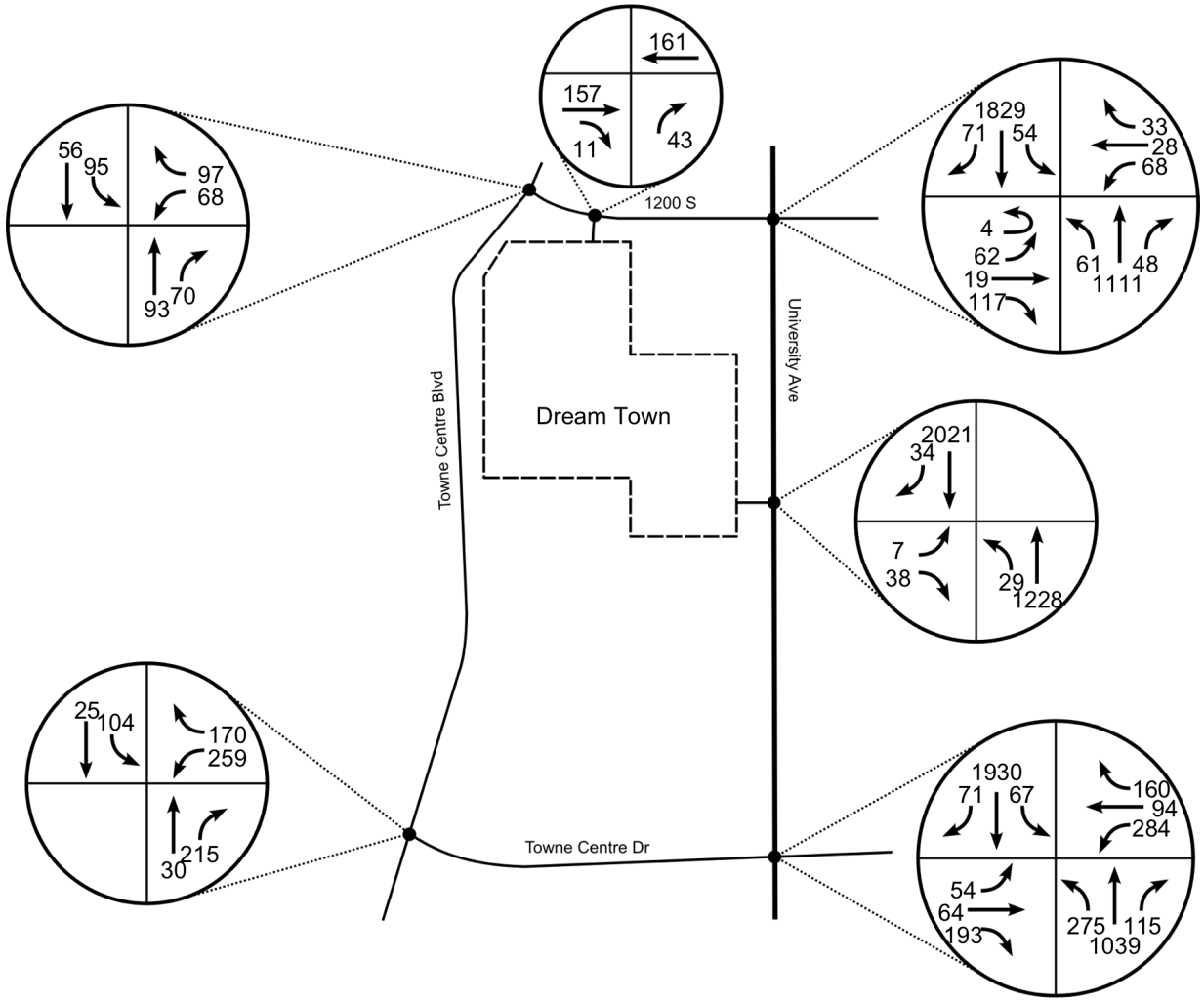

Figure 5.12: Total expected volumes including site and non-site traffic, 5 years after opening (2029).

6. ACCESS AND PARKING LAYOUT

The impact of site traffic on the roadways and intersections in the study area is significantly affected by the design of the site, at least regarding the site access locations. This section discusses various access options and recommends specific access locations for the development. The access locations will also affect the layout of the site, including parking, so a discussion of parking requirements and a proposed site layout is presented as well.

ACCESS

Figure 6.1 shows aerial imagery of the site location in its existing condition (2023). The site location is marked, as well as the existing external and cross-accesses. There is an existing access on University Ave. just to the south of the site, and a convenient cross-access between this development and Dream Town. There is also an existing access on University Ave. at the north of Dream Town, but this access is too close to the northern adjacent access for the Exxon and does not comply with UDOT spacing guidelines. University Ave. is classified as an Access Category 5 roadway (Utah Department of Transportation 2023a), and as such requires 350-foot spacing between driveways per Utah rule R930-6-7 (3)(a) (Utah Department of Transportation 2019). This access therefore will be removed. Even with this access removed, there is not enough space per rule R930-6-7 to put an additional access to Dream Town on University Ave. (the distance between the nearest adjacent driveways is roughly 370 feet), so the only access from University Ave. will be via the cross-access with the development to the south.

There is also an existing driveway on 1200 South, and this is proposed to be used in the site development, though with an updated design. This is critical to aid in NB exiting trips, as turning left directly onto University Ave. is often difficult with even moderate traffic volumes. Since 1200 South is not a UDOT road, the R930-6-7 spacing guidelines do not apply, so the current access location is valid. Additionally, there is a cross-access to/from the Exxon on the northeast of the site; since the Exxon has accesses on 1200 South and University Ave. already, this cross-access is not assumed to affect the trip/driveway assignment, and will remain in the site layout design. There is also a pedestrian access to

Towne Center Blvd. near the south of the site that will be used. All of these proposals for access locations are noted in Figure 6.1.

PARKING REQUIREMENTS

The three land uses in the development are a sit-down restaurant, office, and specialty retail. The office will likely have very low turnover, as most trips will be made by employees with few visitor trips. This would 8.5-foot stall widths for parking near the office. The restaurant and retail locations will have a higher turnover, though, and therefore 9.5-foot stall widths are recommended. However, since this is a multi-use development with a shared parking lot, and since the site is not very large, it would be more reasonable to have consistent stall widths. As such, 9-foot stalls are recommended as the average of the suggested widths.

Because of the site's size and shape, typical one-way angled parking modules would unreasonably restrict vehicle movement. Therefore, 90° parking is recommended. The stall lengths are recommended at 18 feet, with 12 feet of clearance to perform a parking maneuver, both based on common practice. Additionally, since the office, restaurant, and retail have different peak periods, some parking may be shared between the two. This is estimated at 10% of the total parking based on common practice and *Shared Parking* (Urban Land Institute et al. 2020).

There are two relevant methods of calculating the number or required parking spaces for Dream Town: ITE's *Parking Generation* (Institute of Transportation Engineers 2019) and Provo city code (Provo City Utah 2022a). These are presented and compared below.

Figure 6.1: Existing site accesses and proposed changes.

ITE Parking Generation

There are several assumptions that need to be made when using *Parking Generation*. For many land uses, several different parking plots are provided based on setting/location, day of week, and choice of dependent variable.Dream Town is located in a "General Urban/Suburban" setting (as opposed to "Dense Multi-Use Urban"), and this study focuses on weekday parking demand. Additionally, the dependent variable used in all cases is gross floor area (GFA).

Many plots in *Parking Generation* provide both a fitted curve equation and an average rate, either of which may be used to predict parking demand. For the office building (ITE land use code 710), the fitted curve equation tends to over-predict parking demand for small GFA values (less than about 80,000 ft^2), and the average rate is more accurate, so the average rate is used. For the restaurant (code 932), no fitted curve equation is given, so the average rate is used. The hardware store (code 816) presents somewhat of a problem since the sample size is so small. There is an average rate provided; however, it is a very poor predictor of the observed data. However, it is fairly straightforward to interpolate from the observed data points, and a conservative interpolation is done here to estimate parking demand (see Appendix D). All *Parking Generation* plots used can be found in Appendix D, and Table 16 presents the preliminary parking estimates for the site.

Table 16: *Parking Generation Estimates for Site*

Land Use	ITE Code	Equation	GFA	Parking Spaces
Office	710	$\frac{GFA}{1000} \times 2.39$	18,000	43
Restaurant	932	$\frac{GFA}{1000} \times 9.44$	8,000	76
Retail	816	$interpolated$	18,000	7

This gives a total of 126 parking spaces.

Provo City Code

Provo city code §14.37.060 (Provo City Utah 2022a) outlines the following requirements for the land uses in Dream Town:

- **Office:** 1 space per 250 ft^2 of GFA
- **Restaurants:** 1 space per 100 ft^2 of GFA
- **Retail (hardware, etc.):** 1 space per 600 ft^2 of GFA

The gross floor area and required parking of the three land uses is:

- **Office:** 18000 ft^2; 72 spaces
- **Restaurant:** 8000 ft^2; 80 spaces
- **Hardware store:** 18000 ft^2; 30 spaces

This gives a total of 182 parking spaces.

Method Comparison

Provo city code requires more parking than *Parking Generation* predicts (182 vs. 126 spaces), so this analysis will use the former. This value, however, should be reduced by 10% due to the mixed land use. This gives a total number of 164 required parking spaces. Note that per Provo city code §14.37.110 (Provo City Utah 2022a) 6 total handicap parking spaces are required for a parking lot of this size. Since the buildings will be placed adjacent to each other (see Section 6.3), these spaces are spread out strategically along the length of the buildings.

PROPOSED SITE LAYOUT

7. TRAFFIC AND IMPROVEMENT ANALYSIS

This section analyzes the impact of Dream Town on the roadways and intersections in the study area. A LOS analysis is performed for both opening day (2024) and the 5-year horizon (2029). The analysis is done for both the no-build and build scenarios at each horizon year.

LEVEL OF SERVICE

Based on the traffic assignment as determined in Section 5.4, a LOS analysis was performed to determine the average delay per vehicle, and assign a LOS designation (see Table 3). Both the no-build and build scenarios are analyzed, and these results are presented in Tables 17–20. The direct analysis reports are also provided in Appendix E.

Note that because of limitations in the Synchro software used for this analysis (Trafficware 2019), the U-turn movements at the University Ave./1200 South intersection could not be analyzed, so these were added to the left-turn movement.

Table 17: *Opening Year (2024) LOS Analysis, no-build scenario*

Intersection	Direction	Control Delay (sec)				Level of Service			
		Left	Thru	Right	Intersection	Left	Thru	Right	Intersection
Towne Centre Blvd. / 1200 South	SB	9.2	7.1		8.7	A	A		A
	WB	9.3		8		A		A	
	NB		9.1	9.1			A	A	
Towne Centre Blvd. / Towne Centre Dr.	SB	12.1	6.4		10.8	B	A		B
	WB	10.5		-		B		-	
	NB		13.2	-			B	-	
University Ave. / 1200 South	SB	3.2	6.6	4.3	9.6	A	A	A	A
	WB	70.0	0.0	67.3		E	A	E	
	NB	4.6	0.2	0.1		A	A	A	
	EB	71.4	64.6	73.6		E	E	E	
University Ave. / Towne Centre Dr.	SB	72.0	1.3	0.2	24.2	E	A	A	C
	WB	85.2	49.3	53.6		F	D	D	
	NB	77.9	15.3	13.3		E	B	B	
	EB	74.6	57.5	69.1		E	E	E	

Table 18: *Opening Year (2024) LOS Analysis, build scenario*

Intersection	Direction	Control Delay (sec)				Level of Service			
		Left	Thru	Right	Intersection	Left	Thru	Right	Intersection
Towne Centre Blvd. / 1200 South	SB	9.3	7.1		8.8	A	A		A
	WB	9.3		8.1		A		A	
	NB		9.1	9.1			A	A	
Towne Centre Blvd. / Towne Centre Dr.	SB	12.1	6.4		10.9	B	A		B
	WB	10.6		-		B		-	
	NB		13.2	-			B	-	
University Ave. / 1200 South	SB	3.9	7.8	5.1	10.8	A	A	A	B
	WB	66.2	0	63.8		E	A	E	
	NB	5.6	0.2	0.1		A	A	A	
	EB	70.3	61.6	68.3		E	E	E	
University Ave. / Towne Centre Dr.	SB	72.1	1.5	0.2	24.2	E	A	A	C
	WB	85.2	49.3	53.7		F	D	D	
	NB	78.2	15.5	13.3		E	B	B	
	EB	74.6	57.5	69.1		E	E	E	
University Ave. Access	SB		-	-	0.8		-	-	
	NB	44.9	-			E	-		
	EB	27.2		-		D		-	
1200 South Access	WB		-		1.2		-		
	NB			9.7				A	
	EB		-	-			-	-	

