Analysis Procedures Manual Supplemental Materials

The following reference papers and reports are provided for informational purposes only regarding how procedures were developed. Always use the latest version of the Analysis Procedure Manual, or APM, for current procedures.

Contents

Use of Short-term Interval Counts to Determine K Factors: This paper clarifies the traffic count procedures that should be used to determine the K Factor. Data and calculations used in the study can be found in the **K Factor Summary Table**.

K Factor Summary Table

Alternative Traffic Assignment Methods Framework Report: This paper evaluates alternative traffic assignment methods that can be used for various applications in ODOT TPAU's transport models and those of its OMSC partner agencies. For more information refer to APM Chapter 17.

Network Capacity Calculation for Area Type:

- **Report**
- **Field Data and Summary Sheets**
- **Maps: TAZ Density and Average Density Calculations**
- **HCM Capacity Calculations**

Modeling Performance Indicators on Two-Lane Rural Highways: The Oregon Experience: This paper presents a model for estimating follower density on two-lane highways which was found to be superior to current performance measures.

Modeling Follower Density on Two-Lane Rural Highways: This white paper updates a 2010 study and documents a 2013 study to develop an improved follower density-based performance measure and methodology for analysis of two-lane highways in Oregon.

Development of Queue Length Models at Two-way STOP Controlled Intersection: A Surrogate Method: This paper presents a model to estimate queue lengths at two-way stop controlled intersections.

Re-validating and Improving Queue Length Models at Two-Way Stop Controlled Intersections: This paper presents a refined study of queue lengths estimation by lane group at two-way stop controlled intersection approaches.

Queue Lengths at Single Lane Roundabouts in Oregon: This paper summarizes the results of a study of queue lengths at single-lane roundabouts in Oregon and recommends a method that should be used by the Department.

Simulation Guidelines Project - Towards APM Chapter 8 Revision: This project set out to develop a set of rules and criteria for creating, calibrating, and analyzing the results from a microsimulation.

Use of Short-term interval counts to determine K Factors

Don R. Crownover, P.E.
Traffic Monitoring Team Leader
Oregon Department of Transportation

August, 2006

Oregon
Department
of Transportation

Use of Short-term interval counts to determine K Factor

The Question

K factors are basic traffic engineering statistics used throughout highway engineering and planning. For capacity and design purposes the design hour volume is important in representing the amount of traffic occurring at peak times. To derive the design hour volume, the engineer multiplies the average annual daily traffic (AADT) by the K factor. A K factor is the ratio between a peak hour and the ADT. This can be many different factors. The most common is typically referred to as the K-30. It is specifically the ratio between the 30th highest hour and the AADT.

When looking at a graph of the highest hour volumes at a counting station, those at the highest end tend to be outliers, with a steep slope from one to the next. At some point the slope will flatten out to a more reasonable level. Through years of experience, engineers have seen that the thirtieth highest hour occurs often near that point where the slope flattens. So the K-30 is often used in applications because it does a good job of representing the reasonable peak hour. Hours above this are often special events or incidents, too large to reasonably build for.

Many other K factors are available and are used. For a jurisdiction's own use they can choose the K-50, K-100, or simply pick the swing point on the graph. For certain applications, notably the Highway Performance Monitoring System (HPMS), a standard is needed to make it possible to compare locations, even between states. For this reason, the K-30 is mandated in HPMS. The congestion reports can be comparable only if this important factor remains standard.

After reviewing methods used by other states, the author undertook a study to show the ability of short-term counts to produce a K factor that could be used for state and federal purposes. The state of Oregon has approximately 150 Automatic Traffic Recorder (ATR) stations. Those are the only locations where a K-30 is truly measured. K-30s for all other state coverage count locations and HPMS sample locations are assigned by applying engineering judgement about local conditions using those K factors from the ATRs. If interval counts produced reasonable estimates of the K-30 statistic, it would be a good argument to use more interval counters, and have K-30 estimates that are more site-specific.

Current Applicability

Two current topics necessitate an answer to this question. The HPMS is undergoing a reassessment of what data items to collect and use. K Factors are used in the capacity and congestion calculations. A volume to capacity ratio is dependent on the K-30. Possibilities for changes in the HPMS include mixing K-30s from ATRs with short term counts at other sites used as a surrogate.

The other topic is the use of the K Factor in calculations of design hour volumes. The design hour volume is used to determine pavement and geometric aspects of highway design. It is also used in work zone analysis to determine if lanes can be closed or if the traffic will be light enough not to impact mobility.

The Method

An ATR, counting year-round, is the only location where a true K-30 is measured. To determine if short-term counts produced reasonable estimates of the K-30, data was used from fifty-seven ATRs that had no more than five days of data missing from the entire year.

To simulate the short-term counts, samples were taken from the data. Then ADT, high hour, and K factor were determined. For instance, to simulate a 48-hour count such as Oregon collects, data from 11 AM Tuesday to 11 AM Thursday could be used. The 48 hourly volumes would be summed, then the sum divided by two to produce an ADT. The high hour would be chosen. The ratio of the high hour to the ADT is the K factor estimate. The consistency of that statistic throughout the year at any particular station is impressive. The high hours tend to rise and fall with roughly the same seasonality as the volumes.

The K factor estimates were produced for six methodologies. These were three counting styles coupled with two ADT styles.

The counting styles were:

- 48-hour weekday counts. Either Monday to Wednesday, or Tuesday to Thursday. This is Oregon’s short-term count method.
- 72-hour weekday counts. Monday through Friday, with the midnight to midnight data used for Tuesday through Thursday. This is Washington’s short-term count method.
- Seven day counts. Counts started and stopped at the same hour on the same weekday. Count method included for comparison.

The two ADT styles were:

- Use the ADT from the short-term count
- Use the AADT from the station (which would not normally be available)

To create repeatability and reasonability, the counts used were restricted. The counts used these constraints:

- For the Oregon method, only start times on Monday and Tuesday between 6 AM and 7 PM were used
- For the Washington method, counts used a start of zero-hour on Tuesday
- For the seven day method, start times were noon on the five weekdays

- Used data from the week after Spring break (March 22-26, 2004) through October
- Used no statistics that included Memorial Day (May 31, 2004), 4^{th} of July, or Labor Day (September 6, 2004) or the corresponding holiday week
- No extreme incidents – very few incidents were removed from the data
- Ensured no data included any of the few missing days

The Results

Please refer to the attached table (KFactorSummary.xls) during the following explanations.

At each ATR, the table lists the ATR number, name, highway number, functional classification code, missing days, AADT, and the K-30. No station was used that had more than five days missing.

For each of the six calculation methods, there are six statistics presented from each ATR station. Starting at the top is the highest K factor calculated. Second is the average K factor, followed by the lowest K factor. Those factors that were at least as high as the actual K-30 are highlighted.

The next row of statistics is the standard deviation of the K factors computed. The fifth statistic is the corresponding K-level. Since the average K factor is not the K-30, this number shows what the average correlates to. For instance, the first ATR listed, 26-014 has a K-30 factor of 7.4. The first method shows an average K factor of 6.7. Using the list of hourly volumes for the ATR, this volume falls at the 940^{th} highest hour. So instead of a K-30, this corresponds to a K-940. The last statistic shown for each is Percent underestimated. This percentage is how much lower the estimated design hour volume would be if the average estimated K factor were used instead of the measured K-30.

In the third column are averages of the standard deviations, corresponding K-levels and the percent underestimated. Glancing at them it is readily apparent that the best performance is from the seven-day counts. Using seven-day counts raises the average K factor by one or both of two methods. The K factor is a ratio of the design hour to the ADT. That ratio will increase if either the design hour is higher and/or the ADT is lower. If the station is a commuter location with low weekend traffic, the peak hour will likely stay the same while the ADT will be lower because it includes the low weekend days. If the station is recreational, the peak hour could be quite a bit higher, while the weekends account for just two more days and so don't raise the ADT to as great a degree.

Some of the best performers in terms of percent underestimated were the higher volume locations. But these are the locations that tend to have ATRs and are typically not counted with short term tube counts because of safety concerns.

Further research might be useful to see if the better correlation is made to sites at capacity rather than simply that they are a high volume site.

Using Oregon methods, at no stations does the average measured K factor ever reach the true K-30 factor. Using Washington methods, at only one station does the average measured K factor reach the true K-30 factor, using AADT rather than the short term count ADT. The greatest success used seven-day counts. Even this method though did not perform well for most of the counts. The seven day method is also not a common practice because tubes would have a tough time staying in place for that long.

The results using the count ADT were amazingly consistent throughout the count season. They did not match the K-30 well, but repeated the statistic well. This makes sense in terms of the high hours rising and falling in roughly the same pattern as the rise and fall of the total ADT.

The Recommendation

It is the author's opinion that the short-term counts did not perform adequately at representing the K-30. The K factors produced were consistently too low to represent the K-30. For local modeling or design purposes, the short-term counts may produce a factor that is acceptable. But for standardized purposes, such as the HPMS, the short-term counts fall short of the goal.

There are two possible recommendations to make based on this data. For HPMS or other K-30 uses, sound engineering judgement, using knowledge of the local road system, could be used to apply ATR K factors to other road segments.

A K factor that is too low will result in the congestion being under-represented compared to other states that use K-30 factors derived from ATRs.

