

## NON-TRADITIONAL USE OF EMME MODEL

### Incorporating Traffic Operations into a Demand Forecasting Model

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### ABSTRACT

While transportation planning models incorporate some road operational elements; their use in producing detailed traffic forecasts at the intersection level are limited. Recent development efforts took into account approaches that increase consistency and reasonableness of traffic assignment results. Many of these focused on using different volume delay functions, modifying the coefficients of the volume delay functions utilized.

Recently a detailed traffic model was developed for the downtown Kelowna area that incorporated detailed travel demand forecasting, traffic simulation, and capacity analysis. This paper focuses on the detailed travel demand sub-area model that incorporated operational elements to the EMME.

The sub-area model incorporates traffic assignment procedure that takes into account turn capacities, and volume delay functions that explicitly capture intersection operations. Turn capacities were estimated based on the Highway Capacity Manual operational methodology, and take into account intersection laning, traffic control, signal phasing and timing, opposing traffic volumes etc. The model also explicitly incorporated off-street parking lots as "special zones" and traffic was assigned to them by reviewing the available parking spaces, parking occupancy and duration.

The model was calibrated to replicate turning volumes at key intersections as well as link volumes on major roads. After calibration, future year sub-area models were developed that were used to produce

origin-destination (O-D) trip inputs for the micro simulation model and to assess various transportation strategies.

## 1.0 INTRODUCTION

The City of Kelowna is in the process of updating their community plan for the downtown core. As part of the update, implications of transportation and land use changes on traffic operations, particularly along Highway 97 was required. Highway 97 is a provincial highway that runs through the study area. While the City has recently developed a region wide EMME model, this was not considered suitable to assess detailed operation of the road network. A more detailed modeling process was therefore required that incorporated travel demand forecasting, traffic simulation and capacity analysis. The following three-stage approach was applied to capture the detailed traffic operation.

1. Sub-area Demand Model- This covers development of a multi-modal sub-area transportation model using the EMME platform.
2. A micro simulation model. This was used to undertake dynamic traffic assignment, which took into account the interaction of traffic at adjacent intersections and localized capacity constraints. This was also used to present animation of travel conditions.
3. Intersection Capacity Analysis was used to assess performance of the key intersections using the Highway Capacity Manual operational methodology as implemented in Synchro.

The focus of this paper is on the sub-area demand model that involved non-traditional use of EMME.

### 1.1 Regional Model

The regional model is a planning tool that used link-based capacities and speeds. It covers the City of Kelowna, West Kelowna, Lake Country and Peachland. It is currently used to assess transportation implications of various land use scenarios, and determine the long range transportation requirements in the context of the regional network. The geographic area covered by the Regional Model including the zone system is shown in Figure 1.1

The model was calibrated to replicate 2008 fall/spring travel conditions using house hold travel survey, travel time survey, intersection counts and transit ridership data. It was calibrated for each stage of the standard 'four-step' procedure covering the trip generation, trip distribution, mode split and trip assignment.

The limitations of the regional model in respect to the requested detailed traffic operation for the downtown area include:

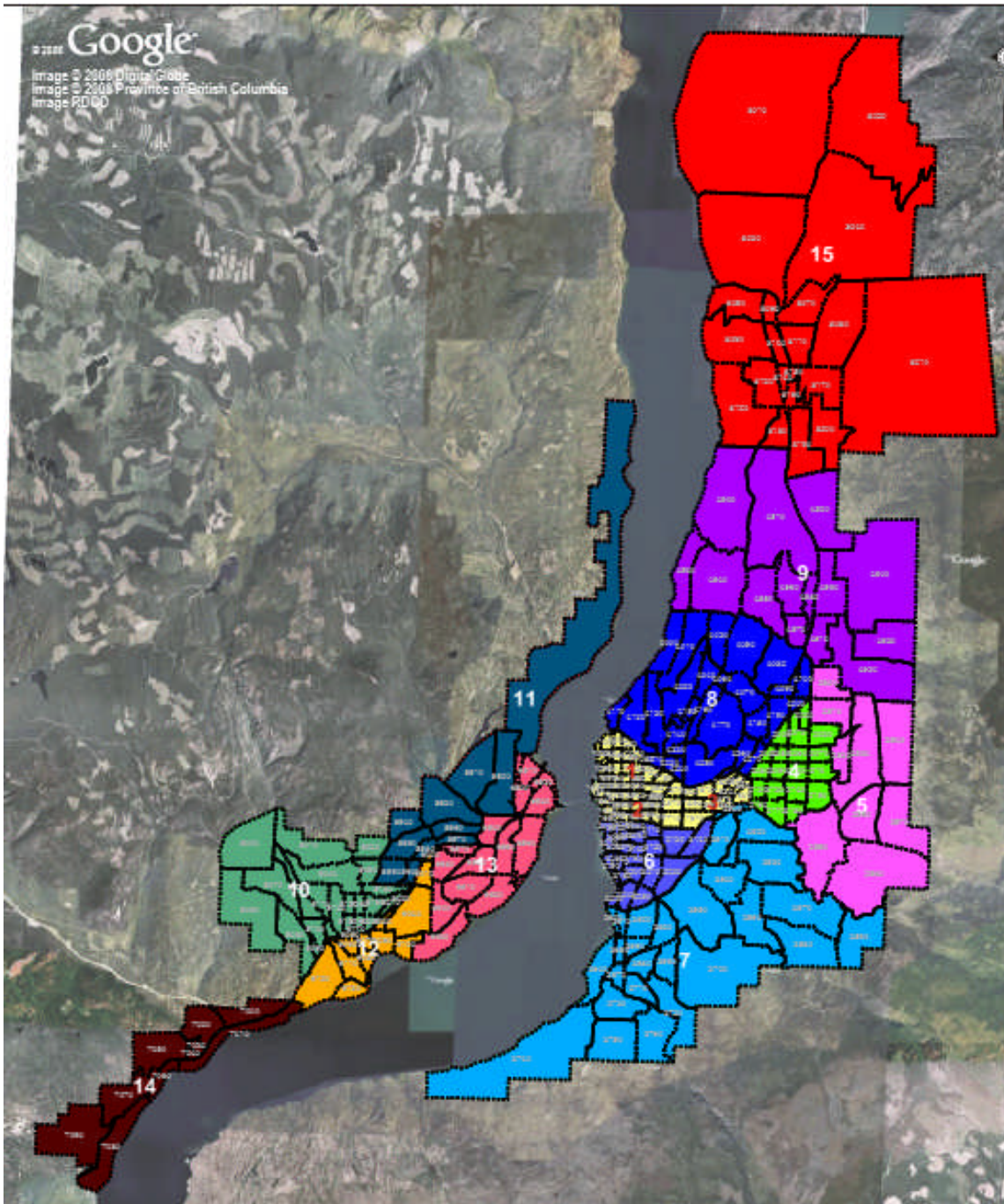
The zone system and network in the downtown area are not coded at a detailed level;

The volume calibration was limited to link volumes at screenlines;

The model does not treat off-street parking lots as independent trip generators;

This limitation was addressed by developing a detailed demand model and traffic operation models.