Table 19: *5-Year (2029) LOS Analysis, no-build scenario*

Intersection	Direction	Control Delay (sec)				Level of Service			
		Left	Thru	Right	Intersection	Left	Thru	Right	Intersection
Towne Centre Blvd. / 1200 South	SB	9.4	7.2		8.9	A	A		A
	WB	9.5		8.2		A		A	
	NB		9.4	9.4			A	A	
Towne Centre Blvd. / Towne Centre Dr.	SB	12.1	6.3		11.0	B	A		B
	WB	10.8		-		B		-	
	NB		13.2	-			B	-	
University Ave. / 1200 South	SB	3.4	7.5	4.6	10.1	A	A	A	B
	WB	69.6	0.0	66.4		E	A	E	
	NB	5.9	0.2	0.1		A	A	A	
	EB	71.0	63.6	72.7		E	E	E	
University Ave. / Towne Centre Dr.	SB	71.5	2.3	0.2	25.7	E	A	A	C
	WB	90.4	47.2	51.8		F	D	D	
	NB	80.7	17.6	14.9		F	B	B	
	EB	74.3	55.9	71.7		E	E	E	

Table 20: *5-Year (2029) LOS Analysis, build scenario*

Intersection	Direction	Control Delay (sec)				Level of Service			
		Left	Thru	Right	Intersection	Left	Thru	Right	Intersection
Towne Centre Blvd. / 1200 South	SB	9.5	7.2		8.8	A	A		A
	WB	9.5		8.3		A		A	
	NB		9.5	9.5			A	A	
Towne Centre Blvd. / Towne Centre Dr.	SB	12.2	6.3		11.2	B	A		B
	WB	11.0		-		B		-	
	NB		13.2	-			B	-	
University Ave. / 1200 South	SB	4.2	8.9	5.4	11.3	A	A	A	B
	WB	65.9	0	63.1		E	A	E	
	NB	7.2	0.2	0.1		A	A	A	
	EB	70.1	60.7	67.8		E	E	E	
University Ave. / Towne Centre Dr.	SB	71.7	2.9	0.2	25.9	E	A	A	C
	WB	90.4	47.2	51.9		F	D	D	
	NB	80.8	17.8	15		F	B	B	
	EB	74.3	55.9	71.7		E	E	E	
University Ave. Access	SB		-	-	1.0		-	-	
	NB	60.7	-			F	-		
	EB	32.2		-		D		-	
1200 South Access	WB		-		1.1		-		
	NB			9.8				A	
	EB		-	-			-	-	

Based on these results, the Dream Town development has virtually no impact on the delay/LOS of the intersections at either the opening day or 5-year horizons. While this analysis does show movements that perform poorly, these do so even with the background traffic alone, and in fact the delay does not substantially increase between the opening day and 5-year background traffic.

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APPENDIX A. SYNCHRO LOS ANALYSIS (EXISTING)

This appendix contains the results of the Synchro LOS analysis for the existing road volumes, signal timings, and lane configurations. The analysis of the AWSC 1200 South / Towne Centre Blvd intersection is given first, followed by the analysis for the remaining (signalized) intersections.

Intersection	
Intersection Delay, s/veh	8.6
Intersection LOS	A

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	60	83	83	55	82	50
Future Vol, veh/h	60	83	83	55	82	50
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	65	90	90	60	89	54
Number of Lanes	1	1	1	0	1	2

Approach	WB	NB	SB
Opposing Approach		SB	NB
Opposing Lanes	0	3	1
Conflicting Approach Left	NB		WB
Conflicting Lanes Left	1	0	2
Conflicting Approach Right	SB	WB	
Conflicting Lanes Right	3	2	0
HCM Control Delay	8.5	9	8.4
HCM LOS	A	A	A

Lane	NBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
Vol Left, %	0%	100%	0%	100%	0%	0%
Vol Thru, %	60%	0%	0%	0%	100%	100%
Vol Right, %	40%	0%	100%	0%	0%	0%
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop
Traffic Vol by Lane	138	60	83	82	25	25
LT Vol	0	60	0	82	0	0
Through Vol	83	0	0	0	25	25
RT Vol	55	0	83	0	0	0
Lane Flow Rate	150	65	90	89	27	27
Geometry Grp	8	8	8	7	7	7
Degree of Util (X)	0.208	0.105	0.116	0.138	0.038	0.025
Departure Headway (Hd)	4.986	5.818	4.615	5.569	5.066	3.324
Convergence, Y/N	Yes	Yes	Yes	Yes	Yes	Yes
Cap	720	616	776	644	707	1074
Service Time	2.719	3.553	2.35	3.298	2.795	1.052
HCM Lane V/C Ratio	0.208	0.106	0.116	0.138	0.038	0.025
HCM Control Delay	9	9.2	8	9.2	8	6.1
HCM Lane LOS	A	A	A	A	A	A
HCM 95th-tile Q	0.8	0.4	0.4	0.5	0.1	0.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	17	92	60	25	29	54	988	43	48	1608	63
Future Volume (veh/h)	32	17	92	60	25	29	54	988	43	48	1608	63
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	35	18	100	65	27	32	59	1074	47	52	1748	68
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	105	143	121	128	54	69	270	3936	1222	487	3931	1220
Arrive On Green	0.08	0.08	0.08	0.08	0.08	0.08	0.06	1.00	1.00	0.03	0.77	0.77
Sat Flow, veh/h	1344	1870	1585	1074	710	903	1781	5106	1585	1781	5106	1585
Grp Volume(v), veh/h	35	18	100	69	0	55	59	1074	47	52	1748	68
Grp Sat Flow(s),veh/h/ln	1344	1870	1585	1147	0	1540	1781	1702	1585	1781	1702	1585
Q Serve(g_s), s	3.8	1.3	9.3	7.9	0.0	5.1	1.0	0.0	0.0	0.9	18.0	1.5
Cycle Q Clear(g_c), s	8.9	1.3	9.3	9.2	0.0	5.1	1.0	0.0	0.0	0.9	18.0	1.5
Prop In Lane	1.00		1.00	0.94		0.59	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	105	143	121	134	0	118	270	3936	1222	487	3931	1220
V/C Ratio(X)	0.33	0.13	0.83	0.52	0.00	0.46	0.22	0.27	0.04	0.11	0.44	0.06
Avail Cap(c_a), veh/h	244	337	285	269	0	277	340	3936	1222	560	3931	1220
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	0.96	0.96	0.96	1.00	1.00	1.00
Uniform Delay (d), s/veh	70.6	64.6	68.3	68.9	0.0	66.3	4.3	0.0	0.0	3.1	6.0	4.2
Incr Delay (d2), s/veh	0.7	0.1	5.3	1.1	0.0	1.1	0.1	0.2	0.1	0.0	0.4	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.4	0.7	4.0	2.7	0.0	2.1	0.3	0.1	0.0	0.3	5.9	0.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	71.3	64.8	73.6	70.1	0.0	67.4	4.5	0.2	0.1	3.1	6.4	4.2
LnGrp LOS	E	E	E	E	A	E	A	A	A	A	A	A
Approach Vol, veh/h		153			124			1180			1868	
Approach Delay, s/veh		72.0			68.9			0.4			6.2	
Approach LOS		E			E			A			A	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	8.9	121.6		19.5	9.1	121.5		19.5				
Change Period (Y+Rc), s	4.5	6.0		8.0	4.5	6.0		8.0				
Max Green Setting (Gmax), s	10.5	94.0		27.0	10.5	94.0		27.0				
Max Q Clear Time (g_c+I1), s	2.9	2.0		11.3	3.0	20.0		11.2				
Green Ext Time (p_c), s	0.0	3.6		0.1	0.0	7.4		0.2				

Intersection Summary	
HCM 6th Ctrl Delay	9.5
HCM 6th LOS	A

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	224	148	22	191	92	22
Future Volume (veh/h)	224	148	22	191	92	22
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	243	0	24	0	100	24
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	611		104		332	1167
Arrive On Green	0.18	0.00	0.06	0.00	0.10	0.33
Sat Flow, veh/h	3456	1585	1870	1585	3456	3647
Grp Volume(v), veh/h	243	0	24	0	100	24
Grp Sat Flow(s),veh/h/ln	1728	1585	1870	1585	1728	1777
Q Serve(g_s), s	1.8	0.0	0.3	0.0	0.8	0.1
Cycle Q Clear(g_c), s	1.8	0.0	0.3	0.0	0.8	0.1
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	611		104		332	1167
V/C Ratio(X)	0.40		0.23		0.30	0.02
Avail Cap(c_a), veh/h	2687		1587		2443	3015
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	1.00	1.00
Uniform Delay (d), s/veh	10.3	0.0	12.8	0.0	11.9	6.4
Incr Delay (d2), s/veh	0.2	0.0	0.4	0.0	0.2	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.5	0.0	0.1	0.0	0.2	0.0
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	10.5	0.0	13.2	0.0	12.1	6.4
LnGrp LOS	B		B		B	A
Approach Vol, veh/h	243		24			124
Approach Delay, s/veh	10.5		13.2			11.0
Approach LOS	B		B			B

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	7.7	7.6		13.0		15.3
Change Period (Y+Rc), s	5.0	6.0		8.0		6.0
Max Green Setting (Gmax), s	20.0	24.0		22.0		24.0
Max Q Clear Time (g_c+I1), s	2.8	2.3		3.8		2.1
Green Ext Time (p_c), s	0.1	0.0		0.4		0.0

Intersection Summary	
HCM 6th Ctrl Delay	10.8
HCM 6th LOS	B

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	48	57	171	252	83	140	242	907	102	57	1692	57
Future Volume (veh/h)	48	57	171	252	83	140	242	907	102	57	1692	57
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	52	62	186	274	90	152	263	986	111	62	1839	62
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	87	465	207	317	702	313	307	3075	955	99	2768	859
Arrive On Green	0.03	0.13	0.13	0.09	0.20	0.20	0.09	0.60	0.60	0.06	1.00	1.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	3456	5106	1585	3456	5106	1585
Grp Volume(v), veh/h	52	62	186	274	90	152	263	986	111	62	1839	62
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1728	1702	1585	1728	1702	1585
Q Serve(g_s), s	2.2	2.3	17.3	11.7	3.1	12.8	11.3	14.3	4.5	2.6	0.0	0.0
Cycle Q Clear(g_c), s	2.2	2.3	17.3	11.7	3.1	12.8	11.3	14.3	4.5	2.6	0.0	0.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	87	465	207	317	702	313	307	3075	955	99	2768	859
V/C Ratio(X)	0.60	0.13	0.90	0.86	0.13	0.49	0.86	0.32	0.12	0.63	0.66	0.07
Avail Cap(c_a), veh/h	346	877	391	346	877	391	415	3075	955	415	2768	859
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00
Upstream Filter(I)	0.99	0.99	0.99	1.00	1.00	1.00	1.00	1.00	1.00	0.90	0.90	0.90
Uniform Delay (d), s/veh	72.4	57.7	64.2	67.2	49.6	53.4	67.4	14.7	12.8	69.9	0.0	0.0
Incr Delay (d2), s/veh	2.4	0.0	5.4	17.4	0.0	0.4	10.0	0.3	0.2	2.2	1.2	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	1.1	7.4	6.0	1.4	5.2	5.4	5.6	1.7	1.2	0.3	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	74.8	57.7	69.6	84.6	49.6	53.9	77.4	15.0	13.0	72.1	1.2	0.1
LnGrp LOS	E	E	E	F	D	D	E	B	B	E	A	A
Approach Vol, veh/h		300			516			1360			1963	
Approach Delay, s/veh		68.0			69.5			26.9			3.4	
Approach LOS		E			E			C			A	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.3	96.3	18.8	25.6	18.3	87.3	8.8	35.6
Change Period (Y+Rc), s	5.0	6.0	5.0	6.0	5.0	6.0	5.0	6.0
Max Green Setting (Gmax), s	18.0	58.0	15.0	37.0	18.0	58.0	15.0	37.0
Max Q Clear Time (g_c+I1), s	4.6	16.3	13.7	19.3	13.3	2.0	4.2	14.8
Green Ext Time (p_c), s	0.0	3.7	0.0	0.3	0.1	9.3	0.0	0.3

Intersection Summary	
HCM 6th Ctrl Delay	24.0
HCM 6th LOS	C

APPENDIX B. SIGNALIZED INTERSECTION SIGNAL TIMINGS

This appendix provides signal timings for the three signalized intersections in the study area (University Ave. / 1200 South, University Ave. / Towne Center Dr. (East Bay Blvd.), and Towne Center Blvd. / Towne Center Dr.). It is worth noting that Phases 1/5, 2/6, 3/7, and 4/8 are active simultaneously (in pairs), and the non–left-turn phases include both the through and right-turn movements.