The second possible recommendation is to not use K-30 as the standard for HPMS. The K factor derived from short term counts remained consistent throughout the count season. It would represent normal peak conditions well. A typical peak could be computed from either short term counts or ATRs.

For construction purposes, the author recommends choosing one method. Mixing K-30s and short term count K factors results in alternate answers. The method should be chosen based on results from past analyses and whether they correctly gave results that did not cost the department or traffic in terms of time or money.

Acknowledgements

The author appreciates receiving Automatic Traffic Recorder data from Oregon, Washington and Idaho departments of Transportation for the preparation of this report. The data managers and crews at all three departments were very helpful.

Actual Measurements	ATR	Averages >	26-014	26-019	26-005	03-011	24-014	22-005	20-008	R021	09-009	22-016	P06
	Name		Hoyt	Minnesota	Stadium Freeway	Wilsonville	Salem Bridges	North Albany	Willakenzie	Spokane Division St	Bend-Revere	Lake Creek	Camas
	Highway		2	1	61	1	30	1	227	WA 2	4	1	WA 14
	Functional Class		11	11	11	01	12	01	11		12	01	
	Missing days		3	0	0	1	1	0	0	0	4	1	0
	AADT		160956	136027	96283	86727	84400	60486	59361	46520	38565	38055	35296
	K-30 Factor		7.4	7.5	8.0	8.9	9.6	9.5	10.1	9.0	10.4	10.5	9.3
Using Count ADT													
Oregon Procedures	Highest K Factor		7.0	7.2	9.9	12.8	14.9	9.0	14.8	8.4	9.2	8.6	8.6
	Average K Factor		6.7	6.8	7.2	7.7	8.9	7.9	9.0	7.9	8.8	7.7	8.1
	Lowest K Factor		6.4	6.5	6.7	7.1	7.5	7.4	8.3	7.4	8.4	7.2	7.7
	Standard Deviation	0.6	0.3	0.1	0.4	0.6	1.0	0.3	0.8	0.2	0.2	0.2	0.2
	Corresponding K-Level	646	940	831	871	529	339	504	484	981	633	792	695
	Percent Underestimated	27	9	9	10	13	7	17	11	12	15	27	13
Washington Procedures	Highest K Factor		6.8	7.0	8.4	11.2	12.9	8.7	12.7	8.2	9.1	8.5	8.6
	Average K Factor		6.6	6.8	7.1	7.8	9.0	8.2	9.1	7.9	8.8	8.0	8.1
	Lowest K Factor		6.4	6.6	6.8	7.3	8.3	7.7	8.4	7.6	8.4	7.6	7.8
	Standard Deviation	0.5	0.1	0.1	0.3	0.7	1.0	0.3	1.0	0.2	0.2	0.2	0.2
	Corresponding K-Level	563	1137	831	1031	460	305	326	448	981	633	595	695
	Percent Underestimated	25	11	9	11	12	6	14	10	12	15	24	13
Seven Day Counts	Highest K Factor		7.3	7.6	8.3	10.1	10.9	10.1	14.9	9.1	10.3	11.1	9.3
	Average K Factor		7.1	7.2	7.7	8.5	9.4	9.1	10.1	8.6	9.8	9.4	8.8
	Lowest K Factor		6.9	6.9	7.4	7.7	8.8	8.3	9.3	8.0	9.2	8.6	8.3
	Standard Deviation	0.9	0.1	0.2	0.2	0.5	0.5	0.6	0.9	0.3	0.2	0.6	0.2
	Corresponding K-Level	181	333	239	202	114	80	89	30	163	177	150	272
	Percent Underestimated	14	4	4	4	4	2	4	0	4	6	10	5
Using Station AADT													
Oregon Procedures	Highest K Factor		7.5	7.7	8.3	8.6	9.7	8.9	10.1	8.7	10.7	9.1	9.4
	Average K Factor		7.2	7.3	7.9	7.7	9.3	7.8	9.5	8.2	9.9	7.5	9.0
	Lowest K Factor		6.9	7.0	7.3	6.9	7.7	7.2	6.0	7.9	9.1	6.5	8.4
	Standard Deviation	1.1	0.1	0.1	0.2	0.3	0.7	0.4	0.9	0.2	0.3	0.6	0.2
	Corresponding K-Level	503	207	140	55	529	127	572	271	505	131	914	131
	Percent Underestimated	24	3	3	1	13	3	18	6	9	5	29	3
Washington Procedures	Highest K Factor		7.5	7.7	8.3	8.6	9.9	9.3	10.6	8.7	10.7	9.1	9.4
	Average K Factor		7.3	7.4	7.9	8.0	9.5	8.3	9.8	8.4	10.1	8.1	9.1
	Lowest K Factor		7.0	7.2	7.6	7.2	9.0	7.7	6.2	8.0	9.5	7.0	8.4
	Standard Deviation	1.1	0.1	0.1	0.2	0.3	0.4	0.4	0.7	0.2	0.3	0.6	0.2
	Corresponding K-Level	363	100	74	55	325	49	293	96	296	79	553	83
	Percent Underestimated	21	1	1	1	10	1	13	3	7	3	23	2
Seven Day Counts	Highest K Factor		7.5	7.7	8.3	9.2	13.5	10.1	10.6	9.3	10.8	11.5	9.6
	Average K Factor		7.3	7.5	8.0	8.7	8.5	9.5	10.0	8.8	10.2	9.9	9.2
	Lowest K Factor		7.0	7.2	7.6	7.7	7.2	8.9	6.2	8.2	9.5	9.0	8.6
	Standard Deviation	1.7	0.1	0.1	0.2	0.3	0.2	0.3	0.7	0.3	0.3	0.6	0.2
	Corresponding K-Level	83	100	32	34	66	462	33	43	79	58	79	52
	Percent Underestimated	7	1	0	0	2	11	0	1	2	2	6	1

Actual Measurements	ATR	270	36-004	09-020	D12	10-007	09-003	219	24-005	211	16-002	17-005	05-006	08-005
	Name	Eagle Rd	Newberg	Redmond	Covington	Oakland	Lava Butte	West ParkCenter	Aumsville	Warm Springs	Madras	Timber Ridge	Rainier	Winchuck
	Highway	ID SH 55	91	4	WA 18	1	4	ID STP 9493	162	ID SMA 7383	4	25	92	9
	Functional Class	14	14	02		01	14	14	02	16	02	02	02	02
	Missing days	0	14	4	1	0	0	0	0	0	0	0	0	0
	AADT	33726	33463	28647	26371	23093	22128	20719	20482	14033	13143	11860	9950	9633
	K-30 Factor	9.7	8.8	10.2	9.9	10.5	10.8	12.1	10.6	10.7	11.1	9.8	12.0	10.9
Using Count ADT														
Oregon Procedures	Highest K Factor	8.9	8.6	9.4	8.7	9.1	9.5	11.0	9.4	10.7	9.5	11.4	9.9	9.3
	Average K Factor	8.4	8.1	8.7	8.1	7.8	8.7	10.2	8.7	9.6	8.6	8.4	8.5	8.6
	Lowest K Factor	7.7	7.7	8.3	7.6	7.3	8.3	9.5	8.2	8.7	7.9	7.7	7.6	8.1
	Standard Deviation	0.3	0.2	0.2	0.2	0.2	0.2	0.3	0.2	0.4	0.3	0.6	0.3	0.2
	Corresponding K-Level	515	384	563	571	778	503	353	478	189	595	546	617	730
	Percent Underestimated	13	8	15	18	26	19	16	18	10	23	14	29	21
Washington Procedures	Highest K Factor	8.7	8.5	9.2	8.7	9.1	9.4	10.6	9.5	10.5	9.3	10.2	9.6	9.2
	Average K Factor	8.4	8.2	8.8	8.3	8.0	8.8	10.2	8.9	9.7	8.8	8.4	8.7	8.7
	Lowest K Factor	7.6	7.8	8.4	7.9	7.6	8.4	9.6	8.4	9.0	8.3	7.8	8.1	8.4
	Standard Deviation	0.3	0.2	0.2	0.2	0.3	0.2	0.2	0.3	0.4	0.3	0.5	0.3	0.2
	Corresponding K-Level	515	331	523	417	669	459	353	388	173	498	546	529	661
	Percent Underestimated	13	7	14	16	24	19	16	16	9	21	14	28	20
Seven Day Counts	Highest K Factor	9.6	9.0	10.3	10.1	9.7	10.5	12.5	10.4	11.7	10.8	10.6	11.5	10.4
	Average K Factor	8.9	8.5	9.6	9.4	8.9	9.7	11.7	9.7	10.1	9.9	9.1	10.0	9.5
	Lowest K Factor	8.4	8.0	9.1	8.5	8.3	8.9	10.9	9.2	9.3	9.1	8.3	9.0	8.9
	Standard Deviation	0.3	0.3	0.3	0.5	0.3	0.3	0.3	0.3	0.5	0.5	0.5	0.5	0.3
	Corresponding K-Level	268	138	160	92	248	160	114	139	102	164	185	179	306
	Percent Underestimated	8	3	6	5	15	10	3	8	6	11	7	17	13
Using Station AADT														
Oregon Procedures	Highest K Factor	9.7	8.9	11.3	9.6	9.8	11.4	12.7	10.3	12.6	10.7	10.0	10.2	11.0
	Average K Factor	9.3	8.5	9.7	8.5	7.8	9.5	11.9	9.2	10.5	9.1	8.9	8.6	9.0
	Lowest K Factor	8.8	8.0	9.0	8.0	6.7	8.2	11.0	8.4	9.2	7.9	8.0	7.6	7.0
	Standard Deviation	0.3	0.2	0.4	0.2	0.8	0.7	0.4	0.4	0.6	0.6	0.5	0.6	1.0
	Corresponding K-Level	118	138	119	297	778	207	69	275	49	358	259	571	499
	Percent Underestimated	4	3	5	14	26	12	2	13	2	18	9	28	17
Washington Procedures	Highest K Factor	9.8	8.9	11.3	9.6	9.5	11.4	12.6	10.6	12.6	10.7	10.0	10.3	11.0
	Average K Factor	9.4	8.6	9.9	8.9	8.3	9.8	12.0	9.6	10.8	9.5	9.1	9.0	9.2
	Lowest K Factor	8.8	8.2	9.5	8.4	7.2	8.6	11.1	8.8	10.1	8.1	8.3	8.1	7.9
	Standard Deviation	0.3	0.2	0.4	0.3	0.8	0.8	0.3	0.5	0.5	0.7	0.4	0.6	0.9
	Corresponding K-Level	86	86	64	161	488	139	51	162	24	262	185	395	399
	Percent Underestimated	3	2	3	10	21	9	1	9	-1	14	7	25	16
Seven Day Counts	Highest K Factor	10.0	9.0	11.3	10.3	11.1	12.1	12.7	11.8	12.6	11.7	10.5	12.6	11.8
	Average K Factor	9.5	8.8	10.1	9.9	9.4	10.5	12.0	10.4	10.8	10.9	9.6	10.7	10.0
	Lowest K Factor	8.9	8.4	9.6	9.6	8.2	9.3	11.1	9.6	10.0	9.4	9.0	9.1	8.9
	Standard Deviation	0.3	0.2	0.4	0.2	0.9	0.8	0.3	0.6	0.5	0.6	0.4	1.1	1.0
	Corresponding K-Level	66	37	39	30	156	47	51	46	24	42	58	91	167
	Percent Underestimated	2	0	1	0	10	3	1	2	-1	2	2	11	8