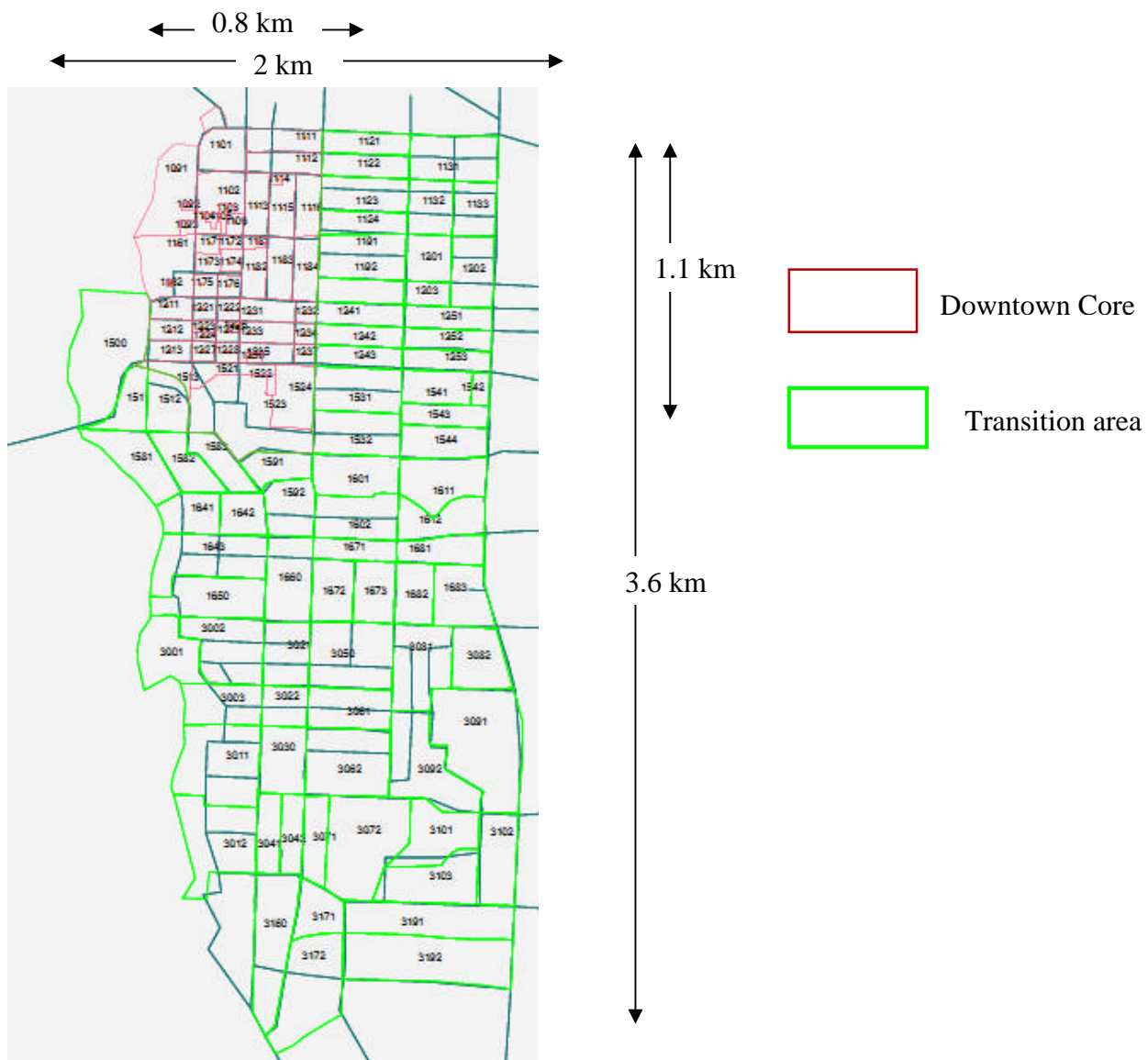
**Figure 1.1: Regional Zone System**



## 1.2 Sub-area Model

The sub-area transportation model was prepared by refining the regional model in the downtown core, i.e., a 'blow-up' of the downtown area within the regional model with no changes to the zone system and model structure outside the downtown. The sub-area model includes a detailed network and zone system within the downtown core, i.e. every block within the downtown area was treated as a zone. The sub-area zone system structure is illustrated in Figure 1.2. A transition area was considered around the downtown core that provides a transition between the detailed sub-area zone system and the coarse regional scale zone system.

**Figure 1.2: Sub-area Model Zone System**



In the sub-area model, off-street parking lots were also treated as special zones, and a model was developed to assign traffic to the parking facilities.

## 2.0 OPERATIONAL ELEMENTS

### 2.1 Model Setup

The sub-area model was developed by adding new zones and links in the downtown core and the transition area. The zone system in the downtown area was refined to a level where each block was represented by a zone, and the road network was refined to incorporate all key links. New zones and links were also added in the transition area as required.

In order to properly capture turn capacities as per the HCM operational methodology, all the necessary road network attributes were properly coded in the EMME model. The attributes coded include:

- Direction of approach legs at intersection (east, west, north, south) coded as 1, 2, 3 and 4 respectively
- Turn types (U-turn, through, left turn, right turn) coded as 0, 1, 2 and 3
- Traffic control type (signal, unsignalized) coded as 1 and 2
- Up to ten laning configurations were considered in the model ranging from a single lane approach to multi-lane approach with dual left, dual right and two or more through lanes.
- Existing signal phasing and timing including advance left turn green time, green time, and cycle length
- Left turn lane types (dedicated or shared) and left turn phases (protected, permitted or protected and permitted).

User defined attributes were created for each attribute in the turn interface, and the inputs were represented by numerical values as detailed in Table 2. 1

**Table 1 : Turn Inputs**

EMME turn attr.	1	2	3	4	5	6	7	8	9	10
Direction	East	West	North	South	-	-	-	-	-	-
Turn types	Through	Left	Right	-	-	-	-	-	-	-
Traffic Control	signal	unsignal								
Laning configuration	1 shared	↵ ↗	↵ ↑↑ ↗	↵ ↗	↵ ↑ ↗	↵ ↵ ↑ ↗	↵ ↑ ↗ ↗	↵ ↵ ↑↑ ↗	↵ ↑↑ ↗ ↗	↵ ↵ ↑↑ ↗ ↗
Left turn type, phase	D, pr+pm	D, pm	S, pr+pm	S, pm	D, pr	prohibited				
Signal time	Existing values									

D= dedicated left turn lane, S= shared left turn lane, pr= protected phase, pm= permitted phase

### 2.2 Model Procedures

The regional model was calibrated for each stage of the standard 'four-step' procedures covering the trip generation, trip distribution, mode split and trip assignment using the household travel survey, travel time

survey, turn counts and transit ridership data. The 2008 calibrated regional model was used as a base for the 2008 sub-area model.

The procedures used in the regional model were applied to estimate the trip generation, trip distribution and mode split of the sub-area model. Few adjustments were applied to some coefficients in the downtown area.

The inputs and procedures used in the traffic assignment of the two models (region and sub-area) are different. The assignment procedure in both models uses a *fixed demand, generalized cost multiclass assignment, with class specific volumes*, i.e., the assignment procedure uses fixed O-D trips and link based generalized cost to produce class specific volumes. The generalized cost includes travel time and travel cost. The auto travel cost comprises vehicle operating cost and parking cost. The approaches used to estimate cost component is similar in both models. The approaches used to estimate auto travel time in both models are described below.

## 2.3 Travel Time Estimation

Travel time is one of the main inputs in the traffic assignment, trip distribution and the mode-split procedures. In principle, there are two main components of the travel time in an urban environment.

- Link Travel time- the portion of travel time when the vehicle is in motion;
- Delay at intersections-time lost by a vehicle at intersections