MAX VIEW TIMING

TRAFFIC SIGNAL TIMING PARAMETERS

Number #: 6403 Current Date: 1/24/2020

Intersection: University Ave & 1200 S

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
Direction	SBLT	NB	WBLT	EB	NBLT	SB	EBLT	WB
Min Green	5	15	0	5	5	15	0	5
Bike Green	0	0	0	0	0	0	0	0
Walk I	0	6	0	4	0	6	0	4
Walk II	0	0	0	0	0	0	0	0
Ped Clr	0	10	0	23	0	13	0	21
Veh Ext	2.0	1.2	0.0	0.9	2.0	1.2	0.0	0.9
Max I	25	45	0	25	25	45	0	25
Max II	0	0	0	0	0	0	0	0
Max III	0	0	0	0	0	0	0	0
Yellow	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Red Clear	1.5	2.0	1.5	4.0	1.5	2.0	1.5	4.0
Min Recall	.	X	.	.	.	X	.	.
Max Recall	.	.	.	.	.	.	.	.
Ped Recall	.	.	.	.	.	.	.	.
Dual Entry	.	X	.	X	.	X	.	X

MAX VIEW TIMING

TRAFFIC SIGNAL TIMING PARAMETERS

Number #: 6402 Current Date: 1/24/2020

Intersection: University Ave & East Bay Blvd

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
Direction	SBLT	NB	WBLT	EB	NBLT	SB	EBLT	WB
Min Green	4	10	4	6	4	10	4	6
Bike Green	0	0	0	0	0	0	0	0
Walk I	0	4	0	4	0	4	0	4
Walk II	0	0	0	0	0	0	0	0
Ped Clr	0	23	0	26	0	22	0	26
Veh Ext	0.6	1.4	0.6	1.2	0.6	1.4	0.6	1.2
Max I	30	60	30	35	30	60	30	35
Max II	0	0	0	0	0	0	0	0
Max III	0	0	0	0	0	0	0	0
Yellow	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Red Clear	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Min Recall	.	X	.	.	.	X	.	.
Max Recall	.	.	.	.	.	.	.	.
Ped Recall	.	.	.	.	.	.	.	.
Dual Entry	.	X	.	X	.	X	.	X

MAX VIEW TIMING

TRAFFIC SIGNAL TIMING PARAMETERS

Number #: 6655 Current Date: 1/24/2020

Intersection: Town Center Dr & Town Center Blvd

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
Direction	SBLT	NB	.	WB	.	SB	.	.
Min Green	5	5	0	5	0	5	0	5
Bike Green	0	0	0	0	0	0	0	0
Walk I	0	0	0	4	0	0	0	4
Walk II	0	0	0	0	0	0	0	0
Ped Clr	0	0	0	10	0	0	0	12
Veh Ext	2.0	2.0	0.0	2.0	0.0	2.0	0.0	0.0
Max I	25	30	0	30	0	30	0	30
Max II	0	0	0	0	0	0	0	0
Max III	0	0	0	0	0	0	0	0
Yellow	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Red Clear	2.0	2.0	2.0	4.0	2.0	2.0	2.0	2.0
Min Recall	.	.	.	X	.	.	.	.
Max Recall	.	.	.	.	.	.	.	.
Ped Recall	.	.	.	.	.	.	.	.
Dual Entry	.	X	.	X	.	X	.	X

APPENDIX C. ITE TRIP GENERATION REFERENCE

This appendix contains the ITE Trip Generation Manual references used for trip generation, including to determine the appropriate study level for this TIA. These are land use codes 710 (General Office Building), 816 (Hardware/Paint Store), and 932 (High-Turnover (Sit-Down) Restaurant), for both the daily trips and the PM peak trips. It also contains a diagram of the multi-use trip reduction.

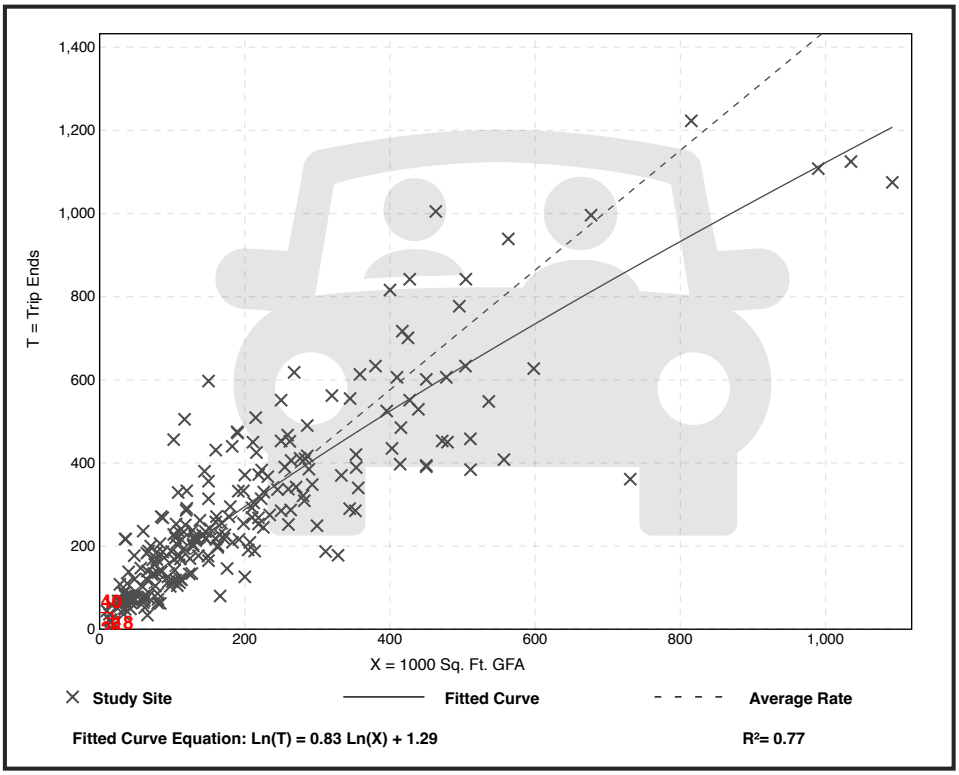
General Office Building
(710)

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: 232
Avg. 1000 Sq. Ft. GFA: 199
Directional Distribution: 17% entering, 83% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
1.44	0.26 - 6.20	0.60

Data Plot and Equation



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General Office Building
(710)

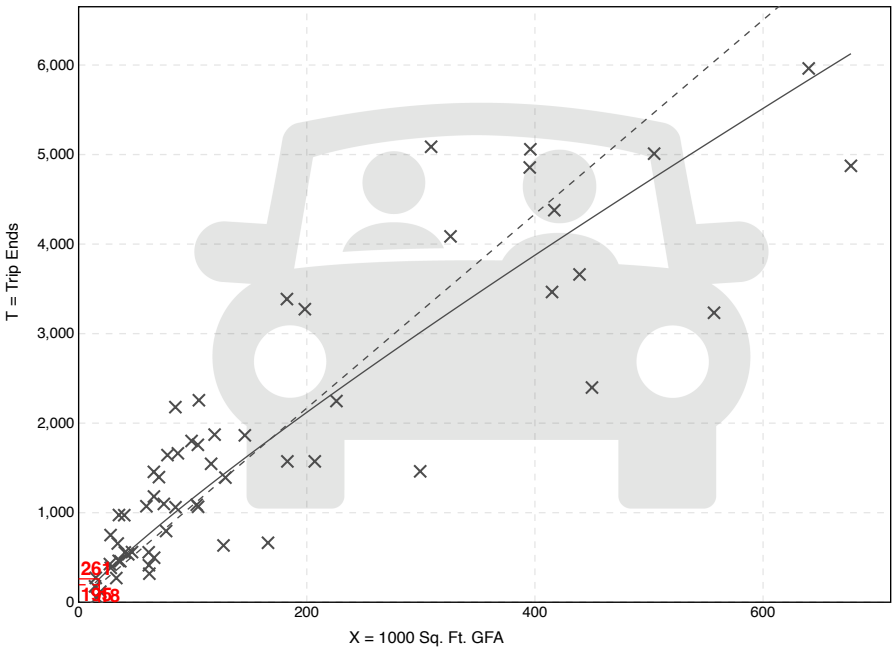
Vehicle Trip Ends vs: 1000 Sq. Ft. GFA
On a: Weekday

Setting/Location: General Urban/Suburban
Number of Studies: 59
Avg. 1000 Sq. Ft. GFA: 163
Directional Distribution: 50% entering, 50% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
10.84	3.27 - 27.56	4.76

Data Plot and Equation



Fitted Curve Equation: Ln(T) = 0.87 Ln(X) + 3.05

R^2= 0.78

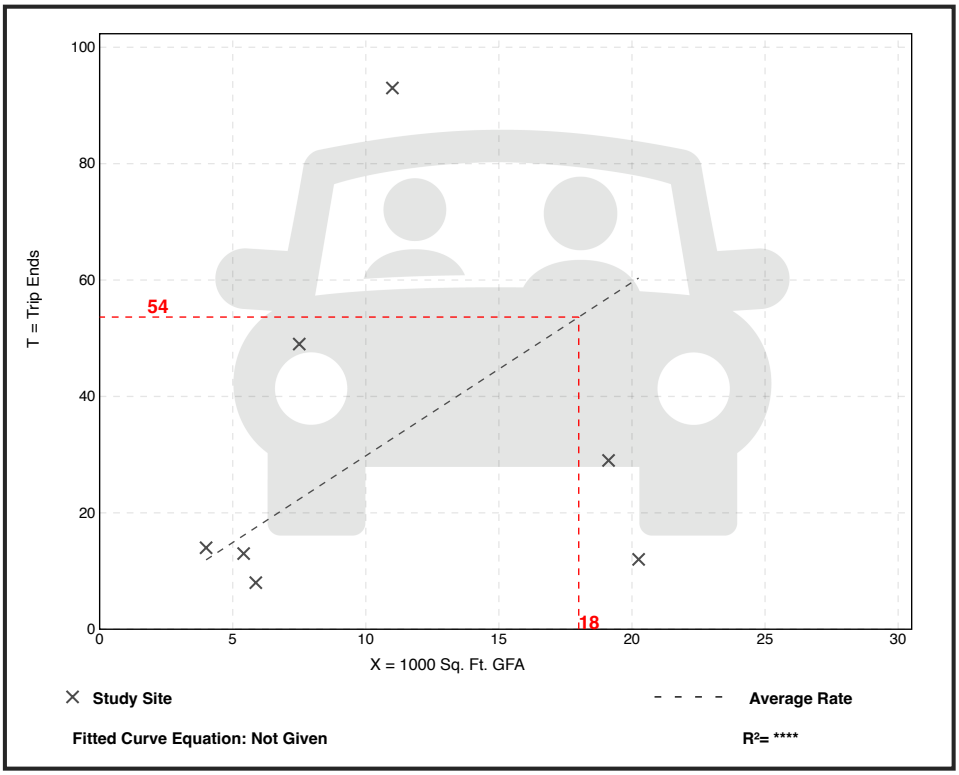
Hardware/Paint Store
(816)

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: 7
Avg. 1000 Sq. Ft. GFA: 10
Directional Distribution: 46% entering, 54% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
2.98	0.59 - 8.45	3.09

Data Plot and Equation



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Hardware/Paint Store
(816)

Vehicle Trip Ends vs: **1000 Sq. Ft. GFA**
On a: **Weekday**

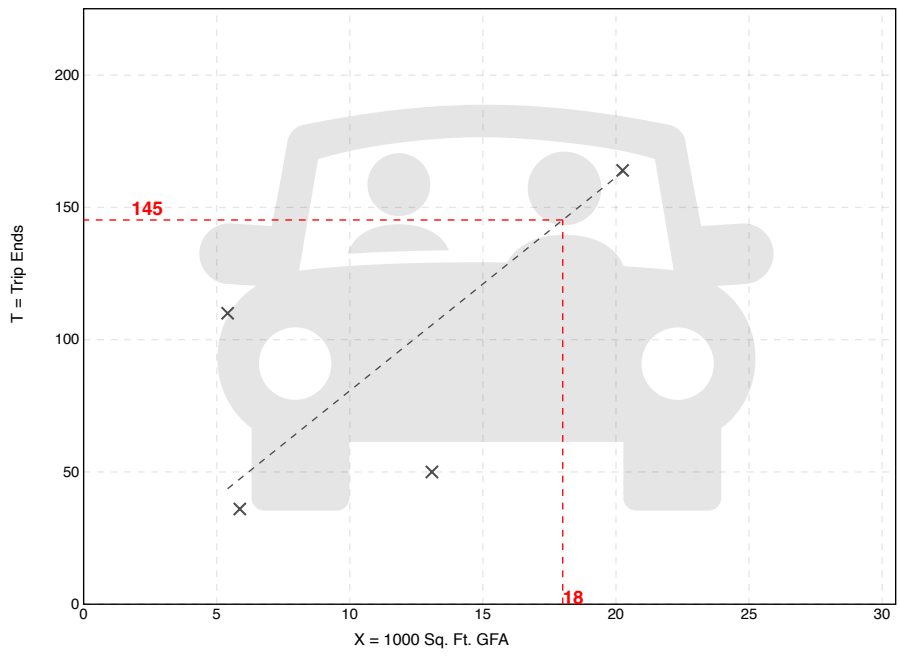
Setting/Location: **General Urban/Suburban**
Number of Studies: 4
Avg. 1000 Sq. Ft. GFA: 11
Directional Distribution: 50% entering, 50% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
8.07	3.82 - 20.33	5.66