Actual Measurements	ATR	09-015	01-011	23-016	15-013	27-005	10	16-006	20-005	10-006	P07	18-022
	Name	3 Sisters Viewpoint	Baker Valley	Huntington	Shady Cove	Monmouth	Dry Creek	Warm Springs	Noti	Brockway	Washougal	Modoc Point
	Highway	17	6	6	22	91	ID SH 55	53	62	35	WA 14	4
	Functional Class	02	01	01	06	06	14	02	02	02		02
	Missing days	0	4	4	0	0	0	0	5	0	7	4
	AADT	9342	9236	8691	8087	7615	7190	7125	6431	6277	6150	6092
	K-30 Factor	12.3	11.4	11.2	10.9	11.3	16.4	13.3	12.7	9.8	10.3	10.8
Using Count ADT												
Oregon Procedures	Highest K Factor	9.7	10.4	9.7	9.1	10.0	10.0	9.8	10.2	9.2	9.4	10.1
	Average K Factor	8.7	7.4	7.4	8.3	9.2	8.7	8.5	8.7	8.0	8.4	8.4
	Lowest K Factor	8.1	6.9	6.8	7.6	8.5	7.9	7.7	7.8	7.4	7.5	7.4
	Standard Deviation	0.3	0.4	0.3	0.3	0.3	0.5	0.4	0.4	0.3	0.4	0.4
	Corresponding K-Level	778	1117	917	643	337	1175	782	687	629	468	677
	Percent Underestimated	29	35	34	24	19	47	36	31	18	18	22
Washington Procedures	Highest K Factor	9.5	10.1	9.5	8.9	9.8	10.2	10.1	10.1	8.9	9.3	9.7
	Average K Factor	9.1	7.8	7.8	8.4	9.2	9.3	9.0	8.9	8.1	8.6	8.5
	Lowest K Factor	8.6	7.1	7.2	7.9	8.7	8.6	8.6	8.1	7.6	7.9	7.8
	Standard Deviation	0.2	0.4	0.4	0.2	0.3	0.5	0.4	0.5	0.3	0.4	0.4
	Corresponding K-Level	604	832	679	596	337	663	589	584	566	357	626
	Percent Underestimated	26	32	30	23	19	43	32	30	17	17	21
Seven Day Counts	Highest K Factor	12.9	10.1	12.9	10.5	14.7	13.3	12.5	11.3	10.0	14.6	10.3
	Average K Factor	10.6	8.8	8.8	9.6	10.6	12.0	11.4	10.3	9.1	9.5	9.3
	Lowest K Factor	9.6	7.8	7.9	8.7	9.2	10.4	10.3	9.4	8.4	8.2	8.3
	Standard Deviation	0.7	0.5	0.8	0.4	1.4	0.7	0.5	0.4	0.4	1.1	0.5
	Corresponding K-Level	175	329	247	148	68	229	150	223	124	120	275
	Percent Underestimated	14	23	21	12	6	27	14	19	7	8	14
Using Station AADT												
Oregon Procedures	Highest K Factor	11.0	10.5	10.9	10.1	11.0	10.5	10.6	12.4	10.1	9.8	12.0
	Average K Factor	9.0	7.9	7.6	8.8	9.6	8.5	8.2	8.5	8.4	8.6	9.2
	Lowest K Factor	7.2	6.4	4.2	7.7	8.8	6.6	6.9	7.0	7.1	7.7	7.6
	Standard Deviation	0.9	0.9	1.1	0.5	0.5	1.0	0.8	1.0	0.5	0.4	0.8
	Corresponding K-Level	644	759	797	396	220	920	911	791	358	357	299
	Percent Underestimated	27	31	32	19	15	48	38	33	14	17	15
Washington Procedures	Highest K Factor	11.0	10.5	10.9	10.1	11.0	12.9	10.4	12.4	9.4	9.8	12.0
	Average K Factor	9.6	8.6	8.3	9.1	9.9	9.4	9.0	9.0	8.6	9.0	9.6
	Lowest K Factor	7.9	7.0	4.6	7.8	9.1	7.0	7.8	7.5	7.7	8.1	8.1
	Standard Deviation	0.9	1.0	1.2	0.6	0.4	1.5	0.8	1.2	0.4	0.4	0.8
	Corresponding K-Level	406	397	402	291	151	638	589	547	275	227	188
	Percent Underestimated	22	25	26	17	12	43	32	29	12	13	11
Seven Day Counts	Highest K Factor	14.8	12.1	12.0	12.6	15.4	17.9	15.2	14.4	10.3	17.2	12.0
	Average K Factor	11.7	9.7	9.4	10.4	11.2	14.2	12.5	11.1	9.5	10.3	10.3
	Lowest K Factor	9.4	7.9	7.9	8.7	9.3	9.2	10.5	8.9	8.6	8.9	8.4
	Standard Deviation	1.2	1.3	1.1	1.2	1.6	2.6	1.6	1.6	0.5	1.6	0.9
	Corresponding K-Level	63	127	145	49	32	106	64	123	58	29	66
	Percent Underestimated	5	15	16	5	1	13	6	13	3	0	5