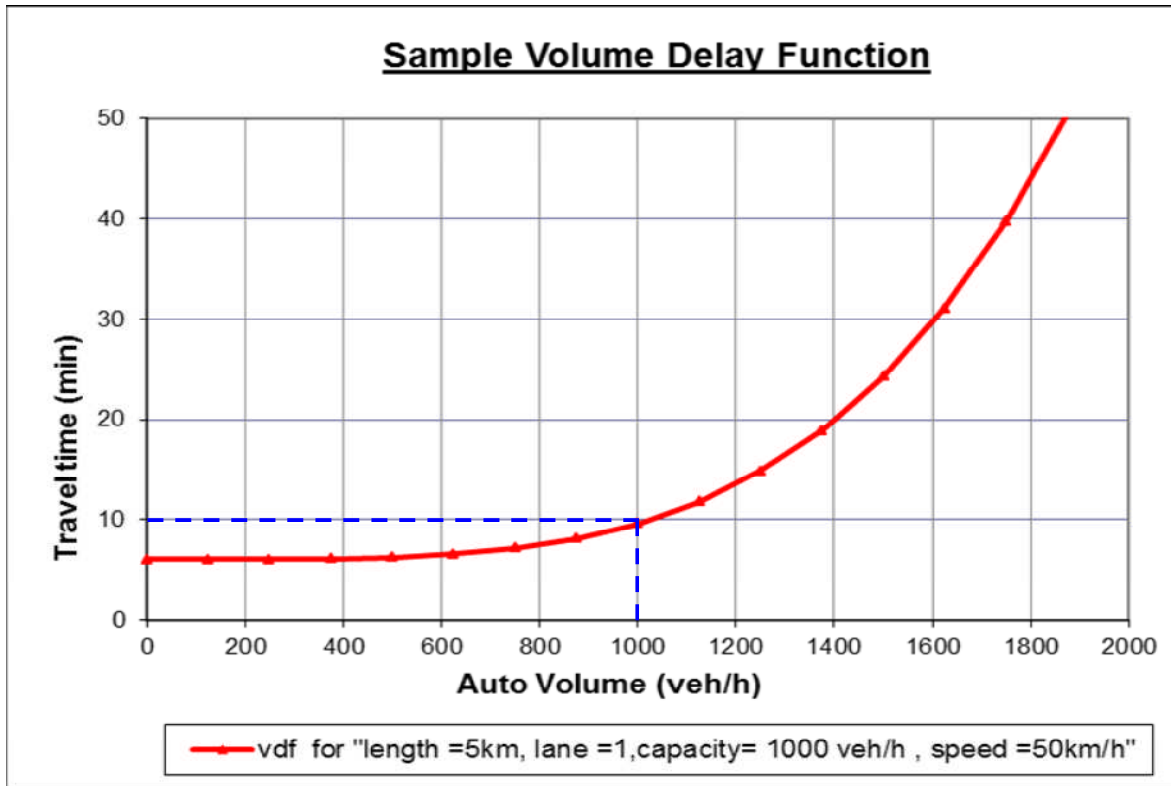
### 2.3.1 Link Travel Time

Link travel time is the portion of the travel time when a vehicle is in motion, and is primarily dependent on speed and distance. For links with low volume to capacity (v/c) ratio, the link travel time is link distance divided by the free flow speed. In this study, the free flow speed was assumed to be equal to the posted speed limit.

For congested road network, as volumes increase the travel time increases. A modified Bureau of Public Roads (BPR) equation (Equation 1) was used to estimate the link travel time in both the regional and the sub-area model. A sample *pdf* function is also illustrated in Figure 2.1.

$$Time_F = Time_I * \left[ 1 + 0.6 * \left( \frac{Volume}{Capacity} \right)^{4.0} \right] * ul1 \dots \dots \dots Eq.1$$

Where Time<sub>F</sub> = travel time on congested links, Time<sub>I</sub> = free flow travel time, Volume = assigned link volume and capacity = link capacity, ul1 = adjustment factor based on road class

**Figure 2.1: Sample Volume Delay Function (Vdf)**

As shown in the figure, at low  $v/c$  ratios, volume has practically no impact on the link travel time, however; at higher  $v/c$  ratios, increase in volume has an exponential effect on the travel time.

Based on Highway Capacity Manual (HCM) guideline, the adjusted saturation flow rate per lane in a downtown area could range between 1600 to 1800 vehicle per hour per lane. In this study 1600 veh/h/lane was used.

The BPR equation was adjusted by a factor ( $u_1$ ) in order to capture differences in link capacities for various road classes. For example, link capacity per lane on local roads is expected to be less than link capacity per lane on major roads. This is primarily due to friction caused on local roads by on-street parking, close spacing between intersections or driveways, pedestrian activity, road width and other constraints. An adjustment factor between 0.80 and 1.1 was applied to travel time to capture this. A value of 0.80 was applied for Highways, 0.90 for arterial roads, 1.0 for most collector roads, and 1.1 for the local roads in the down town area.

### 2.3.2 Intersection Delay

The intersection delay represents a portion of the travel time lost by a vehicle at an intersection. In a congested road network, such as a busy downtown area, the delay at intersections is the key component of the travel time. Reliable estimation of turn delay is therefore very important as it is the key input for trip distribution and assignment. The delay at an intersection can be a function of the geometric and traffic conditions as well as traffic control at the intersection.

Turn capacity is a key input to properly estimate delays at intersections. This is a function of traffic control, signal phase and timing, opposing traffic volumes, laning configuration of approaches etc.,. The approaches used to estimate intersection capacity in the sub-area model in comparison to the regional model are described below.

### 2.4 Regional Model Capacity Estimation

Majority of the regional traffic assignments procedures in British Columbia have been carried out using the link-based capacities. Each link is assigned with a link specific 'volume-delay function' parameter (code). The code is based on link speed, downstream traffic control and number of additional turn lanes for signalized the intersections. The first digit represents capacity based on the traffic control and number of turn lanes (for signalized intersections), while the second digit represents the posted speed. A sample of the *vdf* input codes and the corresponding capacities for a posted speed of 50km/h are shown in Table 2.2.

		<b>vdf</b>	<b>Capacity per lane</b>
	Centroid connector	1	unlimited
<b>Traffic Controls and additional turn lanes</b>	free flow	85	1,600
	Stop	25	400
	Signal & no extra turn lane	35	600
	signal+1 lane turn at intersection	45	800
	signal+2 turn lane at intersection	55	1,000
	signal+3 turn lane at intersection*	65	1,200
	signal+4 turn lane at intersection*	75	1,400

The approach used to estimate capacity for the regional model is appropriate for regional modeling. The link volume forecast could be considered acceptable from a regional context; however, its use in producing detailed traffic forecasts at the intersection level is limited. The drawbacks include:

- All stop-controlled approaches are assumed to have a fixed capacity of 400 veh/h irrespective of the laning configuration of the intersecting roads, and volume on the approaches. All movements on stop-controlled approach are assumed to have the same capacity regardless if the movement is left turn, right turn or through movement,
- The signal phasing and timing are not taken into account in capacity estimation, i.e., two approaches with the same laning configuration are assumed to have the same capacity irrespective of the green time allocation and the cycle length.

- All exclusive turn-lanes are assumed to have a fixed capacity of 200 veh/ h irrespective of conflicting volumes, available green time etc.

## 2.5 Sub-area Model Capacity Estimation

In order to address the implications of land use changes and alternative networks in traffic operation of the road network in the downtown area, a detailed road network system was developed that incorporated intersection operational/capacity parameters.

In the sub-area model, HCM based methodologies were applied to estimate turn capacities at signalized and unsignalized intersections. An algorithm was developed that allows dynamic adjustment of capacities based on projected volumes. This specifically captures the impact of conflicting volumes on capacities. The methodology followed in the sub-area model to estimate turn capacities at signalized and unsignalized intersections are described below.

### 2.5.1 Signalized Intersections

- Each approach was first disaggregated into a number of lane groups as per HCM guidelines;
- A saturation flow rate for each lane group was computed by taking into account the ideal saturation flow rate and various adjustment factors including left-turns and right-turn adjustments. Assumptions were made for road grade, bus, pedestrian, and parking activities.
- Capacity for each lane group was estimated based on existing signal timing plan. The capacity estimation takes into account phasing and green time for each lane group and the cycle length. A fixed green time was used in the capacity calculation. In peak periods, the lane groups were assumed to use most of the allocated green times (no gap-out).

#### 2.5.1.1 Lane Group:

Each approach was disaggregated into a number of lane groups as shown in Table 2.3.

Lane Group ID	Approach configuration	Lane Groups		
1	1 shared lane	1 shared lane		
2	↶ ↷	↶	↷	
3	↶ ↑↑ ↷	↶	↑↑	↷
4	↶ ↷	↶ ↷		
5	↶ ↑ ↷	↶ ↑	↷	
6	↶ ↶ ↑ ↷	↶ ↶	↑ ↷	
7	↶ ↑ ↷ ↷	↶ ↑	↷ ↷	
8	↶ ↶ ↑↑ ↷	↶ ↶	↑↑	↷
9	↶ ↑↑ ↷ ↷	↶	↑↑	↷ ↷
10	↶ ↶ ↑↑ ↷ ↷	↶ ↶	↑↑	↷ ↷

### 2.5.1.2 Saturation Flow Rate:

As per HCM guideline, a base saturation flow rate ( $S_0$ ) of 1,900 passenger cars per hour per lane was assumed for the ideal condition. Saturation flow rate for each lane group was established by applying adjustment factors to the base saturation flow rate (Equation 2).

$$s = s_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \text{-----Eq.2}$$

Where

$s$  = saturation flow rate for the subject lane group;

$S_0$  = base saturation flow rate,  $N$  = number of lanes in lane group,

$f_w, f_{HV}, f_g, f_p, f_{bb}, f_a, f_{LU}, f_{LT}, f_{RT}, f_{Lpb}, f_{Rpb}$  = adjustment factors for lane width, heavy vehicles, approach grade, parking maneuvers, bus blockage, area type, lane utilization, pedestrian effect on left turns and pedestrian effect on right turns respectively.