Data Plot and Equation

Caution – Small Sample Size



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

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

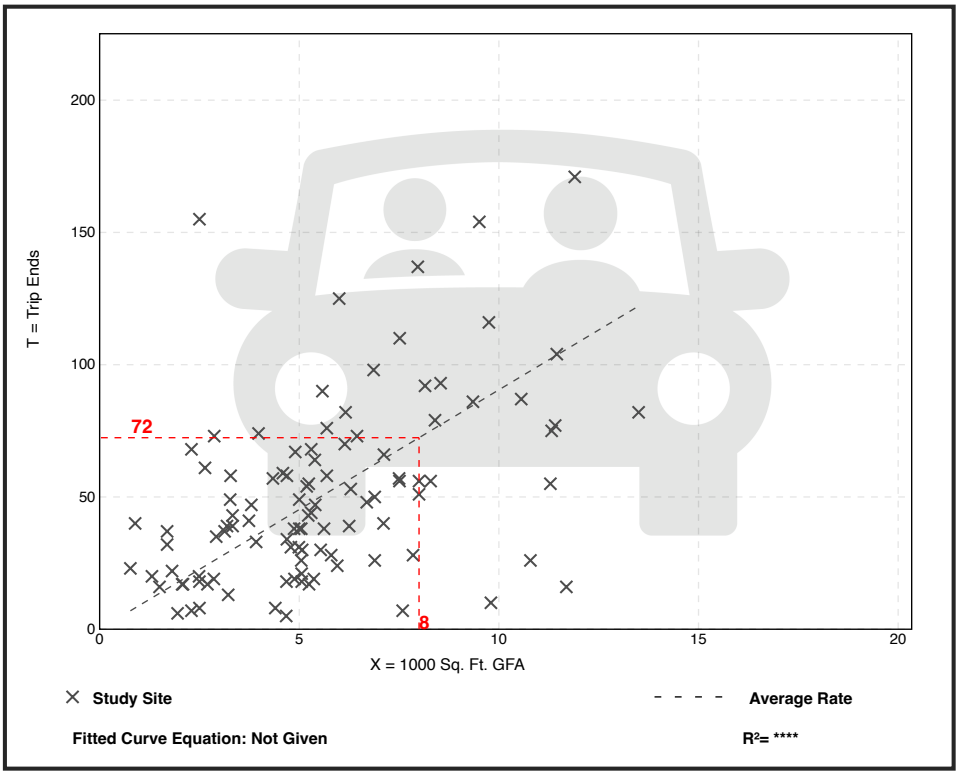
High-Turnover (Sit-Down) Restaurant
(932)

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: 104
Avg. 1000 Sq. Ft. GFA: 6
Directional Distribution: 61% entering, 39% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
9.05	0.92 - 62.00	6.18

Data Plot and Equation



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High-Turnover (Sit-Down) Restaurant
(932)

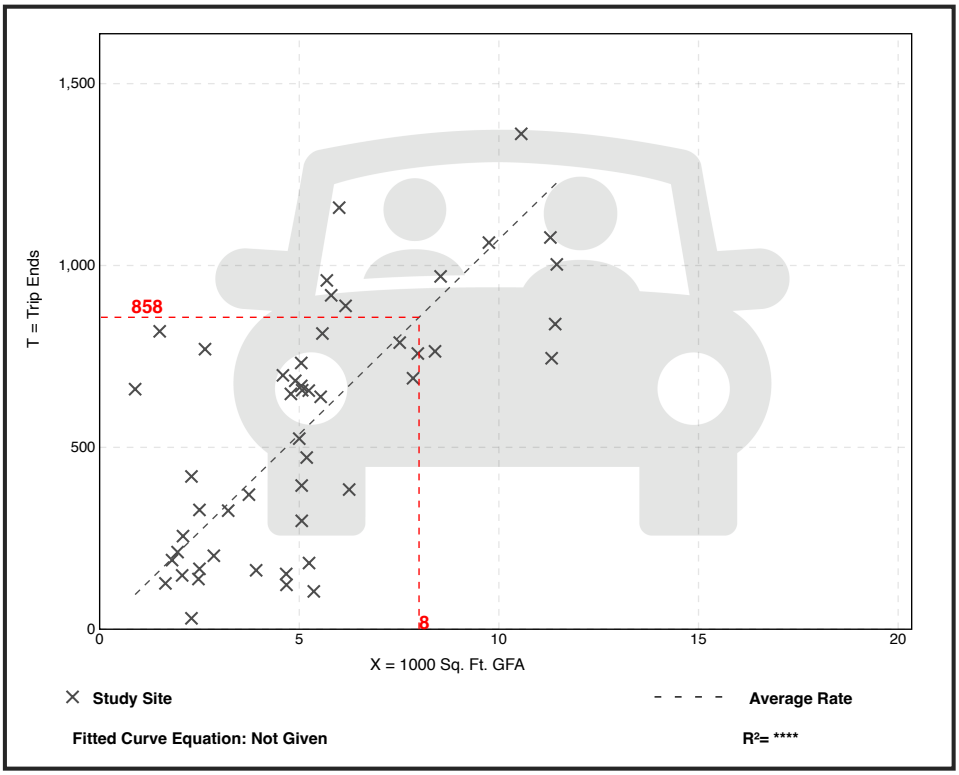
Vehicle Trip Ends vs: **1000 Sq. Ft. GFA**
On a: **Weekday**

Setting/Location: **General Urban/Suburban**
Number of Studies: 50
Avg. 1000 Sq. Ft. GFA: 5
Directional Distribution: 50% entering, 50% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
107.20	13.04 - 742.41	66.72

Data Plot and Equation



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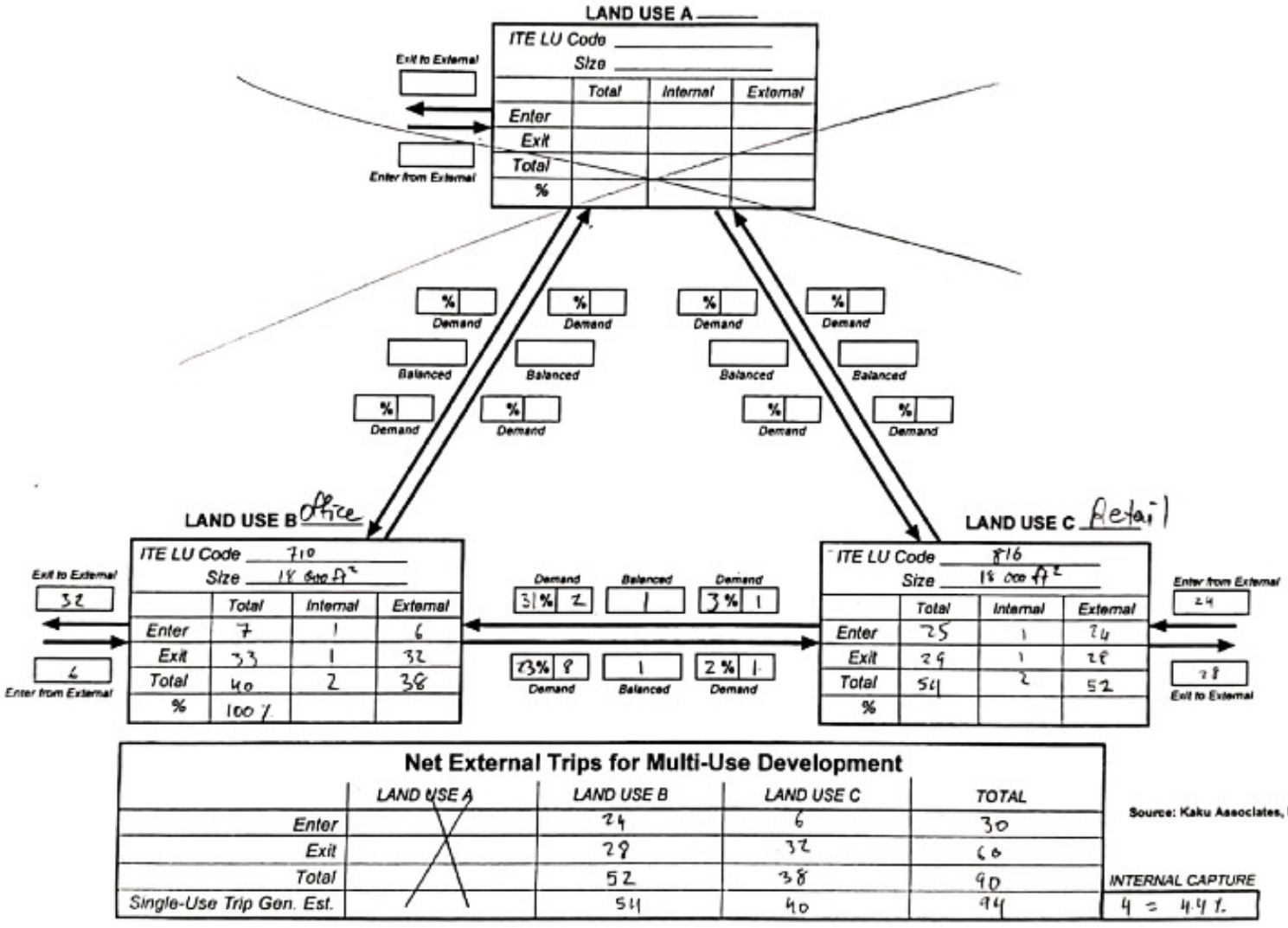
Analyst _____
Date _____

MULTI-USE DEVELOPMENT
TRIP GENERATION
AND INTERNAL CAPTURE SUMMARY

Name of Dvlpt _____
Time Period _____



Net External Trips for Multi-Use Development

	LAND USE A	LAND USE B	LAND USE C	TOTAL
Enter		24	6	30
Exit		29	32	60
Total		52	38	90
Single-Use Trip Gen. Est.		54	40	94

Source: Kaku Associates, Inc.

INTERNAL CAPTURE
4 = 4.4 %

APPENDIX D. ITE PARKING GENERATION REFERENCE

This appendix contains the ITE *Parking Generation* references used for parking generation. These are land use codes 710 (General Office Building), 816 (Hardware/Paint Store), and 932 (High-Turnover (Sit-Down) Restaurant).

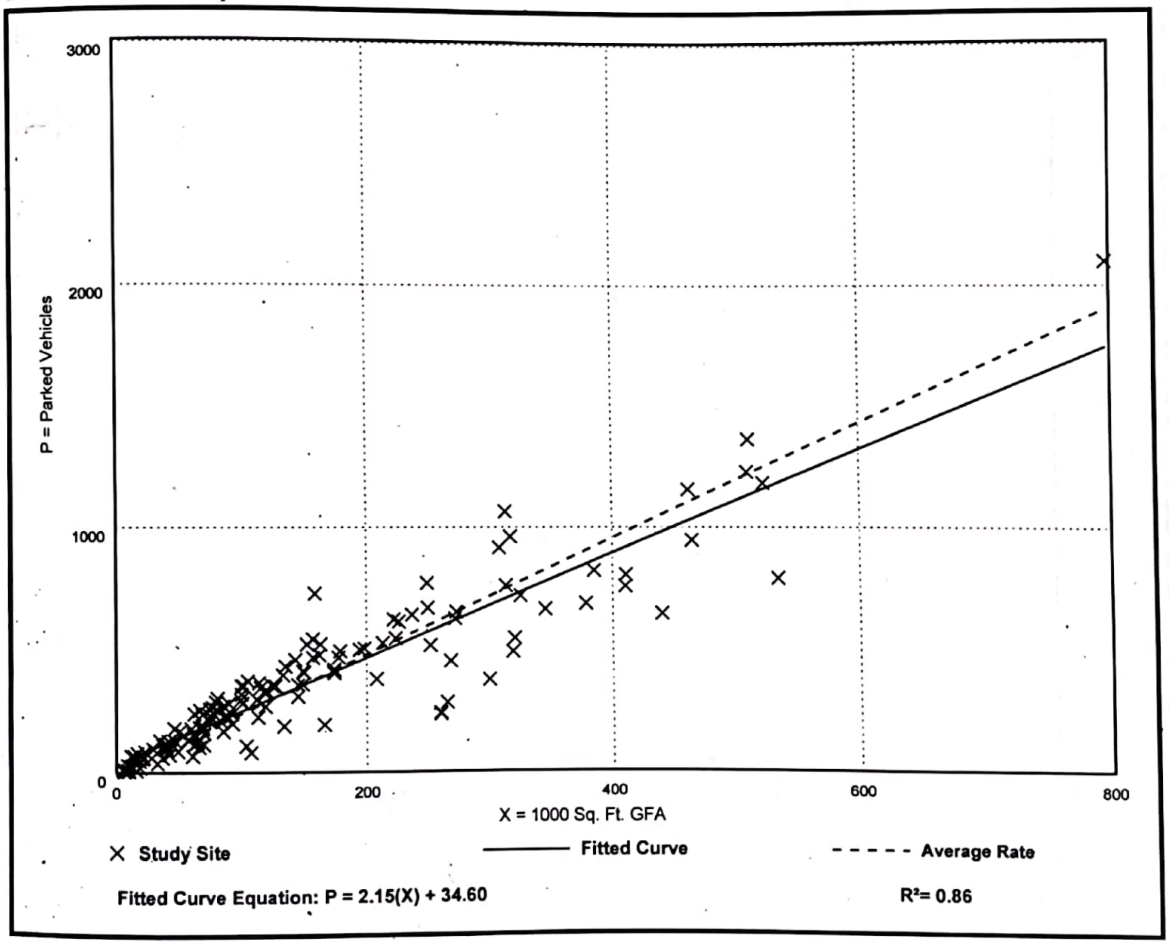
General Office Building
(710)