Actual Measurements	ATR	03-009	02-007	24-013	24-015	80	03-013	17-003	02-003	03-007	03-008	18-017	22-017	14-003	12-009
	Name	Warm Springs Jct	Monroe	Gates	Detroit	Eckert Rd	Marquam	O'Brien	Alsea	Mt Hood Meadows	Timberline	Beatty	Upper Soda	Mt. Hood	Prairie City
	Highway	53	91	162	162	ID SMA 7643	160	25	27	26	173	20	16	26	5
	Functional Class	02	06	02	02	16	06	02	06	02	07	02	02	02	02
	Missing days	3	4	0	1	0	1	0	4	5	0	0	2	5	1
	AADT	5113	5051	4990	4294	4115	3683	3010	2355	1685	1554	1191	1148	1097	1025
	K-30 Factor	20.6	10.9	23.2	22.8	11.9	10.8	17.3	10.8	34.8	26.7	12.3	21.4	22.2	15.2
Using Count ADT															
Oregon Procedures	Highest K Factor	11.5	11.4	11.6	11.9	15.0	10.9	10.5	12.2	17.3	22.1	10.1	14.9	17.6	11.4
	Average K Factor	9.0	8.7	9.2	9.5	10.6	9.5	9.3	8.9	11.4	12.8	8.7	10.6	11.7	9.4
	Lowest K Factor	7.5	7.7	8.1	8.5	8.9	8.3	8.3	7.6	9.3	8.8	7.7	8.7	9.0	8.0
	Standard Deviation	0.7	0.5	0.6	0.6	0.9	0.5	0.5	0.6	1.1	1.8	0.5	1.1	1.4	0.7
	Corresponding K-Level	968	425	977	954	172	287	910	397	752	650	743	817	470	838
	Percent Underestimated	56	20	60	58	11	12	46	18	67	52	29	50	47	38
Washington Procedures	Highest K Factor	10.4	9.7	11.3	11.8	12.0	10.8	10.1	9.6	15.0	16.5	10.0	13.6	16.2	11.2
	Average K Factor	9.5	8.9	10.0	10.2	10.7	9.7	9.5	8.9	12.0	13.0	9.2	11.3	12.5	9.9
	Lowest K Factor	8.3	8.4	9.3	9.3	9.2	8.8	8.5	8.3	10.3	10.7	8.2	9.7	10.0	8.6
	Standard Deviation	0.6	0.4	0.5	0.6	0.7	0.5	0.4	0.4	1.2	1.5	0.5	1.0	1.5	0.8
	Corresponding K-Level	842	338	791	797	160	230	847	397	687	611	510	688	391	692
	Percent Underestimated	54	18	57	55	10	10	45	18	66	51	25	47	44	35
Seven Day Counts	Highest K Factor	17.2	14.7	18.8	19.0	15.9	11.3	14.7	13.4	25.2	29.5	13.7	21.0	22.5	13.5
	Average K Factor	14.9	10.5	16.5	16.4	10.8	10.2	12.1	10.0	18.5	20.5	10.0	16.8	18.0	10.9
	Lowest K Factor	13.3	9.0	14.1	14.8	9.4	9.4	10.7	8.6	12.9	12.9	8.9	13.1	13.5	9.3
	Standard Deviation	0.8	1.5	0.9	0.9	0.7	0.5	0.9	0.9	2.6	3.8	0.9	2.0	2.0	0.9
	Corresponding K-Level	164	56	145	149	149	108	277	84	242	119	266	143	100	419
	Percent Underestimated	28	4	29	28	9	6	30	7	47	23	19	21	19	28
Using Station AADT															
Oregon Procedures	Highest K Factor	13.1	10.4	12.9	13.9	20.5	11.1	13.5	13.3	11.0	18.7	14.1	13.5	17.3	14.3
	Average K Factor	7.8	8.9	8.0	8.2	11.3	9.9	8.9	9.1	6.9	9.7	9.9	8.7	8.6	10.3
	Lowest K Factor	4.8	6.8	4.8	5.1	9.6	8.6	5.9	7.6	3.6	3.4	6.7	4.9	4.8	6.0
	Standard Deviation	1.9	1.0	2.0	2.0	1.5	0.5	3.2	0.9	1.9	4.5	1.5	2.2	2.5	2.0
	Corresponding K-Level	1373	338	1376	1352	80	179	1047	318	1716	1092	309	1322	1049	566
	Percent Underestimated	62	18	66	64	5	8	49	16	80	64	20	59	61	32
Washington Procedures	Highest K Factor	12.6	10.9	12.9	13.9	13.6	11.5	13.0	10.4	11.0	18.7	14.1	13.5	17.3	14.3
	Average K Factor	8.6	9.2	9.1	9.2	11.5	10.4	9.2	9.3	7.6	9.7	10.7	9.7	9.4	11.1
	Lowest K Factor	5.5	7.1	6.2	5.8	10.2	9.5	6.2	8.0	4.1	4.7	6.9	6.0	5.9	6.0
	Standard Deviation	2.0	1.4	2.2	2.3	0.8	0.5	2.1	0.7	1.1	4.4	1.7	2.4	2.6	2.1
	Corresponding K-Level	1095	243	1007	1015	186	75	944	233	1436	1092	140	1007	867	374
	Percent Underestimated	58	16	61	60	3	4	47	14	78	64	13	55	58	27
Seven Day Counts	Highest K Factor	22.9	13.0	27.4	26.4	20.5	11.9	20.4	14.6	22.4	36.7	17.2	25.3	24.2	15.5
	Average K Factor	17.3	10.9	19.4	19.2	11.8	10.8	13.7	10.6	15.9	19.6	11.5	19.8	16.7	12.6
	Lowest K Factor	12.9	8.9	13.7	13.3	10.3	9.9	9.0	9.1	9.2	10.2	7.9	13.9	9.8	8.7
	Standard Deviation	3.5	0.9	4.6	4.3	1.1	0.4	3.2	1.2	4.1	6.1	2.1	3.3	4.0	2.1
	Corresponding K-Level	85	28	71	72	41	34	140	41	365	144	27	54	143	130
	Percent Underestimated	16	0	16	16	1	0	21	2	54	27	7	7	25	17

Actual Measurements	ATR	19-008	01-001	19-010	11-007	26-012	13-001	18-021	11-004
	Name	New Pine	North Powder	Silver Lake	Shutler	Bridal Veil	Burns	Ft. Klamath	Condon
	Highway	19	66	19	5	100	48	22	300
	Functional Class	02	07	06	06	07	02	07	07
	Missing days	0	0	0	0	1	0	1	0
	AADT	993	823	814	787	697	575	521	201
	K-30 Factor	13.5	11.4	14.7	12.1	36.9	14.4	22.3	15.9
Using Count ADT									
Oregon Procedures	Highest K Factor	11.3	11.5	12.6	10.7	21.3	12.6	14.0	15.8
	Average K Factor	9.5	9.7	9.5	8.7	14.7	9.9	10.6	12.2
	Lowest K Factor	8.3	7.9	7.8	7.1	11.4	8.1	8.7	9.6
	Standard Deviation	0.6	0.7	0.7	0.8	2.1	0.8	1.2	1.3
	Corresponding K-Level	654	267	591	836	622	584	1068	202
	Percent Underestimated	30	15	35	28	60	31	52	23
Washington Procedures	Highest K Factor	11.2	10.8	11.5	10.8	20.7	12.3	13.1	15.2
	Average K Factor	9.8	9.8	9.6	9.1	15.3	10.3	10.7	12.4
	Lowest K Factor	8.8	8.6	8.5	7.7	12.1	8.8	8.9	10.0
	Standard Deviation	0.6	0.7	0.7	0.8	2.3	0.9	1.3	1.4
	Corresponding K-Level	518	240	551	612	560	442	1068	202
	Percent Underestimated	27	14	35	25	59	28	52	22
Seven Day Counts	Highest K Factor	13.8	13.8	16.9	14.5	50.9	19.2	17.4	31.6
	Average K Factor	10.9	10.6	11.2	11.1	26.7	11.8	12.6	15.0
	Lowest K Factor	9.5	9.3	9.3	9.2	19.0	9.7	9.6	11.8
	Standard Deviation	0.6	1.0	1.5	1.0	6.6	2.0	1.8	3.7
	Corresponding K-Level	226	102	200	82	149	182	743	43
	Percent Underestimated	19	7	24	8	28	18	43	6
Using Station AADT									
Oregon Procedures	Highest K Factor	14.1	12.2	15.5	15.6	22.1	16.2	20.0	19.4
	Average K Factor	10.6	10.5	10.2	11.2	12.7	11.0	12.9	13.1
	Lowest K Factor	7.7	8.7	7.5	9.5	5.9	7.5	5.4	9.5
	Standard Deviation	1.2	0.9	1.5	0.9	3.8	1.6	4.3	2.1
	Corresponding K-Level	298	114	380	71	822	283	694	128
	Percent Underestimated	21	8	31	7	66	24	42	18
Washington Procedures	Highest K Factor	14.1	12.4	15.5	15.6	18.4	16.2	20.0	19.4
	Average K Factor	11.1	10.9	10.6	11.7	13.0	11.8	13.0	13.6
	Lowest K Factor	9.1	9.6	8.4	10.3	7.7	8.7	6.1	9.5
	Standard Deviation	1.4	0.8	1.7	1.2	3.5	1.9	4.2	2.4
	Corresponding K-Level	194	73	290	43	790	182	694	96
	Percent Underestimated	18	4	28	3	65	18	42	14
Seven Day Counts	Highest K Factor	16.1	15.3	19.9	15.6	59.0	22.4	25.3	36.8
	Average K Factor	12.4	11.4	12.9	12.0	32.9	13.8	15.9	16.2
	Lowest K Factor	9.4	10.0	10.1	9.9	18.4	9.6	6.3	10.4
	Standard Deviation	1.7	1.0	2.5	1.3	9.1	2.9	5.4	5.0
	Corresponding K-Level	71	30	55	32	64	47	294	29
	Percent Underestimated	8	0	12	1	11	4	29	-2

Avoiding:
Memorial Day - May 31, 2004
Independence Day - July 4, 2004
Labor Day - September 6, 2004

Method	
Oregon	Start times 6AM to 7PM Monday and Tuesday
Washington	Start time Zero hour Tuesday
7 Day	Start times Noon hour M-F

ALTERNATIVE TRAFFIC ASSIGNMENT METHODS FRAMEWORK REPORT

Oregon Department of Transportation
Transportation Planning Analysis Unit

May 2018

TECHNICAL REPORT #1
ALTERNATIVE TRAFFIC ASSIGNMENT METHODS FRAMEWORK REPORT

Prepared by:
DKS Associates, Inc.

Study Working Group:
Alex Bettinardi – ODOT TPAU
Brian Dunn – ODOT TPAU
Peter Schuytema - ODOT TPAU
Doug Norval - ODOT TPAU
Christi McDaniel-Wilson – ODOT TPAU
Jin Ren Jinxiang - ODOT TPAU
Chi Mai – ODOT Region 1
Ray Jackson – Mid-Willamette Council of Governments
Oregon Metro – Peter Bosa
Tyler Deke – Bend Metropolitan Planning Organization

 This project was funded in part by the Federal Highway Administration, U.S. Department of Transportation.