The default values assumed for lane width, grade, percentage of heavy vehicles, type of area, parking maneuvers, bus activity, and lane utilization and the respective adjustment factors are summarized in Table 2.4.

The saturation flow estimation assumes no parking maneuvers and bus stops within 75 meters from the intersection. The effect of pedestrian activity on left turn and right turn movements was also not accounted in the saturation flow estimation. As a result, no adjustment factors were applied for parking maneuvers, bus blockage and pedestrian activity; however, the delay calculation takes into account effect of parking, and pedestrian activities by applying a gross adjustment factor ( $u_{l1}$ ) based on road classes (see Section 2.3.1). Estimation of left turns and right turns adjustment factors are described in a separate section below.

	<b>Default value</b>	<b>Adjustment factor</b>
Lane width	3.5 meters	0.99
Heavy vehicle percentages	2%	1.0
Area type	Central business District (CBD)	0.9
Parking maneuvers	0	1.0
Bus blockage	0	1.0
Lane utilization	-	0.95
<i>Combined adjustment factor</i>	-	<i>0.85</i>

After applying the adjustment factors described in Table 2.4, the saturation flow rate for a through lane is approximately 1600 (1900 x 0.85) vehicles per hour per lane. In the study a saturation flow rate of 1600 veh/h/lane was used for a through lane, i.e., a lane with no adjustments needed for left and right turns.

### 2.5.1.3 Right Turn Adjustment Factor

The right turn adjustment factor was estimated based on HCM guide line as shown below:

Exclusive lane,  $F_{RT} = 0.85$

Shared lane,  $F_{RT} = 1 - (0.15)P_{RT}$  -----Eq.3

Single lane,  $F_{RT} = 1 - (0.135)P_{RT}$  -----Eq.4

Where  $P_{RT}$  = proportion of right turns in the lane group

The proportion of turn movements is computed after each assignment cycle using the assigned turn volumes, and the total approach volumes and is stored in a *user defined attribute (@prop)*. This is then used as input to estimate the right turn adjustment factor.

#### 2.5.1.4 Left Turn Adjustment Factor

The calculation of left turn adjustment factors is very complicated and depends on many factors including whether a left turn is made from an exclusive or shared lane, opposing volumes under the permitted phases, type of phasing (protected, permitted or protected plus permitted), and proportion of left-turn traffic. Based on the HCM guideline, the following five cases were defined to estimate the left turn adjustment factor.

- Exclusive lane with protected phasing;
- Exclusive lane with permitted plus protected phase;
- Exclusive lane with permitted phase;
- Shared lane with protected plus permitted phase;
- Shared lane with permitted phase;

The left turn adjustment factors used for each case are summarized in Table 2.5.

Left turn ID	Lane turn type	Left turn factor
1	Exclusive + protected	0.95
2	Exclusive + permitted	Section 2.5.15
3	Exclusive +protected + permitted	Case 1 for protected phase Case 2 for the permitted phase
4	Shared +protected permitted	Case 1 for protected phase Case 2 for the permitted phase
5	Shared + permitted	Section 2.5.15

The impact of permitted left turns on intersection operation is very complex, and the procedure used to estimate the left turn adjustment factor is briefly described in Section e) below.

#### 2.5.1.5 Left turn Adjustment Factor for Permitted Phases

The following two cases were considered for the permitted left turn traffic:

- Permitted phase from shared lane;
- Permitted phase from exclusive lane;

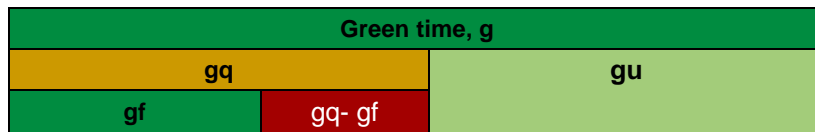
When a left turn movement is made from a shared lane group, with initiation of the green time the opposing queue starts to move, and this will block the left turn traffic. This portion of the green time blocked by clearance of opposing queue is defined by **gq**.

Until the first left turn vehicle arrives the shared lane is unaffected by left turning traffic. The portion of the green time until arrival of first left turn vehicle is referred as **gf**.

After the opposing queue clears, the subject left-turning traffic starts to filter though the opposing traffic at a rate affected by the opposing traffic volume. This portion of green time is referred as **gu**.

The various portions of the green time for a shared lane are shown in Figure 2.2

**Figure 2.2: Green time portions of a Shared lane**



The effect of left turn traffic on the shared lane during the three distinct periods is summarized in Table 2.6.

Table 2.6: Effect of left turn traffic on shared lane	
periods	Left turn factor
gf	1.0
gq-gf	0.0
gu	between 0.0 and 1.0

The left turn adjustment factor for the lane from which permitted left turns are made are computed using Equation 5.

$$f_m = \left(\frac{g_f}{g}\right) * 1.0 + \left(\frac{g_q - g_f}{g}\right) * 0.0 + \left(\frac{g_u}{g}\right) * \left[\frac{1}{1 + P_L (E_{L1} - 1)}\right] \text{-----Eq.5}$$

Where PL = proportion of left turn traffic in shared lane

EL1 = through car-equivalent for permitted left turn

All the parameters in equation 5 (gf, gq, gu, PL, and EL1) were estimated using HCM equations and approaches. The equations and inputs used to estimate the green time periods and other parameters are summarized in Table 2.7

Table 2.7: Equations to Estimate Green Time Periods		
Parameters	Equations	Inputs
gf	$g_f = g e^{-0.882LTC^{-0.717}}$	g= permitted green time, LTC= left turn per cycle ( $V_{LT} \cdot C / 3600$ ) gf =0.0 for exclusive permitted left turn lanes
gq	$g_q = \frac{V_{o/c} q r_o}{0.5 - \frac{V_{o/c} (1 - q r_o)}{g_o}}$	$V_{o/c}$ = opposing flow rate per lane per cycle ( $V_o \cdot C / 3600 N_o$ ) $N_o$ =opposing No. of lanes, $g_o$ =green time for opposing flow $q r_o$ = opposing queue ratio estimated as $1 - g_o / C$ $\frac{V_{o/c} (1 - q r_o)}{g_o} \leq 0.49$ and $0 \leq g_q \leq g$
gu	$g_u = g - g_f$	When $g_q \geq gf$
	$g_u = g - g_q$	for $g_q < gf$
EL1	$E_{L1} = \frac{1900}{S_{LT}} - 1$	Shared lane, $S_{LT} = \frac{V_o e^{\left(\frac{V_o t_c}{3600}\right)}}{1 - e^{\left(\frac{V_o t_f}{3600}\right)}}$
	$E_{L1} = \frac{1900}{S_{LT}}$	Exclusive left turn lane, $S_{LT} = \frac{V_o e^{\left(\frac{V_o t_c}{3600}\right)}}{1 - e^{\left(\frac{V_o t_f}{3600}\right)}}$

Where,

VLT = left turn volume, Vo = opposing flow, C = cycle length, tc= critical gap = 4.5 s, tf = follow up-headway =4.5 sec for shared lane and 2.5 sec for exclusive lane.

The left turn adjustment factor for a lane group from which left turns are made are computed using Equation 6.

$$f_{LT} = \left( \frac{f_m + 0.91(N - 1)}{N} \right) \text{-----Eq.6}$$

Where  $f_{LT}$  = left turn adjustment factor for a lane group

$f_m$  = left turn adjustment factor to a lane from which left turns are made

N= number of lanes on lane group

Estimation of the left turn adjustment factor for the permitted phase is complex as it requires computation of volumes, green time and number of lanes for the approach and opposing lanes.

### 2.5.1.6 Capacity Calculation

Capacity for a lane group at signalized intersection was estimated using Equation 7.

$$c = \left( \frac{sg}{C} \right) \text{-----Eq.7}$$

Where c = capacity for a lane group

s= saturation flow rate for a lane group

g= green time for a lane group,

C= cycle length

The procedures applied in the study to estimate capacity for a given lane group and left turn phase are summarized in Table 2.8.