Peak Period Parking Demand vs: 1000 Sq. Ft. GFA
On a: Weekday (Monday - Friday)
Setting/Location: General Urban/Suburban
Peak Period of Parking Demand: 9:00 a.m. - 3:00 p.m.
Number of Studies: 148
Avg. 1000 Sq. Ft. GFA: 145

Peak Period Parking Demand per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	33rd / 85th Percentile	95% Confidence Interval	Standard Deviation (Coeff. of Variation)
2.39	0.50 - 5.58	2.30 / 3.30	2.28 - 2.50	0.69 (29%)

Data Plot and Equation



Hardware/Paint Store
(816)

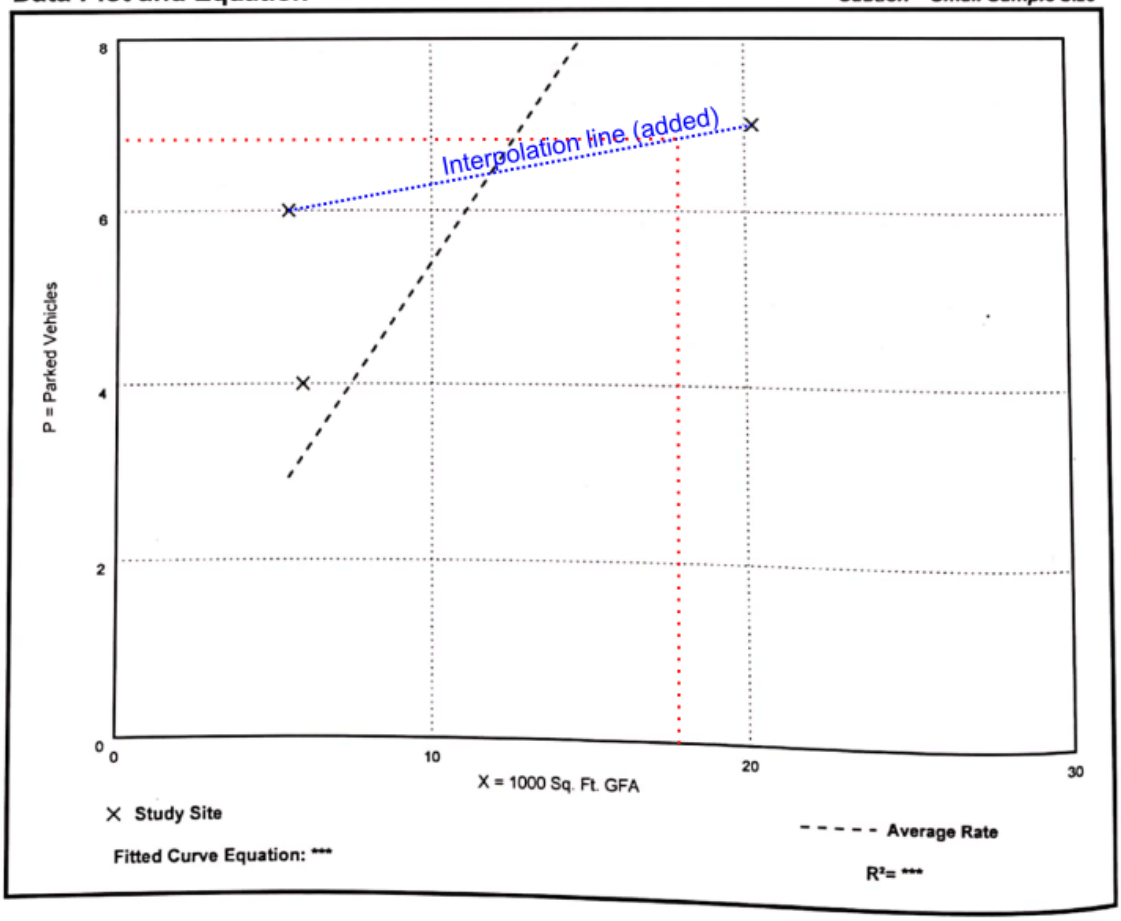
Peak Period Parking Demand vs: 1000 Sq. Ft. GFA
On a: Weekday (Monday - Friday)
Setting/Location: General Urban/Suburban
Peak Period of Parking Demand: 10:00 a.m. - 3:00 p.m.
Number of Studies: 3
Avg. 1000 Sq. Ft. GFA: 11

Peak Period Parking Demand per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	33rd / 85th Percentile	95% Confidence Interval	Standard Deviation (Coeff. of Variation)
0.54	0.35 - 1.11	0.45 / 1.11	***	0.35 (65%)

Data Plot and Equation

Caution – Small Sample Size



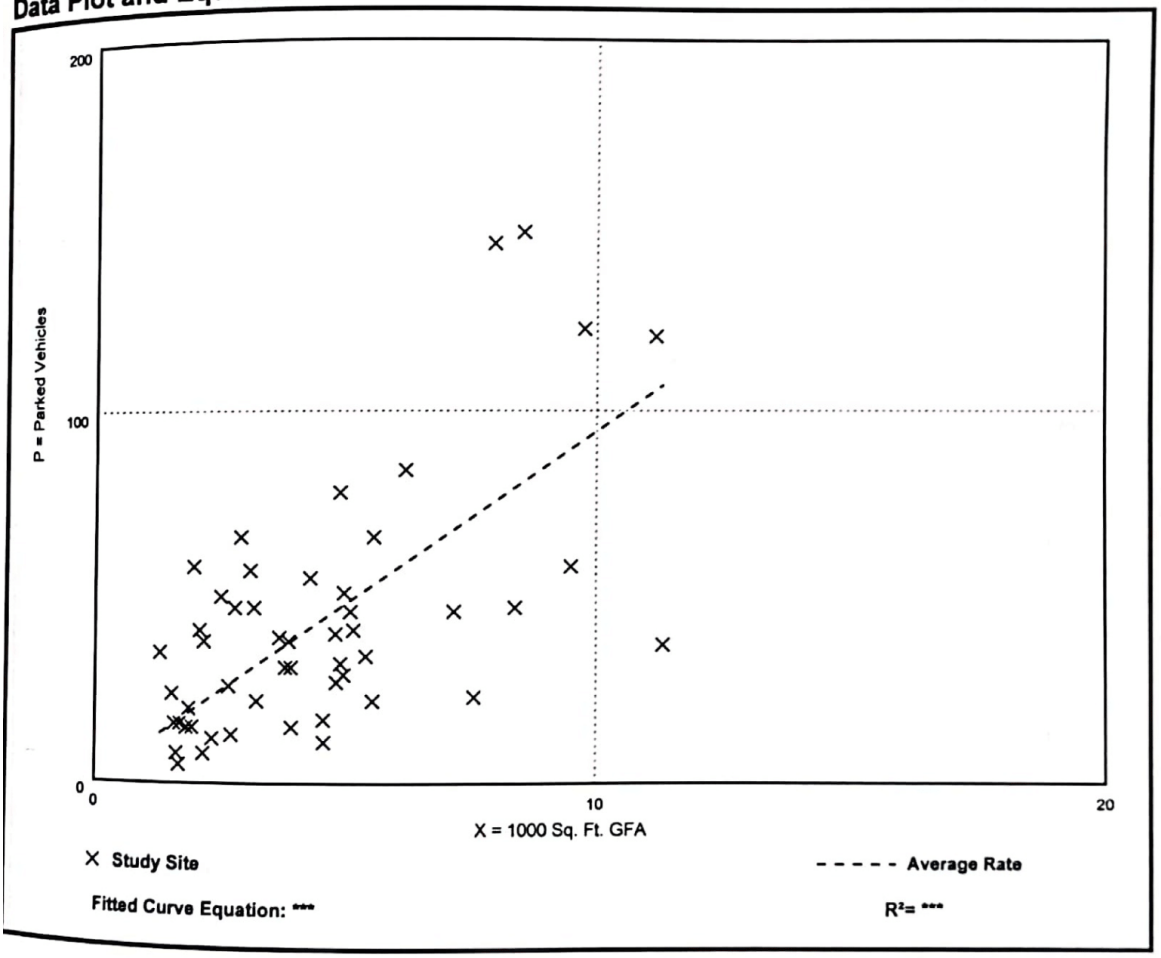
High-Turnover (Sit Down) Restaurant - Family (932)

Peak Period Parking Demand vs: 1000 Sq. Ft. GFA
On a: Weekday (Monday - Thursday)
Setting/Location: General Urban/Suburban
Peak Period of Parking Demand: 12:00 - 1:00 p.m.; 6:00 - 8:00 p.m.
Number of Studies: 51
Avg. 1000 Sq. Ft. GFA: 4.5

Peak Period Parking Demand per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	33rd / 85th Percentile	95% Confidence Interval	Standard Deviation (Coeff. of Variation)
9.44	2.35 - 27.78	6.39 / 17.40	7.96 - 10.92	5.38 (57%)

Data Plot and Equation



APPENDIX E. SYNCHRO LOS ANALYSIS (FUTURE)

This appendix contains the results of the Synchro LOS analysis for the road volumes, signal timings, and lane configurations at both the opening day (2024) and 5-year (2029) horizons.

OPENING DAY (NO-BUILD)

HCM 6th AWSC
7: Towne Centre Blvd & 1200 S 04/01/2023

Intersection	
Intersection Delay, s/veh	8.7
Intersection LOS	A

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	61	85	85	56	84	51
Future Vol, veh/h	61	85	85	56	84	51
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	66	92	92	61	91	55
Number of Lanes	1	1	1	0	1	2

Approach	WB	NB	SB
Opposing Approach		SB	NB
Opposing Lanes	0	3	1
Conflicting Approach Left	NB		WB
Conflicting Lanes Left	1	0	2
Conflicting Approach Right	SB	WB	
Conflicting Lanes Right	3	2	0
HCM Control Delay	8.5	9.1	8.4
HCM LOS	A	A	A

Lane	NBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
Vol Left, %	0%	100%	0%	100%	0%	0%
Vol Thru, %	60%	0%	0%	0%	100%	100%
Vol Right, %	40%	0%	100%	0%	0%	0%
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop
Traffic Vol by Lane	141	61	85	84	26	26
LT Vol	0	61	0	84	0	0
Through Vol	85	0	0	0	26	26
RT Vol	56	0	85	0	0	0
Lane Flow Rate	153	66	92	91	28	28
Geometry Grp	8	8	8	7	7	7
Degree of Util (X)	0.213	0.108	0.119	0.142	0.039	0.026
Departure Headway (Hd)	5.006	5.839	4.636	5.582	5.08	3.337
Convergence, Y/N	Yes	Yes	Yes	Yes	Yes	Yes
Cap	716	614	772	643	705	1070
Service Time	2.741	3.574	2.371	3.313	2.811	1.067
HCM Lane V/C Ratio	0.214	0.107	0.119	0.142	0.04	0.026
HCM Control Delay	9.1	9.3	8	9.2	8	6.2
HCM Lane LOS	A	A	A	A	A	A
HCM 95th-tile Q	0.8	0.4	0.4	0.5	0.1	0.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	49	58	174	257	86	145	249	943	104	61	1751	65
Future Volume (veh/h)	49	58	174	257	86	145	249	943	104	61	1751	65
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	53	63	189	279	93	158	271	1025	113	66	1903	71
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	88	472	210	322	712	318	315	3051	947	104	2739	850
Arrive On Green	0.03	0.13	0.13	0.09	0.20	0.20	0.09	0.60	0.60	0.06	1.00	1.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	3456	5106	1585	3456	5106	1585
Grp Volume(v), veh/h	53	63	189	279	93	158	271	1025	113	66	1903	71
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1728	1702	1585	1728	1702	1585
Q Serve(g_s), s	2.3	2.3	17.6	11.9	3.2	13.3	11.6	15.2	4.6	2.8	0.0	0.0
Cycle Q Clear(g_c), s	2.3	2.3	17.6	11.9	3.2	13.3	11.6	15.2	4.6	2.8	0.0	0.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	88	472	210	322	712	318	315	3051	947	104	2739	850
V/C Ratio(X)	0.60	0.13	0.90	0.87	0.13	0.50	0.86	0.34	0.12	0.64	0.69	0.08
Avail Cap(c_a), veh/h	346	877	391	346	877	391	415	3051	947	415	2739	850
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00
Upstream Filter(I)	0.93	0.93	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	72.3	57.4	64.1	67.1	49.2	53.3	67.2	15.2	13.1	69.7	0.0	0.0
Incr Delay (d2), s/veh	2.3	0.0	5.1	18.1	0.0	0.4	11.0	0.3	0.3	2.4	1.5	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	1.1	7.5	6.1	1.5	5.4	5.6	5.9	1.8	1.2	0.4	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	74.6	57.5	69.1	85.2	49.3	53.7	78.2	15.5	13.3	72.1	1.5	0.2
LnGrp LOS	E	E	E	F	D	D	E	B	B	E	A	A
Approach Vol, veh/h		305			530			1409			2040	
Approach Delay, s/veh		67.7			69.5			27.4			3.7	
Approach LOS		E			E			C			A	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.5	95.6	19.0	25.9	18.7	86.5	8.8	36.1
Change Period (Y+Rc), s	5.0	6.0	5.0	6.0	5.0	6.0	5.0	6.0
Max Green Setting (Gmax), s	18.0	58.0	15.0	37.0	18.0	58.0	15.0	37.0
Max Q Clear Time (g_c+I1), s	4.8	17.2	13.9	19.6	13.6	2.0	4.3	15.3
Green Ext Time (p_c), s	0.0	3.9	0.0	0.3	0.1	10.0	0.0	0.4