For more information, contact:
Alex Bettinardi
Transportation Modeling Program Manager
Oregon Department of Transportation
555 13th Street NE
Salem, Oregon 97301-4178
Telephone: 503.986.4104
E-mail: alexander.o.bettinardi@odot.state.or.us

Contents

List of Figures

List of Tables

1.0 Executive Summary

The purpose of the Alternative Traffic Assignment Methods Study was to investigate alternative traffic assignment methods that can be used for various applications in ODOT TPAU's transport models and those of its OMSC partner agencies. The investigation was limited to traffic assignment only, and did not include transit assignment or other model components.

The study consisted of the following main steps:

- Step 1 – Identify Alternative Assignment Methods
- Step 2 – Evaluate Assignment Methods
- Step 3 – Investigate Assignment-Related Topics

Input was obtained from the study working group on the assignment method objectives, the alternative methods to be evaluated and the evaluation criteria to be used, and assignment-related topics. The group consisted of staff from ODOT TPAU, ODOT Region 1, Oregon Metro, the Mid-Willamette Valley COG, and the Bend MPO.

1.1 Assignment Methods

The primary components of traffic assignment models are a route choice model and a network flow model. A route choice model determines the trip-maker's path selection between origin and destination zones. These models can be classified according to the method of route choice (deterministic or stochastic), the treatment of time (static or dynamic), and the level of vehicle aggregation (microscopic, macroscopic, and mesoscopic). Network flow models define how links and nodes perform under congested conditions. Static network flow models assume that route flows propagate instantaneously through the network. Dynamic flow models move traffic through the network in time slices or distinctive time periods. Microscopic flow models consider vehicles separately and simulate specific characteristics of driver behavior. Mesoscopic flow models are hybrids of macroscopic and microscopic models.

Several of the more frequently found combinations of these model dimensions are:

- Static, macroscopic
- Dynamic, macroscopic
- Dynamic, microscopic or mesoscopic

All of these are equilibrium assignment methods in which demand is distributed according to Wardrop's first principle which states, “Every road user selects his route in such a way that the travel time on all alternative routes is the same, and that switching to a different route would increase personal travel time."

1.2 Evaluation of Assignment Methods

The assignment method objectives defined by the study working group describe the desired properties and capabilities of the assignment methods as well as the types of applications the methods are to be used for. Evaluation criteria were defined that would allow the objectives to be reflected in the comparison of the alternative methods. The criteria were applied to the three assignment methods listed above:

- Method 1 – Static, macroscopic assignment
- Method 2 – Dynamic, macroscopic assignment
- Method 3 – Dynamic, microscopic/mesoscopic assignment

Ratings of “low”, “medium”, or “high” were assigned for each criterion.

The evaluation was <u>not</u> intended to rank the methods to identify a “best” method, but rather to to establish a general framework for considering the advantages and disadvantages of the methods. If there is interest in determining which method may be the most appropriate for a particular urban area, travel demand forecasting model, or model application, the methods can be quantitatively ranked by using a combination of weights and numeric scores for the criteria to develop a weighted total score for each method. A summary of the evaluation results is shown below.

Assignment Method Rating Summary

Rating	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso
Low	28 60.9%	10 21.7%	10 21.7%
Medium	9 19.6%	33 71.7%	15 32.6%
High	9 19.6%	3 6.5%	21 45.7%
Total No. of Criteria	46 (100%)	46 (100%)	46 (100%)

Reading across the diagonal of the table, it can be seen that overall, Method 1 received the highest number of "low" ratings, Method 2 had the highest number of "medium" ratings, and Method 3 received the largest number of "high" ratings. The main reasons that Methods 2 and 3 rated relatively well compared to Method 1 are:

- Better accuracy of link traffic flows;
- Better representation of travel times/speeds;
- More realistic estimation of intersection delay;
- Higher levels of temporal resolution;
- More realistic representation of traffic operations characteristics;
- Ability to represent peak spreading;
- More accurate estimation of the effects of capacity improvements;
- Ability to reflect the effects of small-scale improvements, such as TSMO-type improvements; and
- Better ability to support project selection.

These advantages are primarily related to the more realistic representation of network response to congestion (in terms of delay), the higher level of temporal resolution and, in the case of Method 3, the higher network resolution. The higher overall rating for Method 3 compared Method 2 is also largely accounted for by the greater advantages of Method 3 in these areas.

There are several criteria, however, for which Methods 2 and 3 were rated lower than Method 1. This is related to the following disadvantages of Methods 2 and 3:

- Larger implementation and maintenance time requirements;
- Larger data collection time requirement;
- Less intuitive understanding of the assignment method processes;
- Greater difficulty in interpreting the cause-effect relationships in the assignment outputs;
- Higher level of staff expertise for application and maintenance; and
- Lower degree of assignment convergence.

All of these disadvantages, except the last one, are due to the greater complexity of Methods 2 and 3. The lower degree of assignment convergence is related to the constrained physical capacities used in the these methods compared to the continuous VDFs used in Method 1, which result in the spillover of traffic to adjacent time periods if capacity is exceeded. Also with Methods 2 and 3, fractional vehicles are used in the assignment which reduces the level of convergence possible compared to Method 1, which uses whole vehicles.

1.3 Assignment-Related Topics

Specific topics related to the implementation and use of the assignment methods were identified for investigation by the study working group. The focus of the investigation was on static assignment topics because this will likely be the main method used by ODOT and its partner agencies within the near-term. Brief highlights for each topic are summarized below.

Static, Macroscopic Assignment Topics

<u>Alternative Forms of Volume Delay Functions (VDFs)</u> - The most commonly used VDFs are the Bureau of Public Roads (BPR) function, Davidson's delay model, the Akcelik function, and the conical delay model.

<u>Incorporation of Node-Based Delay in VDFs</u> – Potential advantages of VDFs that include an intersection delay component are more accurate estimation of link traffic flows and travel times/speeds, more accurate estimation of intersection delay, and better representation of the effects of capacity improvements.

<u>Calibration of VDFs</u> – In a study conducted by Florida A&M University - Florida State University, traffic volume and speed data from the Florida Department of Transportation's traffic monitoring stations and statewide transportation engineering warehouse were used to calibrate four types of VDFs across four facility types and three area types.

<u>Representation of Truck Volumes in VDFs</u> - The travel time functions of trucks and cars are not identical, and furthermore depend on not only traffic volume, but traffic composition as well. Thus, there is a need for travel time functions that consider both the volume and proportion of trucks in the traffic stream.

<u>Representation of Network Capacity</u> - The treatment of network capacity varies according to assignment method used. Capacities can be more realistically represented with the dynamic, macroscopic method, providing more accurate and detailed information about capacity improvements compared to the static, macroscopic method. Capacities are an output, not an input, of dynamic, microscopic/mesoscopic models.

<u>Generalized Cost Assignment</u>

Generalized cost assignment attempts to more realistically represent the traveler's path decision-making process by including other factors in addition to travel time. Since the generalized cost for a specific link must be expressed as a single value, all of the factors not measured in monetary units must be converted to a constant dollar amount.

Dynamic, Macroscopic Assignment Topics

Calibration of Dynamic Traffic Assignment (DTA) Models

DTA model results are influenced primarily by the model network, input demand, and the type of queuing model used. Once it has been confirmed that the input demand is reasonable, a multi-step process is followed to calibrate the model to produce estimates of network operating characteristics such as link flows and queue lengths.

Level of Network Disaggregation

DTA model networks are generally more data-intensive than static model networks. For example, DTA requires information on the number of lanes on each link, the presence of acceleration–deceleration lanes and turn bays, and lane connectivity.

Level of Time Resolution

One of the main advantages of DTA models compared to static models is that the higher level of temporal resolution allows a more realistic representation of network response to congestion, in terms of delay. Therefore, relatively short time periods for assignment are used, typically ranging from five to 15 minutes.

Dynamic, Microscopic/Mesoscopic Assignment Topics

Calibration of Microsimulation Models

Calibration of a microsimulation model is the adjustment of parameters to improve the model's ability to reproduce local driver behavior and traffic performance characteristics. Every microsimulation software program comes with a set of user-adjustable parameters for calibrating the model to local conditions.

Size of the Modeling Problem

The additional cost of developing microsimulation models tends to limit their use to a subregional level. In a study of best practices in microsimulation, it was estimated that mesoscopic simulation models tend to cost an order of magnitude (i.e., ten times) more to develop than macroscopic models.[1] On a similar scale, microscopic simulation models tend to cost an additional order of magnitude more to develop than mesoscopic models.

General Assignment Topics

Development of Multi-Resolution Modeling Networks

Multi-resolution modeling (MRM) is the integration of macroscopic, mesoscopic, and microscopic models for the purpose of analyzing transportation projects at different levels of detail by enabling data to be shared across modeling platforms. The objectives in the development of networks at each modeling scale are maintaining consistency between the networks, accuracy, and minimization of effort.

Consideration of Travel Time Reliability in Traffic Assignment

Typically in the traffic assignment process, it is assumed that travelers only consider average travel time when making route choice decisions. However, many empirical studies have shown that travelers also take travel time reliability into consideration when making trip decisions. As a result, the identified optimal paths from traditional models may fail to represent most travelers' risk averse behaviors.