**Table 2.8: Turn Capacity Estimation**

Lane Group		Left turn phase	Capacity	Description
No.	Type			
1	↑↑	-	$1600*(g/C)*N$	N =number of lanes
2	↶	protected	$1600* f_{LT1}*(g/C)$	$f_{LT1}= 0.95$ , $g_l$ = left turn green time
3	↶	Permitted	$1600* f_{LT2}*(g/C)$	$f_{LT2}$ estimated using Eq. (6)
4	↶	Protected + permitted	$1600*f_{LT1}*(g/C)+ 1600*f_{LT2}*(g/C)$	$f_{LT1} = 0.95$ , $f_{LT2}$ estimated using Eq. (6)
5	↷	-	$1600*f_{RT1}*(g/C)$	$f_{RT1}$ estimated using Eq. (3)
6	↶↷	Permitted	$1600*f_{LT2} *f_{RT2}*(g/C)*N$	$f_{RT2}$ estimated using Eq. (4) and $f_{LT2}$ Eq. (6)
*7	↶↷	Protected + permitted	$1600*f_{LT1}*(g/C)+ 1600*f_{LT2} *f_{RT2}*(g/C)*N$	$f_{LT1} = 0.95$ , $f_{RT}$ estimated using Eq. (3) and $f_{LT2}$ Eq. (6)
8	↶↶	protected	$1600*f_{LT1}*(g/C)*N$	$f_{LT} = 0.95$
9	↑↷	-	$1600 *f_{RT2}*(g/C)*N$	$f_{RT2}$ estimated using Eq. (4)
10	↷	-	$1600*f_{RT}*(g/C)$	$f_{RT} =0.85$
11	↷↷	-	$1600*f_{RT}*(g/C)*N$	$f_{RT} =0.85$

For Lane Group No. 7( shared through, left and right + protected and permitted left turn phases), the approach given in HCM to estimate left turn adjustment factor is a bit complicated. The approach used in this study is a simplified version of HCM. For the study area, almost all protected plus permitted phases have an exclusive left turn lane. As a result, this has practically no impact in the capacity estimation.

### 2.5.1.7 Sample Model Inputs and Output Results

The operational attributes, and the equations described above were implemented in the Sub-area EMME model. Multiple user defined turn attributes were created to properly capture all the operational parameters. The key turn attributes used in the sub-area model and their description is given in Table 2.9.

**Table 2.9: Turn Attributes**

Signalized Intersection		Unsignalized Intersection	
Attribute	Description	Attribute	Description
@dir	direction, 1 =east, 2= west, 3=north, 4 =west	@dir	direction
@mov	movement type , 1 = through, 2 = left turn, 3 = right turn	@mov	movement type
lanes	lanes	lanes	lanes
@mainr	Traffic control, 3 = signal	@mainr	Traffic control, 1 = road with priority , 2 = stop -controlled
@Ingrp	Lane Group ID as per Table 2.1	pvolau	turning volumes
@lty	left turn ID, as per Table 2.1	@vlopu	Opposing volume (un signalized intersection)
@advgr	advance green ,existing	@tcrt	critical gap
@green	green time ,existing	@tflow	follow-up time
@cycle	cycle length ,existing	@poj	vehicle impedance adjustment factor
@volop	Opposing volume for left turns (signalized intersection)	@capmt	capacity by movement
@gf	gf ( see fig x.x)	@lkcap	Link capacity
@gq	gq ( see fig x.x)		
@gu	gu ( see fig x.x)		
@el1	EL1		
@flt	left turn adjustment factor		
@frt	Right turn adjustment factor		
@caplt	left turn capacity		
@capth	through capacity		
@caprt	right turn capacity		

A sample of operation input and output attributes for one signalized intersection (Highway 97/ Ellis Street) as obtained from an EMME model are given in Table 2.10.

**Table 2.10: Sample Capacity input and outputs—Signalized Intersections (four leg intersection)**

ID	INPUTS									Outputs								
	@dir	@mov	lanes	@mainr	@Ingrp	@ltty	@advgr	@green	@cycle	@volop	@gf	@gq	@gu	@e1	@flt	@frr	@caplt	@capth
10234-10107-10107	4	0	1	3	6	5	0	40	130	132	0.0	0.0	0.0					
10234-10107-10195	4	3	1	3	6	5	0	40	130	132	0.0	0.0	0.0			0.94		
10234-10107-10199	4	2	1	3	6	5	11	40	130	132	0.0	0.0	40.0				257	
10234-10107-10260	4	1	1	3	6	5	0	40	130	132	0.0	0.0	0.0			0.94		462
10234-10195-10107	1	2	3	3	2	1	14	78	130	1764	0.0	34.5	43.5	6.90	0.08		241	
10234-10195-10195	1	0	3	3	2	1	0	78	130	1764	0.0	0.0	0.0					
10234-10195-10199	1	1	3	3	2	1	0	78	130	1764	0.0	0.0	0.0		0.08	1.00		2878
10234-10195-10260	1	3	3	3	2	1	0	78	130	1764	0.0	0.0	0.0			1.00		
10234-10199-10107	2	3	3	3	2	2	0	59	130	1170	0.0	0.0	0.0			0.99		
10234-10199-10195	2	1	3	3	2	2	0	59	130	1170	0.0	0.0	0.0		0.19	0.99		2161
10234-10199-10199	2	0	3	3	2	2	0	59	130	1170	0.0	0.0	0.0					
10234-10199-10260	2	2	3	3	2	2	0	59	130	1170	0.0	14.4	44.6	3.90	0.19		141	
10234-10260-10107	3	1	1	3	2	2	0	28	130	374	0.0	0.0	0.0		0.14	0.96		331
10234-10260-10195	3	2	1	3	2	2	0	28	130	374	0.0	23.6	4.4	1.86	0.14		49	
10234-10260-10199	3	3	1	3	2	2	0	28	130	374	0.0	0.0	0.0			0.96		
10234-10260-10260	3	0	1	3	2	2	0	28	130	374	0.0	0.0	0.0					

### 2.5.1.8 Capacity Comparison with Operational Models (Synchro)

The sub-area model turn capacities were compared with the capacities of synchro operational model, and a summary of the comparison for a selected intersection is shown in Table 2.11. Both models use the same approach volumes. The approach capacities were also estimated based on the approach volumes and turn capacities and a comparison is also given in the same table.

**Table 2.11: Signalized Intersection Capacity Comparison (Highway 97/Ellis Street)**

L= left, T= through, R=right	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Volume by turns	121	1170	5	42	1665	100	38	88	44	222	126	243
Lane group volumes	121	1175		42	1765		38	132		222	369	
Synchro turn capacity	183	3092		191	2486		93	293		172	467	
Sub-area model turn capacity	241	2878		140	2161		49	331		257	461	
Synchro approach capacity	2820			2433			248			356		
Sub-area approach capacity	2632			2161			331			384		
Regional approach capacity	2400			2400			800			1000		

As shown in the Table, the sub-area model capacities replicate the synchro capacity results closely; however, the regional model capacities for the northbound and southbound movements do not.

## 2.5.2 Unsignalized Intersections

The capacity for unsignalized intersections was also estimated using HCM guideline. The procedures followed to estimate the capacity are as follows.

- Identify conflicting movements for each movement on the approach;
- Determine conflicting volumes for each movement as per HCM equations;
- Estimate critical gap and follow-up time
- Estimate base capacity for each movement using the gap acceptance approach;
- Adjust base capacity

### 2.5.2.1 Capacity Calculation

- a) **Conflicting Volumes:** the conflicting volume for each movement waiting a gap on the major road (left turn traffic on major road as well as all movements on the minor road) was estimated using HCM guidelines. For instance, for a northbound through traffic on a minor road, the conflicting volume is estimated using Equation 8.

$$V_{NT} = 2V_{EL} + V_{ET} + 0.5V_{ER} + 2V_{WL} + V_{WT} + V_{WR} \text{ -----Eq. 8}$$

Where V= volume, N=north, E =east, W= west, L = left, T= through, R= right

- b) **Critical Gap and follow-up time:** the critical gap and the follow-up time were estimated using HCM approaches. The critical gap was estimated using equation 9, while the follow-up time equation 10. The base critical gap and follow-up time are shown in Table 2.12.

$$t_{cx} = t_{cbase} + t_{cHV} P_{HV} + t_{cG} G - t_{cT} - t_{3LT} \text{ ----- Eq. 9}$$

Where,  $t_{cx}$ =critical gap for movement X,  $t_{cbase}$  = base critical gap as shown in Table 2.12,  $t_{cHV}$  = adjustment factor for heavy vehicles (1.0 for two-lane major road and 2.0 for four-lane major road),  $P_{HV}$ = percentage of heavy vehicles,  $t_{cG}$ = adjustment factor for grade,  $G$ = grade (%),  $t_{cT}$  = adjustment factor for two-stage gap acceptance process,  $t_{3LT}$  = adjustment factor for intersection geometry (0.7 for minor street left turn movement at three leg intersection, 0.0 otherwise)

For simplicity, a default value of 2% heavy vehicles and 0% grade was assumed in the study. There are no two-stage gap acceptance processes within the study area.