Intersection Summary	
HCM 6th Ctrl Delay	24.2
HCM 6th LOS	C

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	235	195	27	195	94	22
Future Volume (veh/h)	235	195	27	195	94	22
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	255	0	29	0	102	24
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	607		112		336	1183
Arrive On Green	0.18	0.00	0.06	0.00	0.10	0.33
Sat Flow, veh/h	3456	1585	1870	1585	3456	3647
Grp Volume(v), veh/h	255	0	29	0	102	24
Grp Sat Flow(s),veh/h/ln	1728	1585	1870	1585	1728	1777
Q Serve(g_s), s	1.9	0.0	0.4	0.0	0.8	0.1
Cycle Q Clear(g_c), s	1.9	0.0	0.4	0.0	0.8	0.1
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	607		112		336	1183
V/C Ratio(X)	0.42		0.26		0.30	0.02
Avail Cap(c_a), veh/h	2669		1576		2427	2995
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	1.00	1.00
Uniform Delay (d), s/veh	10.5	0.0	12.8	0.0	12.0	6.4
Incr Delay (d2), s/veh	0.2	0.0	0.4	0.0	0.2	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.5	0.0	0.2	0.0	0.2	0.0
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	10.6	0.0	13.2	0.0	12.1	6.4
LnGrp LOS	B		B		B	A
Approach Vol, veh/h	255		29			126
Approach Delay, s/veh	10.6		13.2			11.0
Approach LOS	B		B			B

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	7.8	7.7		13.0		15.5
Change Period (Y+Rc), s	5.0	6.0		8.0		6.0
Max Green Setting (Gmax), s	20.0	24.0		22.0		24.0
Max Q Clear Time (g_c+I1), s	2.8	2.4		3.9		2.1
Green Ext Time (p_c), s	0.1	0.1		0.4		0.0

Intersection Summary	
HCM 6th Ctrl Delay	10.9
HCM 6th LOS	B

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	63	17	107	61	26	30	55	1006	44	49	1640	64
Future Volume (veh/h)	63	17	107	61	26	30	55	1006	44	49	1640	64
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	68	18	116	66	28	33	60	1093	48	53	1783	70
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	134	185	156	155	72	88	255	3821	1186	469	3816	1185
Arrive On Green	0.10	0.10	0.10	0.10	0.10	0.10	0.06	1.00	1.00	0.03	0.75	0.75
Sat Flow, veh/h	1341	1870	1585	1101	730	890	1781	5106	1585	1781	5106	1585
Grp Volume(v), veh/h	68	18	116	70	0	57	60	1093	48	53	1783	70
Grp Sat Flow(s),veh/h/ln	1341	1870	1585	1180	0	1542	1781	1702	1585	1781	1702	1585
Q Serve(g_s), s	7.5	1.3	10.7	7.7	0.0	5.2	1.2	0.0	0.0	1.0	20.3	1.8
Cycle Q Clear(g_c), s	12.7	1.3	10.7	9.0	0.0	5.2	1.2	0.0	0.0	1.0	20.3	1.8
Prop In Lane	1.00		1.00	0.95		0.58	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	134	185	156	163	0	152	255	3821	1186	469	3816	1185
V/C Ratio(X)	0.51	0.10	0.74	0.43	0.00	0.38	0.24	0.29	0.04	0.11	0.47	0.06
Avail Cap(c_a), veh/h	243	337	285	267	0	278	325	3821	1186	541	3816	1185
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	69.2	61.5	65.7	65.5	0.0	63.3	5.4	0.0	0.0	3.8	7.4	5.0
Incr Delay (d2), s/veh	1.1	0.1	2.6	0.7	0.0	0.6	0.2	0.2	0.1	0.0	0.4	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.6	0.6	4.5	2.6	0.0	2.1	0.4	0.1	0.0	0.3	6.9	0.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	70.3	61.6	68.3	66.2	0.0	63.8	5.6	0.2	0.1	3.9	7.8	5.1
LnGrp LOS	E	E	E	E	A	E	A	A	A	A	A	A
Approach Vol, veh/h		202			127			1201			1906	
Approach Delay, s/veh		68.4			65.1			0.5			7.6	
Approach LOS		E			E			A			A	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	9.0	118.2		22.8	9.1	118.1		22.8				
Change Period (Y+Rc), s	4.5	6.0		8.0	4.5	6.0		8.0				
Max Green Setting (Gmax), s	10.5	94.0		27.0	10.5	94.0		27.0				
Max Q Clear Time (g_c+I1), s	3.0	2.0		14.7	3.2	22.3		11.0				
Green Ext Time (p_c), s	0.0	3.7		0.1	0.0	7.6		0.2				

Intersection Summary	
HCM 6th Ctrl Delay	10.8
HCM 6th LOS	B

OPENING DAY (BUILD)

HCM 6th AWSC
7: Towne Centre Blvd & 1200 S 04/01/2023

Intersection	
Intersection Delay, s/veh	8.8
Intersection LOS	A

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	61	89	85	64	87	51
Future Vol, veh/h	61	89	85	64	87	51
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	66	97	92	70	95	55
Number of Lanes	1	1	1	0	1	2

Approach	WB	NB	SB
Opposing Approach		SB	NB
Opposing Lanes	0	3	1
Conflicting Approach Left	NB		WB
Conflicting Lanes Left	1	0	2
Conflicting Approach Right	SB	WB	
Conflicting Lanes Right	3	2	0
HCM Control Delay	8.6	9.2	8.5
HCM LOS	A	A	A

Lane	NBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
Vol Left, %	0%	100%	0%	100%	0%	0%
Vol Thru, %	57%	0%	0%	0%	100%	100%
Vol Right, %	43%	0%	100%	0%	0%	0%
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop
Traffic Vol by Lane	149	61	89	87	26	26
LT Vol	0	61	0	87	0	0
Through Vol	85	0	0	0	26	26
RT Vol	64	0	89	0	0	0
Lane Flow Rate	162	66	97	95	28	28
Geometry Grp	8	8	8	7	7	7
Degree of Util (X)	0.225	0.108	0.125	0.147	0.039	0.026
Departure Headway (Hd)	5.003	5.871	4.668	5.602	5.099	3.356
Convergence, Y/N	Yes	Yes	Yes	Yes	Yes	Yes
Cap	717	611	766	640	702	1063
Service Time	2.737	3.608	2.405	3.334	2.831	1.087
HCM Lane V/C Ratio	0.226	0.108	0.127	0.148	0.04	0.026
HCM Control Delay	9.2	9.3	8.1	9.3	8	6.2
HCM Lane LOS	A	A	A	A	A	A
HCM 95th-tile Q	0.9	0.4	0.4	0.5	0.1	0.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	49	58	174	257	86	145	249	943	104	61	1751	65
Future Volume (veh/h)	49	58	174	257	86	145	249	943	104	61	1751	65
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	53	63	189	279	93	158	271	1025	113	66	1903	71
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	88	472	210	322	712	318	315	3051	947	104	2739	850
Arrive On Green	0.03	0.13	0.13	0.09	0.20	0.20	0.09	0.60	0.60	0.06	1.00	1.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	3456	5106	1585	3456	5106	1585
Grp Volume(v), veh/h	53	63	189	279	93	158	271	1025	113	66	1903	71
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1728	1702	1585	1728	1702	1585
Q Serve(g_s), s	2.3	2.3	17.6	11.9	3.2	13.3	11.6	15.2	4.6	2.8	0.0	0.0
Cycle Q Clear(g_c), s	2.3	2.3	17.6	11.9	3.2	13.3	11.6	15.2	4.6	2.8	0.0	0.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	88	472	210	322	712	318	315	3051	947	104	2739	850
V/C Ratio(X)	0.60	0.13	0.90	0.87	0.13	0.50	0.86	0.34	0.12	0.64	0.69	0.08
Avail Cap(c_a), veh/h	346	877	391	346	877	391	415	3051	947	415	2739	850
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00
Upstream Filter(I)	0.93	0.93	0.93	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	72.3	57.4	64.1	67.1	49.2	53.3	67.2	15.2	13.1	69.7	0.0	0.0
Incr Delay (d2), s/veh	2.3	0.0	5.1	18.1	0.0	0.4	11.0	0.3	0.3	2.4	1.5	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	1.1	7.5	6.1	1.5	5.4	5.6	5.9	1.8	1.2	0.4	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	74.6	57.5	69.1	85.2	49.3	53.7	78.2	15.5	13.3	72.1	1.5	0.2
LnGrp LOS	E	E	E	F	D	D	E	B	B	E	A	A
Approach Vol, veh/h		305			530			1409			2040	
Approach Delay, s/veh		67.7			69.5			27.4			3.7	
Approach LOS		E			E			C			A	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.5	95.6	19.0	25.9	18.7	86.5	8.8	36.1
Change Period (Y+Rc), s	5.0	6.0	5.0	6.0	5.0	6.0	5.0	6.0
Max Green Setting (Gmax), s	18.0	58.0	15.0	37.0	18.0	58.0	15.0	37.0
Max Q Clear Time (g_c+I1), s	4.8	17.2	13.9	19.6	13.6	2.0	4.3	15.3
Green Ext Time (p_c), s	0.0	3.9	0.0	0.3	0.1	10.0	0.0	0.4

Intersection Summary	
HCM 6th Ctrl Delay	24.2
HCM 6th LOS	C

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	235	195	27	195	94	22
Future Volume (veh/h)	235	195	27	195	94	22
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	255	0	29	0	102	24
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	607		112		336	1183
Arrive On Green	0.18	0.00	0.06	0.00	0.10	0.33
Sat Flow, veh/h	3456	1585	1870	1585	3456	3647
Grp Volume(v), veh/h	255	0	29	0	102	24
Grp Sat Flow(s),veh/h/ln	1728	1585	1870	1585	1728	1777
Q Serve(g_s), s	1.9	0.0	0.4	0.0	0.8	0.1
Cycle Q Clear(g_c), s	1.9	0.0	0.4	0.0	0.8	0.1
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	607		112		336	1183
V/C Ratio(X)	0.42		0.26		0.30	0.02
Avail Cap(c_a), veh/h	2669		1576		2427	2995
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	1.00	1.00
Uniform Delay (d), s/veh	10.5	0.0	12.8	0.0	12.0	6.4
Incr Delay (d2), s/veh	0.2	0.0	0.4	0.0	0.2	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.5	0.0	0.2	0.0	0.2	0.0
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	10.6	0.0	13.2	0.0	12.1	6.4
LnGrp LOS	B		B		B	A
Approach Vol, veh/h	255		29			126
Approach Delay, s/veh	10.6		13.2			11.0
Approach LOS	B		B			B

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	7.8	7.7		13.0		15.5
Change Period (Y+Rc), s	5.0	6.0		8.0		6.0
Max Green Setting (Gmax), s	20.0	24.0		22.0		24.0
Max Q Clear Time (g_c+I1), s	2.8	2.4		3.9		2.1
Green Ext Time (p_c), s	0.1	0.1		0.4		0.0