Cost Effectiveness of Dynamic vs. Static Assignment Methods

If congestion can be expected to be low then there is little added value in a detailed accounting of it in the model system. Given that without congestion there is only limited physical connection between the network conditions of different time slices, static network assignment may be fully adequate.

[1] Best Practices in the Use of Micro Simulation Models, American Association of State Highway and Transportation Officials (AASHTO), 2010.

Use of Demand Adjustment Procedures

Demand adjustment is a procedure used to update a seed origin-destination matrix using traffic counts. This procedure is most often applied to adjust base year trip matrices to better fit existing conditions, as reflected in the count data.

2.0 Introduction

The purpose of the Alternative Traffic Assignment Methods Study was to investigate alternative traffic assignment methods that can be used for various applications in ODOT TPAU's transport models and those of its OMSC partner agencies. The investigation was limited to traffic assignment only, and did not include transit assignment or other model components.

The study consisted of the following main steps:

- Step 1 – Identify Alternative Assignment Methods
- Step 2 – Evaluate Assignment Methods
- Step 3 – Investigate Assignment-Related Topics

At an initial meeting of the study working group, a list of assignment method objectives was identified. The working group consisted of staff from ODOT TPAU, ODOT Region 1, Oregon Metro, the Mid-Willamette Valley COG, and the Bend MPO. The objectives were used as a guide for the identification of alternative assignment methods, based on a literature review of currently-used methods.

The alternative methods were reviewed at a second working group meeting to determine the methods to be evaluated in Step 2, as well as a proposed set of evaluation criteria reflecting the assignment method objectives. Suggested assignment-related topics to be investigated in Step 3 were also reviewed at the meeting.

In Step 2 of the study, the assignment methods were evaluated using the criteria defined in Step 1. The evaluation was done by assigning ratings of "low", "medium", or "high" to the alternatives for each of the evaluation criteria.

Specific topics related to the implementation of the assignment methods were investigated in Step 3 of the study.

This report is organized according to the following sections:

- Section 1: Assignment Methods – General discussion of assignment model components and the categories of assignment methods.
- Section 2: Evaluation of Assignment Methods – Description of the assignment method objectives and evaluation criteria, evaluation procedure, and evaluation results.
- Section 3: Assignment-Related Topics – General discussion of topics related to implementation of the assignment methods.

3.0 Assignment Methods

3.1 Assignment Model Components

The primary components of traffic assignment models are a route choice model and a network flow model.

Route Choice Models

A route choice model determines the trip-maker's path selection between origin and destination zones. The choice of a route is based on the route's properties, typically cost in the form of travel time.

Route choice models can be classified according to the method of route choice, the treatment of time, and the level of vehicle aggregation. Deterministic route choice models assume that the trip-maker selects a path that has the minimum cost as defined in the model, i.e., no route with higher than the minimum cost is used. Stochastic route choice models are discrete choice models that specify the probability that a specific route will be chosen. This approach accounts for unobserved factors related to route choice, such as travelers' subjective perceptions about a preferred route. Often this is accomplished through the use of random utility theory, and results in the choice of routes that can have higher than minimum cost in the model.

Static and dynamic route choice models differ in the way they treat time. Static route choice models do not account for time, neither when evaluating route costs nor when predicting route choice. Travel times are derived from the instantaneous network conditions at the time of starting a trip. Dynamic route choice models account for the time dependence of travel times and predict, accordingly, time-dependent route choice. The experienced network conditions depend on when a vehicle has reached a particular point in the network.

The level of vehicle aggregation in route choice models varies between microscopic, macroscopic, and mesoscopic. Microscopic route choice models define discrete route choices by individual vehicle. Macroscopic models distribute vehicle flows across alternative routes.

Typically, this is done deterministically, with flows concentrated on routes of minimum cost. The level of aggregation in mesoscopic models is between that of microscopic and macroscopic, in which vehicles are grouped into packets, for which a single common route is selected.

Network Flow Models

Network flow models define how links and nodes perform under congested conditions. Static network flow models assume that route flows propagate instantaneously through the network and use VDFs to compute travel times. Travel times are considered an explicit function of link flow rather than an implicit result of traffic flow propagation; thus, link flows are not constrained to capacities. In practice, nearly all static flow models are macroscopic and deterministic.

Dynamic flow models are fundamentally different than static flow models in that they move traffic through the network in time slices or distinctive time periods and estimate travel times based on traffic flow theory. Macroscopic flow models use speed-density relationships in the form of a fundamental diagram to model traffic flow. In these models, flow can never exceed capacity, so that queues build up and can spill over between time periods. Simple models assume vertical queues without any physical length, while more complex models reflect horizontal queues. In addition, macroscopic flow models include node models that restrict the flow of traffic to determine the severity and direction of congestion at intersections.

Instead of using fundamental diagrams and macroscopic flow theory, microscopic network flow models consider vehicles separately. These models simulate car-following behavior, gap acceptance, speed adaptation, ramp merging, lane-changing, and overtaking behavior. They require a high level of detail, but are able to represent the behavior of each vehicle.

Mesoscopic flow models are hybrids of macroscopic and microscopic models. They are based on macroscopic traffic flow theory, but propagate individual vehicles or packets of vehicles through the network. The strength of this approach is that it relies on robust macroscopic flow theory, while at the same time retaining information on individual vehicles (e.g., route selection and vehicle class).

3.2 Categories of Assignment Methods

The main criterion for classifying network assignment models is the representation of time. The following categories define the range of assignment models resulting from the combination of different types route choice and network flow models:

- The combination of a static route choice model and a static network flow model results in the typical assignment model used to implement the fourth stage of a traditional four-step model. Nearly all of these models are macroscopic and deterministic.

- The combination of a dynamic route choice model and a dynamic network flow model results in a dynamic traffic assignment (DTA) model. Common subcategories of DTA's consist of combinations of macroscopic route choice/macroscopic network flow models and microscopic route choice/micro or mesoscopic flow models.

- Intermediate approaches exist that simplify the dynamic route choice and dynamic network flow processes by specifying a static assignment for a series of time slices and using simplified dynamic coupling procedures to simulate the carry-over of vehicle queues from one time slice to the next. An example of this approach is the dynamic user equilibrium (DUE) assignment method.

Several of the more frequently found combinations of these model dimensions are:

- Static, macroscopic
- Dynamic, macroscopic
- Dynamic, microscopic or mesoscopic

All of these are equilibrium assignment methods. A second class of methods performs non-equilibrium, stochastic assignment, in which routes are selected probabilistically rather than strictly according to travel time minimization. These methods are not widely used and have several difficulties for use in strategic planning and implementation. Therefore, they were not further investigated as a part of this study.

With all equilibrium assignment methods, demand is distributed according to Wardrop's first principle which states, “Every road user selects his route in such a way that the travel time on all alternative routes is the same, and that switching to a different route would increase personal travel time." The dynamic equilibrium assignment methods extend this idea to include all departure time slices or distinctive time periods considered in the evaluation of the traffic assignment.

The state of equilibrium is achieved through a multi-step iteration process where flows are shifted between alternative set of paths/routes based various mathematical models. Thus, the objective of all equilibrium assignment methods is the same - equilibration of travel times/generalized costs on alternate routes for all origin-destination zone pairs in the network. However, the algorithms applied for this objective may differ.

Static, Macroscopic Assignment

There are three commonly-used static, macroscopic assignment procedures. These are:

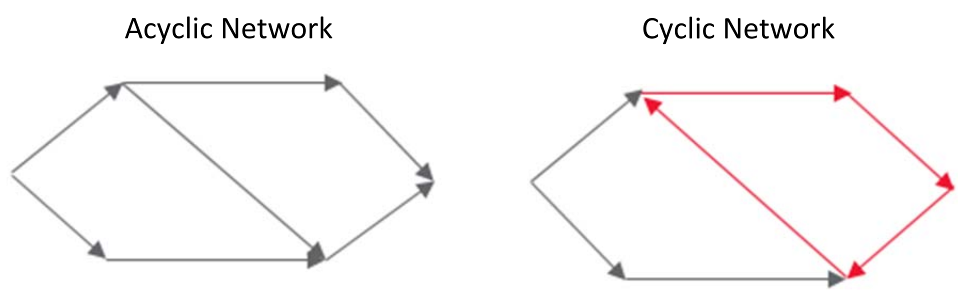
- Link-based
- Path-based
- Bush (origin)-based

With the link-based procedure, an initial all-or-nothing assignment is followed by assignment iterations in which the impedance for several shortest routes between each zone pair are estimated from the impedance associated with the current volume and the impedance from the previous iteration. Route flows are balanced by updating link flows with a method of successive averages. This process continues until the stopping criteria of maximum iterations or convergence is satisfied. This procedure is also referred to as the Frank-Wolfe assignment method.

The path-based procedure starts with an all-or-nothing or incremental assignment, followed by iterative assignments in which shortest paths are searched between each zone pair to augment the existing path set. In the inner loop of each iteration, two or more routes on an O-D pair are brought into a state of equilibrium by shifting vehicles based on the difference in path impedance or generalized cost. The outer loop then checks if there are new routes with lower impedance, based on the current network state. This process continues until the stopping criteria of maximum iterations or convergence measure are satisfied.