Table 2.12: Base Critical Gap and follow-up time			
Vehicle movement	Base critical gap		Base follow up time
	Two-lane major St.	Four-lane major St.	
Left turn from major	4.1	4.1	2.2
Right turn from minor	6.2	6.9	3.3
Through from minor	6.5	6.5	4.0
Left turn from minor	7.1	7.5	3.5

$$t_{fx} = t_{fbase} + t_{fHV} P_{HV} \text{ -----Eq. 10}$$

Where,  $t_{fx}$ =critical follow-up time for movement X,  $t_{fbase}$  = base follow-up time as shown in Table 2.12 ,  $t_{fHV}$  = adjustment factor for heavy vehicles (0.9 for two-lane major road and 1.0 for four-lane major road),  $P_{HV}$ = percentage of heavy vehicles.

- c) **Capacity Estimation:** the gap acceptance model as given in HCM manual (equation 11) was used to estimate the base capacity at unsignalized intersections.

$$c_{p,x} = \frac{V_{ox} e^{\left(\frac{V_{ox} t_{cx}}{3600}\right)}}{1 - e^{\left(\frac{V_{ox} t_{fx}}{3600}\right)}} \text{ -----Eq. 11}$$

where  $C_{p,x}$ =base capacity for movement X,  $V_{ox}$ =conflicting movement for movement X

- d) **Capacity Adjustment:** vehicles use available gaps on the major street according to their priority. For instance, left turn movement on major Street has priority over minor road movements, through movement on minor street has priority over left turn movement on minor street. Higher priority movements can impede lower priority movements from using gaps created on the major street and this could result in capacity reduction of the minor street. The base capacity estimated using

equation 11 was adjusted to account for this impedance. The impedance factor was estimated for each movement using HCM equations (not shown here). The final capacity was estimated using equation 12.

$$C_{m,x} = C_{p,x} f_x \text{ ----- Eq. 12}$$

where  $C_{m,x}$ =capacity for movement X,  $f_x$ =vehicle impedance adjustment factor for movement X

### 2.5.2.2 Sample Model Inputs and Output Results

The turn attributes used for unsignalized intersections are shown in Table 2.9. A sample of sub-area model input and output values for one unsignalized intersection (Ethel Street/ Lawrence Ave) are shown in Table 2.13.

<b>Table 2.13: Sample Capacity input and outputs– Unsignalized Intersections</b>											
id	INPUT				OUPUT						
	@dir	@mov	lanes	@mainr	pvolau	@vlopu	@tcrt	@tflow	@poj	@cap	@lkap
10190-10045-10064	1	1	1	2	79	922	6.52	4.02	0.81	219	492
10190-10045-10156	1	2	1	2	18	922	7.12	3.52	0.73	183	
10190-10045-10226	1	3	1	2	89	237	6.22	3.32	1.00	802	
10190-10064-10045	2	1	1	2	45	903	6.52	4.02	0.81	225	280
10190-10064-10156	2	3	1	2	6	255	6.22	3.32	1.00	784	
10190-10064-10226	2	2	1	2	4	981	7.12	3.52	0.62	142	
10190-10156-10045	4	3	1	1	11	1			1.00	1360	1352
10190-10156-10064	4	2	1	1	172	279	4.12	2.22	1.00	1027	
10190-10156-10226	4	1	1	1	220	1			1.00	1600	
10190-10226-10045	3	2	1	1	44	242	4.12	2.22	1.00	1059	1488
10190-10226-10064	3	3	1	1	47	1			1.00	1360	
10190-10226-10156	3	1	1	1	223	1			1.00	1600	

### 2.5.2.3 Capacity Comparison with Operational Models (Synchro)

The sub-area model capacities were compared with synchro and regional model capacities using the same approach volumes. A summary of the comparison is shown in Table 2.14.

**Table 2.14: Capacity Comparison (Ethel Street/Lawrence Ave)**

L= left, T= through, R=right	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Volume by turns	18	79	89	4	45	6	44	223	47	172	220	11
Lane group volumes		186			55			314			403	
Synchro turn capacity		332			229			1510			1657	
Sub-area model turn capacity		492			280			1352			1488	
Synchro approach capacity	332			229			1510			1657		
Sub-area approach capacity	492			280			1352			1488		
Regional approach capacity	400			400			1600			1600		

As shown in the Table, the sub-area model capacities are close to the synchro capacity results. The regional model capacities are insensitive to conflicting volumes.

## 3.0 TRIP ASSIGNMENT TO OFF-STREET PARKING LOTS

In the sub area model, off street parking lots were treated as 'special zones'. The overall objective was to re-allocate trips to the off-street parking lots. For regular zones, the trip generation was estimated using land use data which covers population by age category, number of employees for each employment type, grade school, and post-secondary school enrolments. For the off street parking lots, the same procedures could not be applied as there are no associated land use data.

The auto trips to the parking lots are however related to the trips of the nearby regular zones. This is because a portion of the auto trips of the nearby regular zones end at the parking lots. After parking, the drivers and passengers walk to the adjacent regular zones.

### 3.1 Trip Allocation Approach

The following procedures were adopted to estimate traffic to and from the off street parking lots:

- Identify parking lots
  - Establish a 'base year' inbound and outbound traffic at the accesses of each off-street parking lots
  - Estimate origin-destination (O-D) auto trips of the parking lots;
  - Deduct the auto trips ending at the parking lots from the auto trips of the adjacent regular zones
- a) *Base year volumes:* Existing volumes for the off street parking lots were estimated based on a review of available parking spaces, observed parking demand profile, and parking duration. Split between inbound and outbound trips were estimated by reviewing the split provided in the

Institute of Transportation Engineers (ITE) trip generation manual for the dominant land uses in the nearby zones.