Intersection Summary	
HCM 6th Ctrl Delay	10.9
HCM 6th LOS	B

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	63	17	107	61	26	30	55	1006	44	49	1640	64
Future Volume (veh/h)	63	17	107	61	26	30	55	1006	44	49	1640	64
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	68	18	116	66	28	33	60	1093	48	53	1783	70
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	134	185	156	155	72	88	255	3821	1186	469	3816	1185
Arrive On Green	0.10	0.10	0.10	0.10	0.10	0.10	0.06	1.00	1.00	0.03	0.75	0.75
Sat Flow, veh/h	1341	1870	1585	1101	730	890	1781	5106	1585	1781	5106	1585
Grp Volume(v), veh/h	68	18	116	70	0	57	60	1093	48	53	1783	70
Grp Sat Flow(s),veh/h/ln	1341	1870	1585	1180	0	1542	1781	1702	1585	1781	1702	1585
Q Serve(g_s), s	7.5	1.3	10.7	7.7	0.0	5.2	1.2	0.0	0.0	1.0	20.3	1.8
Cycle Q Clear(g_c), s	12.7	1.3	10.7	9.0	0.0	5.2	1.2	0.0	0.0	1.0	20.3	1.8
Prop In Lane	1.00		1.00	0.95		0.58	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	134	185	156	163	0	152	255	3821	1186	469	3816	1185
V/C Ratio(X)	0.51	0.10	0.74	0.43	0.00	0.38	0.24	0.29	0.04	0.11	0.47	0.06
Avail Cap(c_a), veh/h	243	337	285	267	0	278	325	3821	1186	541	3816	1185
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	69.2	61.5	65.7	65.5	0.0	63.3	5.4	0.0	0.0	3.8	7.4	5.0
Incr Delay (d2), s/veh	1.1	0.1	2.6	0.7	0.0	0.6	0.2	0.2	0.1	0.0	0.4	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.6	0.6	4.5	2.6	0.0	2.1	0.4	0.1	0.0	0.3	6.9	0.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	70.3	61.6	68.3	66.2	0.0	63.8	5.6	0.2	0.1	3.9	7.8	5.1
LnGrp LOS	E	E	E	E	A	E	A	A	A	A	A	A
Approach Vol, veh/h		202			127			1201			1906	
Approach Delay, s/veh		68.4			65.1			0.5			7.6	
Approach LOS		E			E			A			A	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	9.0	118.2		22.8	9.1	118.1		22.8				
Change Period (Y+Rc), s	4.5	6.0		8.0	4.5	6.0		8.0				
Max Green Setting (Gmax), s	10.5	94.0		27.0	10.5	94.0		27.0				
Max Q Clear Time (g_c+I1), s	3.0	2.0		14.7	3.2	22.3		11.0				
Green Ext Time (p_c), s	0.0	3.7		0.1	0.0	7.6		0.2				

Intersection Summary	
HCM 6th Ctrl Delay	10.8
HCM 6th LOS	B

Intersection	
Int Delay, s/veh	0.8

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	7	38	29	1112	1832	34
Future Vol, veh/h	7	38	29	1112	1832	34
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	-	0	0	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	8	41	32	1209	1991	37

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	2558	1014	2028	0	-	0
Stage 1	2010	-	-	-	-	-
Stage 2	548	-	-	-	-	-
Critical Hdwy	5.74	7.14	5.34	-	-	-
Critical Hdwy Stg 1	6.64	-	-	-	-	-
Critical Hdwy Stg 2	6.04	-	-	-	-	-
Follow-up Hdwy	3.82	3.92	3.12	-	-	-
Pot Cap-1 Maneuver	46	203	121	-	-	-
Stage 1	56	-	-	-	-	-
Stage 2	496	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	34	203	121	-	-	-
Mov Cap-2 Maneuver	37	-	-	-	-	-
Stage 1	41	-	-	-	-	-
Stage 2	496	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	27.2	1.1	0
HCM LOS	D		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	121	-	203	-	-
HCM Lane V/C Ratio	0.261	-	0.203	-	-
HCM Control Delay (s)	44.9	-	27.2	-	-
HCM Lane LOS	E	-	D	-	-
HCM 95th %tile Q(veh)	1	-	0.7	-	-

Intersection	
Int Delay, s/veh	1.2

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	142	11	0	146	0	43
Future Vol, veh/h	142	11	0	146	0	43
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	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	154	12	0	159	0	47

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	-	-	-	83
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-
Critical Hdwy	-	-	-	-	-	7.14
Critical Hdwy Stg 1	-	-	-	-	-	-
Critical Hdwy Stg 2	-	-	-	-	-	-
Follow-up Hdwy	-	-	-	-	-	3.92
Pot Cap-1 Maneuver	-	-	0	-	0	815
Stage 1	-	-	0	-	0	-
Stage 2	-	-	0	-	0	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	-	-	-	815
Mov Cap-2 Maneuver	-	-	-	-	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0	9.7
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBT
Capacity (veh/h)	815	-	-	-
HCM Lane V/C Ratio	0.057	-	-	-
HCM Control Delay (s)	9.7	-	-	-
HCM Lane LOS	A	-	-	-
HCM 95th %tile Q(veh)	0.2	-	-	-

YEAR 5 (NO-BUILD)

HCM 6th AWSC
7: Towne Centre Blvd & 1200 S 04/01/2023

Intersection	
Intersection Delay, s/veh	8.9
Intersection LOS	A

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	68	93	93	62	92	56
Future Vol, veh/h	68	93	93	62	92	56
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	74	101	101	67	100	61
Number of Lanes	1	1	1	0	1	2

Approach	WB	NB	SB
Opposing Approach		SB	NB
Opposing Lanes	0	3	1
Conflicting Approach Left	NB		WB
Conflicting Lanes Left	1	0	2
Conflicting Approach Right	SB	WB	
Conflicting Lanes Right	3	2	0
HCM Control Delay	8.7	9.4	8.5
HCM LOS	A	A	A

Lane	NBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
Vol Left, %	0%	100%	0%	100%	0%	0%
Vol Thru, %	60%	0%	0%	0%	100%	100%
Vol Right, %	40%	0%	100%	0%	0%	0%
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop
Traffic Vol by Lane	155	68	93	92	28	28
LT Vol	0	68	0	92	0	0
Through Vol	93	0	0	0	28	28
RT Vol	62	0	93	0	0	0
Lane Flow Rate	168	74	101	100	30	30
Geometry Grp	8	8	8	7	7	7
Degree of Util (X)	0.238	0.122	0.133	0.157	0.043	0.029
Departure Headway (Hd)	5.084	5.922	4.719	5.647	5.144	3.4
Convergence, Y/N	Yes	Yes	Yes	Yes	Yes	Yes
Cap	704	605	757	635	695	1048
Service Time	2.827	3.665	2.462	3.386	2.883	1.138
HCM Lane V/C Ratio	0.239	0.122	0.133	0.157	0.043	0.029
HCM Control Delay	9.4	9.5	8.2	9.4	8.1	6.2
HCM Lane LOS	A	A	A	A	A	A
HCM 95th-tile Q	0.9	0.4	0.5	0.6	0.1	0.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	54	64	193	284	94	160	275	1039	115	67	1934	71
Future Volume (veh/h)	54	64	193	284	94	160	275	1039	115	67	1934	71
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	59	70	210	309	102	174	299	1129	125	73	2102	77
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	96	518	231	346	775	346	342	2936	911	112	2596	806
Arrive On Green	0.03	0.15	0.15	0.10	0.22	0.22	0.10	0.57	0.57	0.06	1.00	1.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	3456	5106	1585	3456	5106	1585
Grp Volume(v), veh/h	59	70	210	309	102	174	299	1129	125	73	2102	77
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1728	1702	1585	1728	1702	1585
Q Serve(g_s), s	2.5	2.6	19.6	13.3	3.5	14.5	12.8	18.1	5.5	3.1	0.0	0.0
Cycle Q Clear(g_c), s	2.5	2.6	19.6	13.3	3.5	14.5	12.8	18.1	5.5	3.1	0.0	0.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	96	518	231	346	775	346	342	2936	911	112	2596	806
V/C Ratio(X)	0.62	0.14	0.91	0.89	0.13	0.50	0.87	0.38	0.14	0.65	0.81	0.10
Avail Cap(c_a), veh/h	346	877	391	346	877	391	415	2936	911	415	2596	806
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00
Upstream Filter(I)	0.91	0.91	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	72.1	55.8	63.1	66.7	47.2	51.5	66.7	17.4	14.7	69.3	0.0	0.0
Incr Delay (d2), s/veh	2.2	0.0	8.6	23.7	0.0	0.4	14.2	0.4	0.3	2.4	2.9	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.2	1.2	8.5	7.0	1.6	5.9	6.3	7.2	2.1	1.4	0.7	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	74.3	55.9	71.7	90.4	47.2	51.9	80.8	17.8	15.0	71.7	2.9	0.2
LnGrp LOS	E	E	E	F	D	D	F	B	B	E	A	A
Approach Vol, veh/h		339			585			1553			2252	
Approach Delay, s/veh		68.9			71.4			29.7			5.0	
Approach LOS		E			E			C			A	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.9	92.2	20.0	27.9	19.9	82.3	9.2	38.7
Change Period (Y+Rc), s	5.0	6.0	5.0	6.0	5.0	6.0	5.0	6.0
Max Green Setting (Gmax), s	18.0	58.0	15.0	37.0	18.0	58.0	15.0	37.0
Max Q Clear Time (g_c+I1), s	5.1	20.1	15.3	21.6	14.8	2.0	4.5	16.5
Green Ext Time (p_c), s	0.0	4.4	0.0	0.3	0.1	12.1	0.0	0.4

Intersection Summary	
HCM 6th Ctrl Delay	25.9
HCM 6th LOS	C

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	259	170	30	215	104	25
Future Volume (veh/h)	259	170	30	215	104	25
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	282	0	33	0	113	27
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	598		124		357	1217
Arrive On Green	0.17	0.00	0.07	0.00	0.10	0.34
Sat Flow, veh/h	3456	1585	1870	1585	3456	3647
Grp Volume(v), veh/h	282	0	33	0	113	27
Grp Sat Flow(s),veh/h/ln	1728	1585	1870	1585	1728	1777
Q Serve(g_s), s	2.1	0.0	0.5	0.0	0.9	0.1
Cycle Q Clear(g_c), s	2.1	0.0	0.5	0.0	0.9	0.1
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	598		124		357	1217
V/C Ratio(X)	0.47		0.27		0.32	0.02
Avail Cap(c_a), veh/h	2631		1554		2392	2952
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	1.00	1.00
Uniform Delay (d), s/veh	10.8	0.0	12.8	0.0	12.0	6.3
Incr Delay (d2), s/veh	0.2	0.0	0.4	0.0	0.2	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.6	0.0	0.2	0.0	0.3	0.0
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	11.0	0.0	13.2	0.0	12.2	6.3
LnGrp LOS	B		B		B	A
Approach Vol, veh/h	282		33			140
Approach Delay, s/veh	11.0		13.2			11.1
Approach LOS	B		B			B

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	8.0	7.9		13.0		15.9
Change Period (Y+Rc), s	5.0	6.0		8.0		6.0
Max Green Setting (Gmax), s	20.0	24.0		22.0		24.0
Max Q Clear Time (g_c+I1), s	2.9	2.5		4.1		2.1
Green Ext Time (p_c), s	0.2	0.1		0.5		0.1

Intersection Summary	
HCM 6th Ctrl Delay	11.2
HCM 6th LOS	B

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	66	19	117	68	28	33	61	1111	48	54	1829	71
Future Volume (veh/h)	66	19	117	68	28	33	61	1111	48	54	1829	71
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	72	21	127	74	30	36	66	1208	52	59	1988	77
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	138	198	168	164	75	92	219	3781	1174	428	3777	1172
Arrive On Green	0.11	0.11	0.11	0.11	0.11	0.11	0.06	1.00	1.00	0.03	0.74	0.74
Sat Flow, veh/h	1335	1870	1585	1100	713	871	1781	5106	1585	1781	5106	1585
Grp Volume(v), veh/h	72	21	127	76	0	64	66	1208	52	59	1988	77
Grp Sat Flow(s),veh/h/ln	1335	1870	1585	1138	0	1545	1781	1702	1585	1781	1702	1585
Q Serve(g_s), s	8.0	1.5	11.7	8.7	0.0	5.8	1.3	0.0	0.0	1.2	24.9	2.0
Cycle Q Clear(g_c), s	13.8	1.5	11.7	10.2	0.0	5.8	1.3	0.0	0.0	1.2	24.9	2.0
Prop In Lane	1.00		1.00	0.97		0.56	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	138	198	168	168	0	163	219	3781	1174	428	3777	1172
V/C Ratio(X)	0.52	0.11	0.76	0.45	0.00	0.39	0.30	0.32	0.04	0.14	0.53	0.07
Avail Cap(c_a), veh/h	237	337	285	261	0	278	289	3781	1174	498	3777	1172
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	69.0	60.7	65.2	65.2	0.0	62.6	6.9	0.0	0.0	4.1	8.3	5.3
Incr Delay (d2), s/veh	1.1	0.1	2.6	0.7	0.0	0.6	0.3	0.2	0.1	0.1	0.5	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.8	0.7	4.9	2.9	0.0	2.3	0.4	0.1	0.0	0.4	8.6	0.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	70.1	60.7	67.8	65.9	0.0	63.1	7.2	0.2	0.1	4.2	8.9	5.4
LnGrp LOS	E	E	E	E	A	E	A	A	A	A	A	A
Approach Vol, veh/h		220			140			1326			2124	
Approach Delay, s/veh		67.9			64.7			0.6			8.6	
Approach LOS		E			E			A			A	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	9.1	117.1		23.9	9.2	117.0		23.9				
Change Period (Y+Rc), s	4.5	6.0		8.0	4.5	6.0		8.0				
Max Green Setting (Gmax), s	10.5	94.0		27.0	10.5	94.0		27.0				
Max Q Clear Time (g_c+I1), s	3.2	2.0		15.8	3.3	26.9		12.2				
Green Ext Time (p_c), s	0.0	4.2		0.1	0.0	9.3		0.2				