In the general framework for the bush-based procedure, flows from one origin are moved using a special structure called a bush. A bush is a subnetwork of the original network rooted at a given origin. It is acyclic, i.e., it does not contain loops or cycles, and has the property of allowing every node to be reached that was reachable in the original network.

Figure 1 – Acyclic and Cyclic Networks



Source: PTV Group, Traffic Assignments in Visum, 2017

Each bush is initialized with a shortest path tree rooted at the origin that is created with an all-or-nothing assignment. In subsequent iterations, the bush is modified by traversing the network and adding efficient links and removing unused (zero flow) links. A link without flow is not removed if the connectivity of the bush would be broken, and a new link is only admitted if it does not create a cycle.

Dynamic, Macroscopic Assignment

Dynamic, macroscopic assignment is fundamentally different from static assignment in the following ways:

- Traffic flows are dynamically propagated through the network in different time slices rather than statically on all parts of the network at the same time.

- Queues and spillovers between time periods are explicitly represented.
- Link delays are modeled based on a fundamental diagram rather than VDFs.
- Intersection delays are calculated using a queue dissipation model.
- Intersection and link capacities are based on physical constraints, with volume-to-capacity ratios never exceeding 1.0.

As with static assignment, this assignment approach is iterative, with the number of iterations and convergence gap used as stopping criteria.

Dynamic, Microscopic/Mesoscopic Assignment

Dynamic, microscopic assignment is a simulation-based procedure that accounts for node impedances and allows modeling of the forming and dissolving of queues over time by simulating time-varying network flows. The supply and demand may be varied over time as well. The network is loaded with demand based on a simulation, which means that individual vehicles are simulated and a simple car following model is applied to have the vehicles follow the paths they are assigned. The assignment is an iterative procedure that includes the following steps: 1) route search; 2) redistribution of path flows; and 3) network loading. These steps are repeated until a relative gap or the maximum number of iterations is reached. Microscopic assignment requires a network with intersection geometries and control data defined. The intersection geometry defines the lanes and turns a vehicle uses during the simulation to follow the route it is assigned. Along with the intersection control data, geometry forms the basis for calculating wait times at the intersection and on its upstream links.

Dynamic, mesoscopic assignment is a variation of the microscopic assignment method, in which vehicle groups, rather than individual vehicles, are moved through the network according to aggregate speed/density relationships.

Examples of software packages containing these types of equilibrium assignment methods are shown below. Most of the packages offer multiple assignment methods.

Table 1 – Equilibrium Assignment Software Packages

Software	Static, Macroscopic	Dynamic, Macroscopic	Dynamic	
			Mesoscopic	Microscopic
Aimsun	√	√	√	
Cube - Avenue			√	
Cube - Voyager	√			
Dynameq				√
DynusT			√	
Emme	√			
TransCAD	√	√		
TransModeler		√		√
Visum	√	√	√	

4.0 Evaluation of Assignment Methods

4.1 Assignment Method Objectives and Evaluation Criteria

The assignment method objectives defined by the study working group describe the desired properties and capabilities of the assignment methods as well as the types of applications the methods are to be used for. Evaluation criteria were defined that would allow the objectives to be reflected in the comparison of the alternative methods. These are listed below.

Table 2 – Assignment Method Objectives and Evaluation Criteria

Assignment Method Objective	Evaluation Criteria
I. Desired Properties and Capabilities	
A. General Properties and Capabilities	
1. Accurate estimation of link traffic flows	a. Accurate estimation of traffic flows by vehicle class, facility type, and V/C level
2. Reasonable representation of travel times/speeds	a. Reasonable representation of travel times/speeds by vehicle class, facility type, and V/C level b. Reasonable representation of zone-to-zone travel times by vehicle class and time period
3. Reasonable model run times	a. Minimization of run time, assuming a representative network[2]
4. Reasonable level of effort for implementation and calibration	a. Minimization of implementation, assuming a representative network b. Minimization of calibration time, assuming a representative network
5. Reasonable input data requirements	a. All data readily available from existing sources b. Minimization of data collection time for initial implementation of method, assuming a representative network
6. Reasonable level of staff skills and staff time required for application and	a. Method can be applied by entry-level modeling staff (1-2 years experience)[3]

[2] Roughly 20,000 links and 1,500 zones.

Assignment Method Objective	Evaluation Criteria
maintenance	b. Minimization of time requirement for typical application, assuming a representative network[4] c. Minimization of annual maintenance time requirement, assuming a representative network
7. Robust outputs	a. Ability of reflect uncertainty b. Reasonable marginal impact of variable values on assignment results
8. Transparency/understandability of method	a. Ability to intuitively understand assignment method processes b. Ability to interpret cause-effect relationships in assignment outputs
9. Flexibility and extendibility of method	a. Ability to adapt method to changing future conditions that may affect travel behavior or transportation system operations
B. Specific Properties and Capabilities	
1. Convergence of network flows	a. Degree of convergence b. Rate of convergence
2. Compatibility with applicable model form	a. Consistency with applicable model and potential to enhance usefulness of model
3. Realistic estimation of intersection delay	a. Accuracy of estimated delays by traffic movement b. Accuracy of estimated delays for all V/C ranges
4. Multiple levels of output resolution	a. Levels of temporal resolution b. Levels of network resolution
5. Representation of new technologies (e.g., shared mobility)	Same as objective
6. Representation of traffic operations characteristics (e.g., intersection spillback, queuing, and lane overflows)	Same as objective
7. Representation of peak spreading	Same as objective

[3] Application includes data collection, network coding, assignment method application, and interpretation and reporting of results.
[4] Typical application would be an assignment to reflect minor network modifications.

Assignment Method Objective	Evaluation Criteria
II. Applications	
A. Scenario Testing	
1. Road capacity improvements	a. Reasonableness of response to capacity improvements b. Ability to reflect effects of small-scale capacity improvements c. Ability to represent effects of capacity improvements on both travel time and traffic operations
2. Road pricing schemes	a. Reasonableness of response to pricing schemes b. Range of pricing schemes that can be represented c. Ability to support representation of pricing effects in travel demand model
3. TSMO strategies	a. Ability to reflect effects of small-scale TSMO strategies b. Ability to represent effects of TSMO strategies on both travel time and traffic operations
B. Planning and Analysis Support	
1. GHG reduction and air quality analysis	a. Accuracy of outputs used in GHG reduction/AQ analysis b. Number of assignment outputs that can be used for GHG reduction/AQ analysis
2. Regional scenario planning	a. B/C of implementing/applying method for regional scenario planning
3. Performance measure calculation	a. Ability to produce performance measure outputs for large, medium, and small-scale improvements
4. Project selection	a. Ability to support project selection
5. Subarea planning	a. Minimization of effort to implement/apply method for subarea planning b. Ability to reflect both capacity and operational effects of improvements for subarea planning c. Ability to reflect effects of large, medium,

Assignment Method Objective	Evaluation Criteria
	and small-scale improvements for subarea planning
6. Policy analysis (e.g., related to TPR)	a. B/C of implementing/applying method for policy analysis b. Range of policies that can be represented
7. B/C analysis of transportation improvements (large, medium, and small-scale)	a. Number and accuracy of outputs that can be used for estimating benefits (travel time savings, traffic operations benefits) b. Number and accuracy of outputs that can be used for cost estimation (i.e., travel delay, facility sizing)

4.2 Assignment Method Evaluation Results

The evaluation criteria were applied to the three assignment methods discussed in the previous section – static/macroscopic, dynamic/macroscopic assignment, and dynamic/microscopic or mesoscopic assignment. The methods were were evaluated by assigning a rating of “low”, “medium”, or “high” for each criterion, indicating the general degree to which the methods satisfied the criteria in absolute or relative terms. The absolute assessment considered how well a method satisfies the criteria on its own merits, while the relative assessment reflected how well the method satisfies the criteria compared to the other methods. The evaluation results are presented in detail in Appendix A. For each criterion, the ratings for the methods are listed, together with notes on the rating rationale and characteristics of the methods.

The evaluation was <u>not</u> intended to rank the methods to identify a “best” method, but rather to to establish a general framework for considering the advantages and disadvantages of the methods. If there is interest in determining which method may be the most appropriate for a particular urban area, travel demand forecasting model, or model application, the methods can be quantitatively ranked by using a combination of weights and numeric scores for the criteria to develop a weighted total score for each method. As documented in Technical Memorandum #3 for this study, this approach was used to evaluate three specific assignment methods for use in the Southern Oregon ABM. Excerpts from Technical Memorandum #3, including the evaluation results, are included in Appendix B.

For purposes of discussing the evaluation, the methods are labeled in the following manner:

- Method 1 – Static, macroscopic assignment
- Method 2 – Dynamic, macroscopic assignment
- Method 3 – Dynamic, microscopic/mesoscopic assignment

A summary of the evaluation is shown in Table 3.