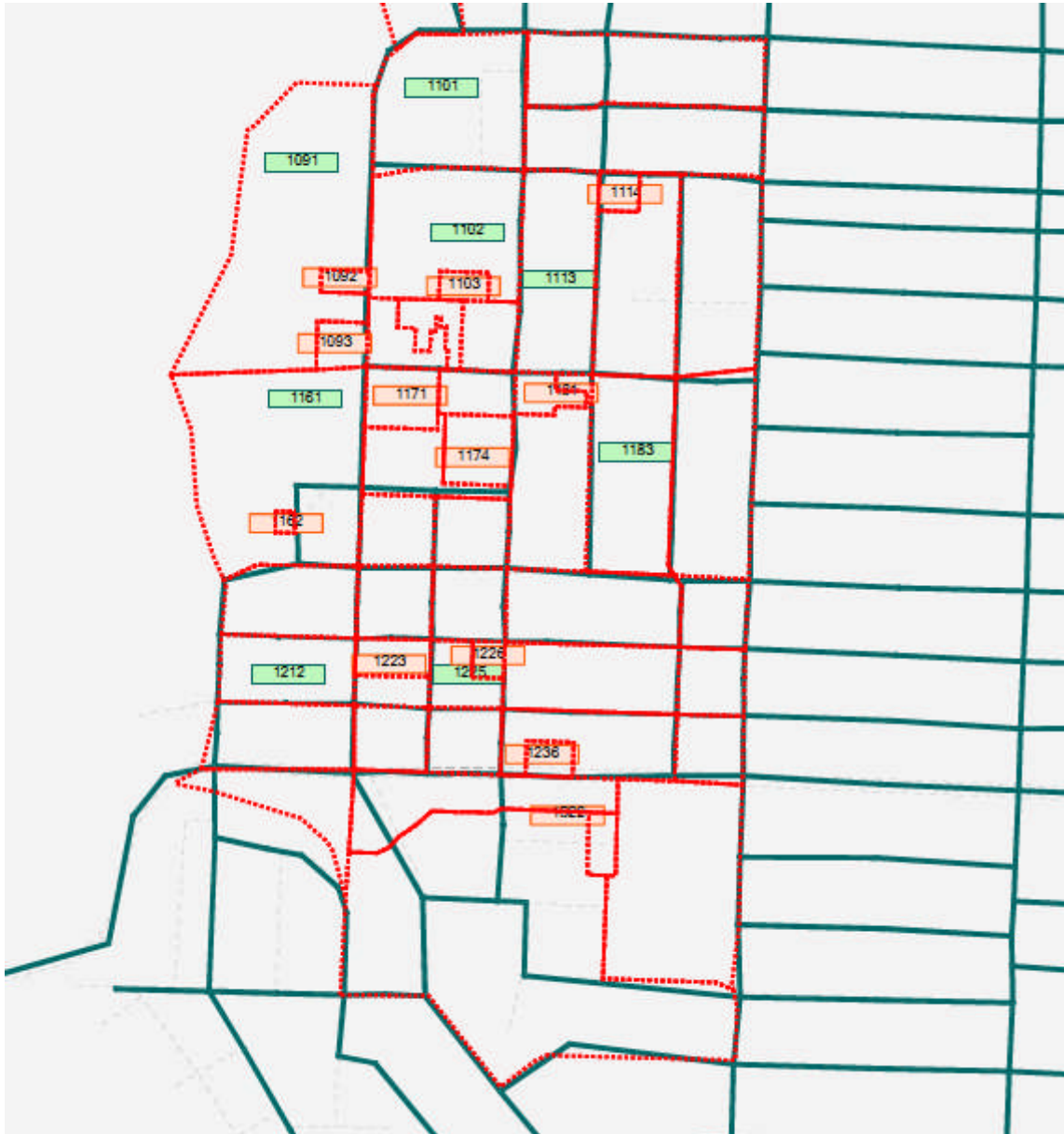
- b) *Volume Assignment to parking Lots:* A procedure was prepared within EMME that estimates and allocates auto O-D trips to the off-street parking zones. The procedure takes into account the base volume described above, travel impedances and accessibility factors of the various trip purposes in the adjacent area.
- c) *Volume deduction from the regular zones:* The auto O-D trips allocated to the parking lots are component of the O-D trips of the nearby zones. As a result, the original auto O\_D trips of the nearby regular zones were reduced to account for this. A procedure was prepared that estimates the trips to be deducted from each nearby zone. The deduction was limited to zones within 1.0 kilometer distance from the parking lots. A gravity model was used to estimate the trip reduction from each nearby zone, i.e., the reduction is proportional to the trip generation of the zones and inversely proportional to the travel impedance between the parking lot and the adjacent zone.

### 3.2 Sample EMME Results

Trip allocation to the off-street parking lots and deduction of auto trips from sample nearby regular zones is presented in Table 3.1. The off street parking zones and the sample nearby zones are illustrated in Figure 3.1.

Table 3.1: Trip Allocation to Off-street Parking Lots									
Parking Lots			Regular nearby zones						
O-D trips			Trip deduction examples						
			Zone No.	Original trips		Deductions		Adjusted trips	
Zone No.	To	from		To	from	To	from	To	from
1092	2	5	1091	144	342	19	39	125	303
1093	11	34	1101	49	79	3	9	46	70
1103	60	180	1102	56	65	5	20	51	45
1114	3	10	1113	52	53	14	19	38	34
1162	2	5	1161	96	144	16	50	80	94
1171	17	52	1183	57	88	7	20	50	68
1174	23	69	1212	104	125	11	57	93	68
1181	10	31	1225	19	45	7	19	12	26
1223	68	205							
1226	5	14							
1236	3	8							
1522	13	38							

**Figure 3.1: Off-Street Parking Lot Zones**



## 4.0 VOLUME CALIBRATION AND VALIDATION

One of the key measures used to validate a transportation model is its ability to replicate the observed volumes. In the calibration process, adjustments were applied to centroid connectors, volume delay functions, and minor adjustments to other network attributes as necessary; however, no manual local adjustments were applied. The auto volumes estimated using the auto assignment sub-model was validated using the following three criteria:

- Link volume Deviations
- Goodness of fit test
- Turn volumes at intersections

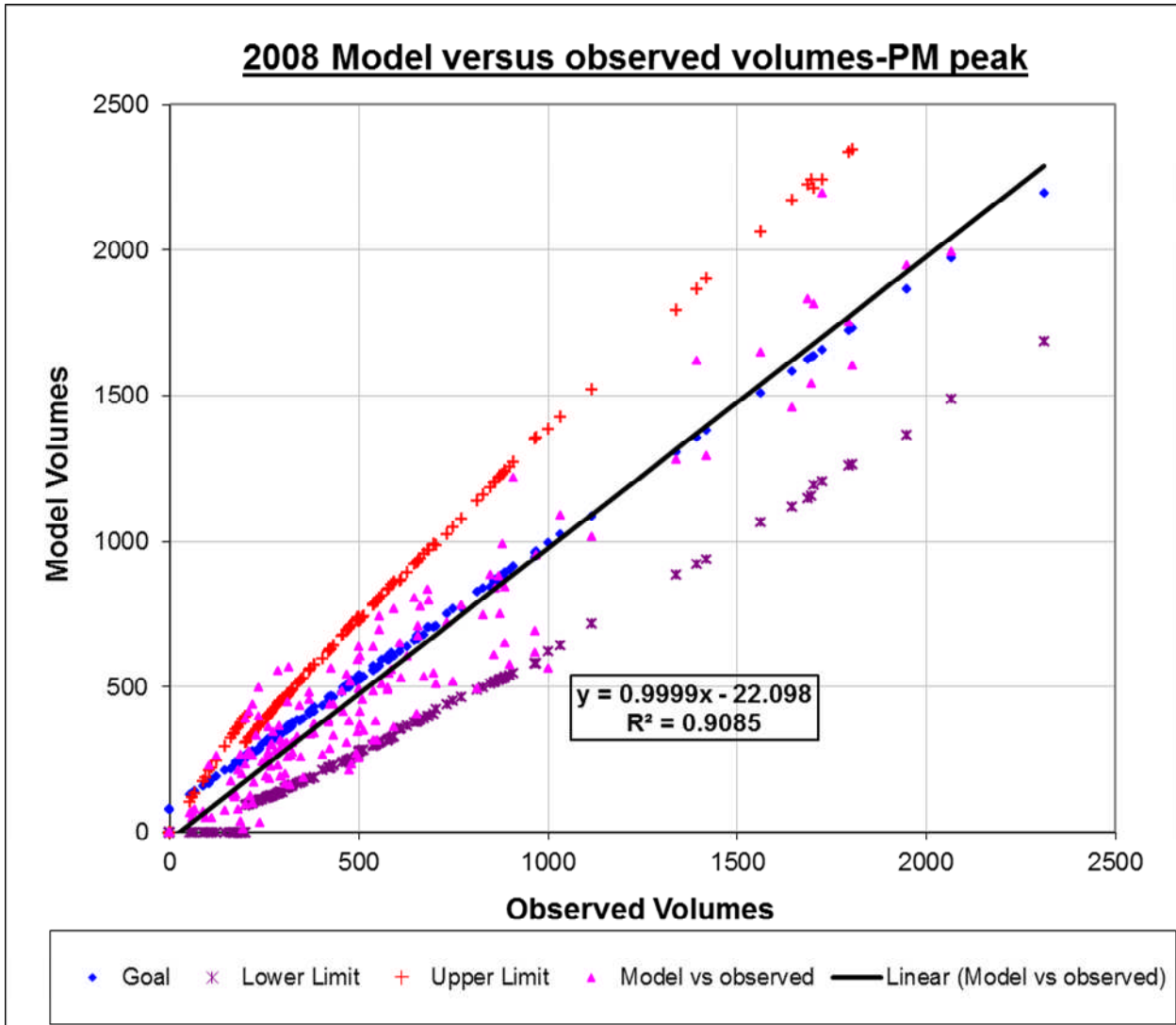
### 4.1 Volume Deviations

Deviation of model volume from the observed volumes was assessed Go. Based on this guideline, the permitted percentage errors are low on high volume roads and high on low volume roads, i.e., the higher the link volumes the lower the percentage errors permitted.