Intersection Summary												
HCM 6th Ctrl Delay			11.3									
HCM 6th LOS			B									

YEAR 5 (BUILD)

HCM 6th AWSC
7: Towne Centre Blvd & 1200 S 04/01/2023

Intersection	
Intersection Delay, s/veh	9
Intersection LOS	A

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	68	97	93	70	95	56
Future Vol, veh/h	68	97	93	70	95	56
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	74	105	101	76	103	61
Number of Lanes	1	1	1	0	1	2

Approach	WB	NB	SB
Opposing Approach		SB	NB
Opposing Lanes	0	3	1
Conflicting Approach Left	NB		WB
Conflicting Lanes Left	1	0	2
Conflicting Approach Right	SB	WB	
Conflicting Lanes Right	3	2	0
HCM Control Delay	8.8	9.5	8.6
HCM LOS	A	A	A

Lane	NBLn1	WBLn1	WBLn2	SBLn1	SBLn2	SBLn3
Vol Left, %	0%	100%	0%	100%	0%	0%
Vol Thru, %	57%	0%	0%	0%	100%	100%
Vol Right, %	43%	0%	100%	0%	0%	0%
Sign Control	Stop	Stop	Stop	Stop	Stop	Stop
Traffic Vol by Lane	163	68	97	95	28	28
LT Vol	0	68	0	95	0	0
Through Vol	93	0	0	0	28	28
RT Vol	70	0	97	0	0	0
Lane Flow Rate	177	74	105	103	30	30
Geometry Grp	8	8	8	7	7	7
Degree of Util (X)	0.25	0.122	0.139	0.163	0.044	0.029
Departure Headway (Hd)	5.083	5.954	4.75	5.667	5.164	3.419
Convergence, Y/N	Yes	Yes	Yes	Yes	Yes	Yes
Cap	705	601	752	632	692	1041
Service Time	2.829	3.702	2.498	3.406	2.903	1.158
HCM Lane V/C Ratio	0.251	0.123	0.14	0.163	0.043	0.029
HCM Control Delay	9.5	9.5	8.3	9.5	8.1	6.3
HCM Lane LOS	A	A	A	A	A	A
HCM 95th-tile Q	1	0.4	0.5	0.6	0.1	0.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	54	64	193	284	94	160	275	1039	115	67	1934	71
Future Volume (veh/h)	54	64	193	284	94	160	275	1039	115	67	1934	71
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	59	70	210	309	102	174	299	1129	125	73	2102	77
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	96	518	231	346	775	346	342	2936	911	112	2596	806
Arrive On Green	0.03	0.15	0.15	0.10	0.22	0.22	0.10	0.57	0.57	0.06	1.00	1.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	3456	5106	1585	3456	5106	1585
Grp Volume(v), veh/h	59	70	210	309	102	174	299	1129	125	73	2102	77
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1728	1702	1585	1728	1702	1585
Q Serve(g_s), s	2.5	2.6	19.6	13.3	3.5	14.5	12.8	18.1	5.5	3.1	0.0	0.0
Cycle Q Clear(g_c), s	2.5	2.6	19.6	13.3	3.5	14.5	12.8	18.1	5.5	3.1	0.0	0.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	96	518	231	346	775	346	342	2936	911	112	2596	806
V/C Ratio(X)	0.62	0.14	0.91	0.89	0.13	0.50	0.87	0.38	0.14	0.65	0.81	0.10
Avail Cap(c_a), veh/h	346	877	391	346	877	391	415	2936	911	415	2596	806
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00
Upstream Filter(I)	0.91	0.91	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	72.1	55.8	63.1	66.7	47.2	51.5	66.7	17.4	14.7	69.3	0.0	0.0
Incr Delay (d2), s/veh	2.2	0.0	8.6	23.7	0.0	0.4	14.2	0.4	0.3	2.4	2.9	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.2	1.2	8.5	7.0	1.6	5.9	6.3	7.2	2.1	1.4	0.7	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	74.3	55.9	71.7	90.4	47.2	51.9	80.8	17.8	15.0	71.7	2.9	0.2
LnGrp LOS	E	E	E	F	D	D	F	B	B	E	A	A
Approach Vol, veh/h		339			585			1553			2252	
Approach Delay, s/veh		68.9			71.4			29.7			5.0	
Approach LOS		E			E			C			A	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	9.9	92.2	20.0	27.9	19.9	82.3	9.2	38.7
Change Period (Y+Rc), s	5.0	6.0	5.0	6.0	5.0	6.0	5.0	6.0
Max Green Setting (Gmax), s	18.0	58.0	15.0	37.0	18.0	58.0	15.0	37.0
Max Q Clear Time (g_c+I1), s	5.1	20.1	15.3	21.6	14.8	2.0	4.5	16.5
Green Ext Time (p_c), s	0.0	4.4	0.0	0.3	0.1	12.1	0.0	0.4

Intersection Summary	
HCM 6th Ctrl Delay	25.9
HCM 6th LOS	C

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (veh/h)	259	170	30	215	104	25
Future Volume (veh/h)	259	170	30	215	104	25
Initial Q (Qb), veh	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00	
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach	No		No			No
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	282	0	33	0	113	27
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2
Cap, veh/h	598		124		357	1217
Arrive On Green	0.17	0.00	0.07	0.00	0.10	0.34
Sat Flow, veh/h	3456	1585	1870	1585	3456	3647
Grp Volume(v), veh/h	282	0	33	0	113	27
Grp Sat Flow(s),veh/h/ln	1728	1585	1870	1585	1728	1777
Q Serve(g_s), s	2.1	0.0	0.5	0.0	0.9	0.1
Cycle Q Clear(g_c), s	2.1	0.0	0.5	0.0	0.9	0.1
Prop In Lane	1.00	1.00		1.00	1.00	
Lane Grp Cap(c), veh/h	598		124		357	1217
V/C Ratio(X)	0.47		0.27		0.32	0.02
Avail Cap(c_a), veh/h	2631		1554		2392	2952
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.00	1.00	1.00
Uniform Delay (d), s/veh	10.8	0.0	12.8	0.0	12.0	6.3
Incr Delay (d2), s/veh	0.2	0.0	0.4	0.0	0.2	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.6	0.0	0.2	0.0	0.3	0.0
Unsig. Movement Delay, s/veh						
LnGrp Delay(d),s/veh	11.0	0.0	13.2	0.0	12.2	6.3
LnGrp LOS	B		B		B	A
Approach Vol, veh/h	282		33			140
Approach Delay, s/veh	11.0		13.2			11.1
Approach LOS	B		B			B

Timer - Assigned Phs	1	2		4		6
Phs Duration (G+Y+Rc), s	8.0	7.9		13.0		15.9
Change Period (Y+Rc), s	5.0	6.0		8.0		6.0
Max Green Setting (Gmax), s	20.0	24.0		22.0		24.0
Max Q Clear Time (g_c+I1), s	2.9	2.5		4.1		2.1
Green Ext Time (p_c), s	0.2	0.1		0.5		0.1

Intersection Summary	
HCM 6th Ctrl Delay	11.2
HCM 6th LOS	B

Notes

Unsignalized Delay for [NBR, WBR] is excluded from calculations of the approach delay and intersection delay.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	66	19	117	68	28	33	61	1111	48	54	1829	71
Future Volume (veh/h)	66	19	117	68	28	33	61	1111	48	54	1829	71
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	72	21	127	74	30	36	66	1208	52	59	1988	77
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
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	138	198	168	164	75	92	219	3781	1174	428	3777	1172
Arrive On Green	0.11	0.11	0.11	0.11	0.11	0.11	0.06	1.00	1.00	0.03	0.74	0.74
Sat Flow, veh/h	1335	1870	1585	1100	713	871	1781	5106	1585	1781	5106	1585
Grp Volume(v), veh/h	72	21	127	76	0	64	66	1208	52	59	1988	77
Grp Sat Flow(s),veh/h/ln	1335	1870	1585	1138	0	1545	1781	1702	1585	1781	1702	1585
Q Serve(g_s), s	8.0	1.5	11.7	8.7	0.0	5.8	1.3	0.0	0.0	1.2	24.9	2.0
Cycle Q Clear(g_c), s	13.8	1.5	11.7	10.2	0.0	5.8	1.3	0.0	0.0	1.2	24.9	2.0
Prop In Lane	1.00		1.00	0.97		0.56	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	138	198	168	168	0	163	219	3781	1174	428	3777	1172
V/C Ratio(X)	0.52	0.11	0.76	0.45	0.00	0.39	0.30	0.32	0.04	0.14	0.53	0.07
Avail Cap(c_a), veh/h	237	337	285	261	0	278	289	3781	1174	498	3777	1172
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	2.00	2.00	2.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	69.0	60.7	65.2	65.2	0.0	62.6	6.9	0.0	0.0	4.1	8.3	5.3
Incr Delay (d2), s/veh	1.1	0.1	2.6	0.7	0.0	0.6	0.3	0.2	0.1	0.1	0.5	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.8	0.7	4.9	2.9	0.0	2.3	0.4	0.1	0.0	0.4	8.6	0.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	70.1	60.7	67.8	65.9	0.0	63.1	7.2	0.2	0.1	4.2	8.9	5.4
LnGrp LOS	E	E	E	E	A	E	A	A	A	A	A	A
Approach Vol, veh/h		220			140			1326			2124	
Approach Delay, s/veh		67.9			64.7			0.6			8.6	
Approach LOS		E			E			A			A	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	9.1	117.1		23.9	9.2	117.0		23.9				
Change Period (Y+Rc), s	4.5	6.0		8.0	4.5	6.0		8.0				
Max Green Setting (Gmax), s	10.5	94.0		27.0	10.5	94.0		27.0				
Max Q Clear Time (g_c+I1), s	3.2	2.0		15.8	3.3	26.9		12.2				
Green Ext Time (p_c), s	0.0	4.2		0.1	0.0	9.3		0.2				

Intersection Summary			
HCM 6th Ctrl Delay		11.3	
HCM 6th LOS		B	

Intersection	
Int Delay, s/veh	1

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	7	38	29	1228	2021	34
Future Vol, veh/h	7	38	29	1228	2021	34
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	-	0	100	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	8	41	32	1335	2197	37

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	2814	1117	2234	0	-	0
Stage 1	2216	-	-	-	-	-
Stage 2	598	-	-	-	-	-
Critical Hdwy	5.74	7.14	5.34	-	-	-
Critical Hdwy Stg 1	6.64	-	-	-	-	-
Critical Hdwy Stg 2	6.04	-	-	-	-	-
Follow-up Hdwy	3.82	3.92	3.12	-	-	-
Pot Cap-1 Maneuver	33	173	95	-	-	-
Stage 1	41	-	-	-	-	-
Stage 2	467	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	22	173	95	-	-	-
Mov Cap-2 Maneuver	25	-	-	-	-	-
Stage 1	27	-	-	-	-	-
Stage 2	467	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	32.2	1.4	0
HCM LOS	D		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	95	-	173	-	-
HCM Lane V/C Ratio	0.332	-	0.239	-	-
HCM Control Delay (s)	60.7	-	32.2	-	-
HCM Lane LOS	F	-	D	-	-
HCM 95th %tile Q(veh)	1.3	-	0.9	-	-

Intersection	
Int Delay, s/veh	1.1

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	157	11	0	161	0	43
Future Vol, veh/h	157	11	0	161	0	43
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	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	171	12	0	175	0	47

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	-	-	-	92
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-
Critical Hdwy	-	-	-	-	-	7.14
Critical Hdwy Stg 1	-	-	-	-	-	-
Critical Hdwy Stg 2	-	-	-	-	-	-
Follow-up Hdwy	-	-	-	-	-	3.92
Pot Cap-1 Maneuver	-	-	0	-	0	804
Stage 1	-	-	0	-	0	-
Stage 2	-	-	0	-	0	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	-	-	-	804
Mov Cap-2 Maneuver	-	-	-	-	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-

Approach	EB	WB	NB
HCM Control Delay, s	0	0	9.8
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBT
Capacity (veh/h)	804	-	-	-
HCM Lane V/C Ratio	0.058	-	-	-
HCM Control Delay (s)	9.8	-	-	-
HCM Lane LOS	A	-	-	-
HCM 95th %tile Q(veh)	0.2	-	-	-