Traffic counts were undertaken at all the major intersections within the downtown area. In order to properly calibrate and validate the volumes on all major roads, up to 20 screenlines were selected within the downtown area.

The volume comparison between the model and observed volumes are presented in Figure 4.1. The tolerances permitted for each volume range as per NCHRP 255 report are also shown in the same Figure. As shown in the figure, up to 92% of the link volumes fall within the acceptable deviations. The key link volumes are all well within the acceptable limits. This indicates that the model link volumes in good agreement with the observed link volumes.

Figure 4.1: Link Volume Comparison



## 4.2 Goodness of Fit Test

The following two statistical parameters were used to test the goodness of fit between the model volumes and the observed volumes.

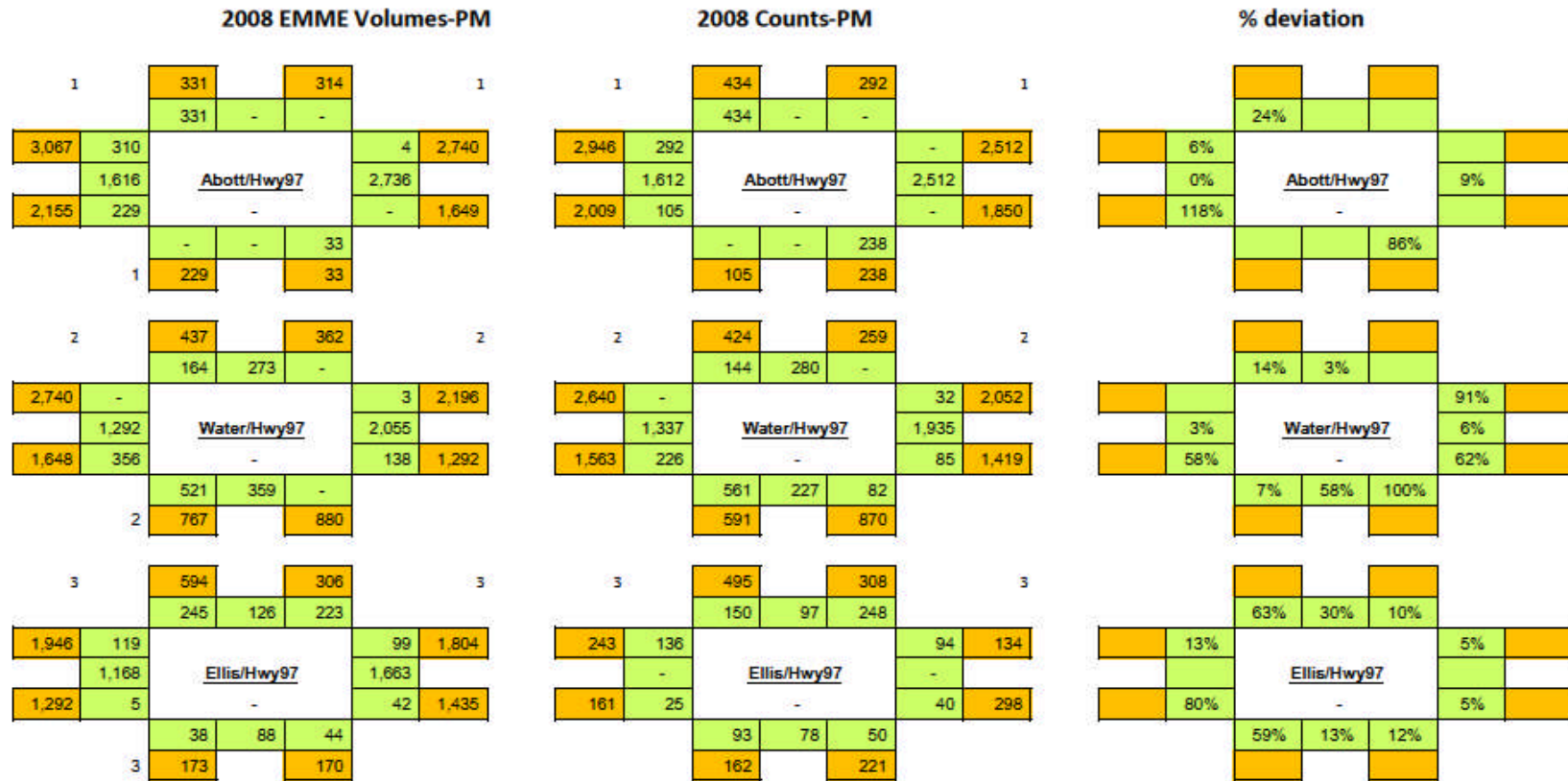
- Coefficient of determination ( $R^2$ ),
- The slope between model and observed volumes,

A value of  $R^2$  (coefficient of determination) close to 1.0 shows good agreement between model and observed volumes, while a value close to 0.0 means poor agreement. Slope is also used to determine the relationship between the predicted and the observed volumes in linear models. A value close to 1 indicates good agreement between the two volumes. As shown in the figure 4.2, the slope is approximately 1 and the  $R^2$  value is 0.91. This also indicates that there is a good fit and strong linear relationship between the two volumes.

## 4.3 Intersection Turning Volumes

Model turn volumes were also compared with the observed turn volumes for the key intersections. A sample of the turning volume comparison for key intersections along Highway 97 is illustrated in Figure 4.2. As shown in the figure, there is good agreement between the model and observed turn volumes for the major turn movements.

Figure 4.2: Turn Volume Comparison



**2008 EMME Volumes-PM**

4		773		380		4
		79	551	143		
1,833	49	<u>Richter/Hwy97</u>			87	1,815
	1,395				1,686	
1,457	13	-			42	1,541
					68	244
	4	606		315		

5		483		433		5
		108	247	128		
1,815	64	<u>Ethel/Hwy97</u>			121	1,746
	1,469				1,578	
1,541	8	-			47	1,605
					129	248
	5	302		385		

6		780		882		6
		54	450	276		
1,754	48	<u>Gordon/Hwy97</u>			304	1,949
	1,534				1,572	
1,605	23	-			73	1,993
					128	530
	6	546		841		

**2008 Counts-PM**

4		662		543		4
		51	479	132		
1,687	56	<u>Richter/Hwy97</u>			117	1,702
	1,495				1,531	
1,644	93	-			54	1,698
					105	370
		626		546		

5		455		342		5
		86	200	169		
217	74	<u>Ethel/Hwy97</u>			50	97
	-				-	
139	65	-			47	241
					131	218
		312		421		

6		770		848		6
		59	481	230		
1,797	81	<u>Gordon/Hwy97</u>			165	1,949
	1,673				1,618	
1,804	50	-			166	2,068
					120	602
		697		887		

**% deviation**

		55%	15%	8%		
	13%	<u>Richter/Hwy97</u>			26%	
	7%				10%	
	86%	-			22%	
					35%	34%

		26%	24%	24%		
	14%	<u>Ethel/Hwy97</u>			142%	
	88%	-			0%	
					2%	14%

		8%	6%	20%		
	41%	<u>Gordon/Hwy97</u>			84%	
	8%				3%	
	54%	-			56%	
					7%	12%

## 5.0 APPLICATION

After calibration, future year sub-area models were developed for the 2030 horizon year. The future sub-area models were used to produce O-D trip inputs for the micro simulation model. The model was also directly used to evaluate implications of conversion of two one-way roads (Leon Avenue and Lawrence Avenue) into two-way roads. The road network changes also include turn restrictions at some key intersections. In the future, the model could independently be used to evaluate:

- Alternative packages of road/transit network improvements;
- The transportation impact of various land use scenarios; and
- Alternative transportation demand management strategies

## 6.0 SUMMARY OF FINDINGS

- Demonstrated capability to replicate HCM turn capacities;
- Potential applications include
  - Sub-area model,
  - Local intersection improvements
  - Projection of turn capacities at busy intersections
- Limitations include
  - Intensive coding and error prone
  - Application is limited to short term
  - Detail input required for new or changed traffic control
  - All vehicles are served irrespective of capacity

## Acknowledgment

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