Table 3 – Assignment Method Rating Summary

Rating	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso
Low	28 60.9%	10 21.7%	10 21.7%
Medium	9 19.6%	33 71.7%	15 32.6%
High	9 19.6%	3 6.5%	21 45.7%
Total No. of Criteria	46 (100%)	46 (100%)	46 (100%)

Reading across the diagonal of the table, it can be seen that overall, Method 1 received the highest number of "low" ratings, Method 2 had the highest number of "medium" ratings, and Method 3 received the largest number of "high" ratings. The main reasons that Methods 2 and 3 rated relatively well compared to Method 1 are:

- Better accuracy of link traffic flows;
- Better representation of travel times/speeds;
- More realistic estimation of intersection delay;
- Higher levels of temporal resolution;
- More realistic representation of traffic operations characteristics;
- Ability to represent peak spreading;
- More accurate estimation of the effects of capacity improvements;
- Ability to reflect the effects of small-scale improvements, such as TSMO-type improvements; and

- Better ability to support project selection.

These advantages are primarily related to the more realistic representation of network response to congestion (in terms of delay), the higher level of temporal resolution and, in the case of Method 3, the higher network resolution. The higher overall rating for Method 3 compared Method 2 is also largely accounted for by the advantages of Method 3 in these areas.

There are several criteria, however, for which Methods 2 and 3 were rated lower than Method 1. This is related to the following disadvantages of Methods 2 and 3:

- Larger implementation and maintenance time requirements;
- Larger data collection time requirement;
- Less intuitive understanding of the assignment method processes;
- Greater difficulty in interpreting the cause-effect relationships in the assignment outputs;
- Higher level of staff expertise for application and maintenance; and
- Lower degree of assignment convergence.

All of these disadvantages, except the last one, are due to the greater complexity of Methods 2 and 3. The lower degree of assignment convergence is related to the constrained physical capacities used in the these methods compared to the continuous VDFs used in Method 1, which result in the spillover of traffic to adjacent time periods if capacity is exceeded. Also with Methods 2 and 3, fractional vehicles are used in the assignment which reduces the level of convergence possible compared to Method 1, which uses whole vehicles.

5.0 Assignment-Related Topics

Specific topics related to the implementation and use of the assignment methods were identified for investigation by the study working group. The focus of the investigation was on static assignment topics because this will likely be the main method used by ODOT and its partner agencies within the near-term. A brief discussion of each of the topics is provided below.

5.1 Static, Macroscopic Assignment Topics

Volume Delay Functions

The following VDF-related topics for the static assignment method were identified by the working group:

- Alternative forms of VDFs
- Incorporation of node-based delay in VDFs
- Calibration of VDF parameter values
- Representation of truck volumes in VDFs
- Representation of network capacity

Alternative Forms of VDFs

The most commonly used VDFs are the Bureau of Public Roads (BPR) function, Davidson's delay model, the Akcelik function, and the conical delay model. These functions are presented in Figure 2.

Figure 2 – Alternative VDFs

BPR	$u = \dfrac{u_0}{\left[1.0 + \alpha(x)^{\beta}\right]}$
Conical	$u = \dfrac{u_0}{\left[2 + \sqrt{\beta^2(1-x)^2 + \alpha^2} - \beta(1-x) - \alpha\right]}$ where, $\alpha = \dfrac{\beta - 0.5}{\beta - 1}$ and $\beta > 1$
Modified Davidson	$u = \begin{cases} \dfrac{u_0}{1 + \dfrac{Jx}{(1-x)}}, & \text{for } x \le \mu \quad (i) \\ \dfrac{u_0}{1 + \dfrac{J\mu}{(1-\mu)} + \dfrac{J(x-\mu)}{(1-\mu)^2}}, & \text{for } x > \mu \quad (ii) \end{cases}$
Akcelik	$u = \dfrac{u_0}{\left(1 + 0.25u_0\left[(x-1) + \sqrt{(x-1)^2 + 8\tau\dfrac{x}{u_0 c}}\right]\right)}$

Source: "*Calibration and Evaluation of Link Congestion Functions: Applying Intrinsic Sensitivity of Link Speed as a Practical Consideration to Heterogeneous Facility Types within Urban Network*," Journal of Transportation Technologies, 4, 2014.

The terms in the expressions in Figure 2 are:

u_0	=	free flow speed;
u	=	operating speed;
c	=	capacity;
x	=	v/c ratio; and
$\alpha, \beta, \mu, J, \tau$	=	parameters to be calibrated

The BPR function is one of the oldest and most widely used VDFs in travel demand models in the U.S. because of its simple mathematical form and minimum data requirements. It has a number of limitations, however, including the lack of representation of operating conditions and lack of accounting for facility characteristics, such as signalization on arterials.

The conical function was developed to overcome the drawbacks associated with high β exponent values in the BPR function, which can reduce the rate of convergence by giving undue penalties to overloaded links during the first few iterations of an equilibrium assignment and cause numerical problems, such as overflow conditions and loss of precision. Additionally, for links with volumes that are far below capacity, BPR functions with high β values yield free-flow speeds that do not match those of actual traffic volumes.

Two versions of the Davidson function are shown in Figure 2. The first version, which is the original function, became popular because of its flexibility and ability to handle a wide range of traffic conditions and environments. The parameter J is associated with the land use or area type surrounding the highway link. This function has a serious flaw, however, because it cannot define a travel time if link volumes exceed capacity. Therefore, a second version of this function was developed which allows for link oversaturation.

The Akcelik function is a form of the Davidson function that attempts to encompass intersection delay. It can improve the modeling of link travel speed when a significant portion of the travel time is comprised of intersection delay. This delay is captured by the parameter τ. Lower values of τ are recommended for freeways and coordinated signal systems, while higher values are used for arterial roads without signal coordination. It has been reported that this function also produces better convergence and more realistic speed estimates under congested conditions.

Incorporation of Node-Based Delay in VDFs

Most forms of VDFs currently in use do not directly reflect the intersection delay component of travel time along interrupted flow facilities. Some functions, however, incorporate both a link travel time component and intersection delay component in an attempt to more realistically represent all elements of travel time on interrupted flow roadway segments. The intersection component estimates delay as a function of volume and specific intersection characteristics such as capacity, green time ratio, cycle length, and the presence/absence of signal coordination.

Based on the evaluation criteria presented in the previous section, the main advantages of VDFs that include an intersection delay component compared to those that do not are:

- More accurate estimation of link traffic flows and travel times/speeds.
- More accurate estimation of intersection delay.
- Better representation of the effects of capacity improvements.

The primary disadvantage of this method compared to the link travel time only method is the additional data collection and implementation required to represent intersections in the model network.

Calibration of VDFs

The calibration of VDFs involves the use of observed traffic volume and speed/travel time data to adjust the coefficients and free flow speed and capacity values within the functions to fit the data. An example of this is a study conducted by Florida A&M University - Florida State University to calibrate the four types of VDFs described above – the BPR function, Davidson’s delay model, the Akcelik function, and the conical delay model.

In this study, traffic volume and speed data from the Florida Department of Transportation’s traffic monitoring stations and statewide transportation engineering warehouse were used to calibrate the functions across four facility types (freeway, toll road, HOV/HOT lanes, and arterial) and three area types (urban, residential, and rural).

Free-flow speeds were estimated for uninterrupted flow facilities based on the average speeds of vehicles under low flow conditions of density less than 10 passenger cars per hour per mile per lane (pc/h/ln). For interrupted flow facilities, vehicles were considered to be free-flowing if the headway to the vehicle ahead was eight seconds or more and the headway to the vehicle behind was five seconds or more. Practical capacities were estimated as the 99th percentile flow in pc/h/ln rather than maximum hourly flow to exclude outliers. The estimated free flow speeds and capacities are shown in Table 4.

Table 4 – Estimated Florida Free-Flow Speeds and Capacities

Facility Type	Area Type	Number of Sites	Sample size	Speed Limit (mph)	Mean FFS (mph)	q_{max}(pc/h/ln)	Capacity (pc/h/ln)
Freeway	Urban	3	6810	55	64.671	1891	1686
Freeway	Urban	6	13081	65	66.790	2384	2027
Freeway	Residential	3	12083	55	60.537	1632	1418
Freeway	Residential	4	14115	65	67.783	2108	1887
Freeway	Residential	17	71033	70	71.131	2435	1722
Freeway	Rural	4	14115	65	67.783	2108	1878
Freeway	Rural	17	71033	70	71.131	2435	1742
Toll road	Urban	2	24104	60	64.324	1916	1748
Toll road	Urban	3	35586	65	68.503	2315	1938
Toll road	Residential	2	33872	55	63.324	2235	2074
Toll road	Residential	2	52570	65	71.441	1877	1741
Toll road	Residential	2	36288	70	74.031	2183	2025
Toll road	Rural	2	54210	65	73.720	1802	1772
Toll road	Rural	4	68446	70	75.627	2377	2205
HOV/HOT	Urban	1	18445	65	71.116	1917	1857
HOV/HOT	Residential	2	15367	65	70.451	1823	1702
Arterial	Urban	4	16015	30	34.609	984	846
Arterial	Urban	3	10046	45	52.046	969	825
Arterial	Residential	4	12125	35	41.920	936	884

Source: "*Calibration and Evaluation of Link Congestion Functions: Applying Intrinsic Sensitivity of Link Speed as a Practical Consideration to Heterogeneous Facility Types within Urban Network*," Journal of Transportation Technologies, 4, 2014.

The coefficients for each of the VDFs were statistically calibrated using the traffic volume and speed data. The estimated coefficients are shown in Table 5.

Table 5 – Estimated Florida VDF Coefficients

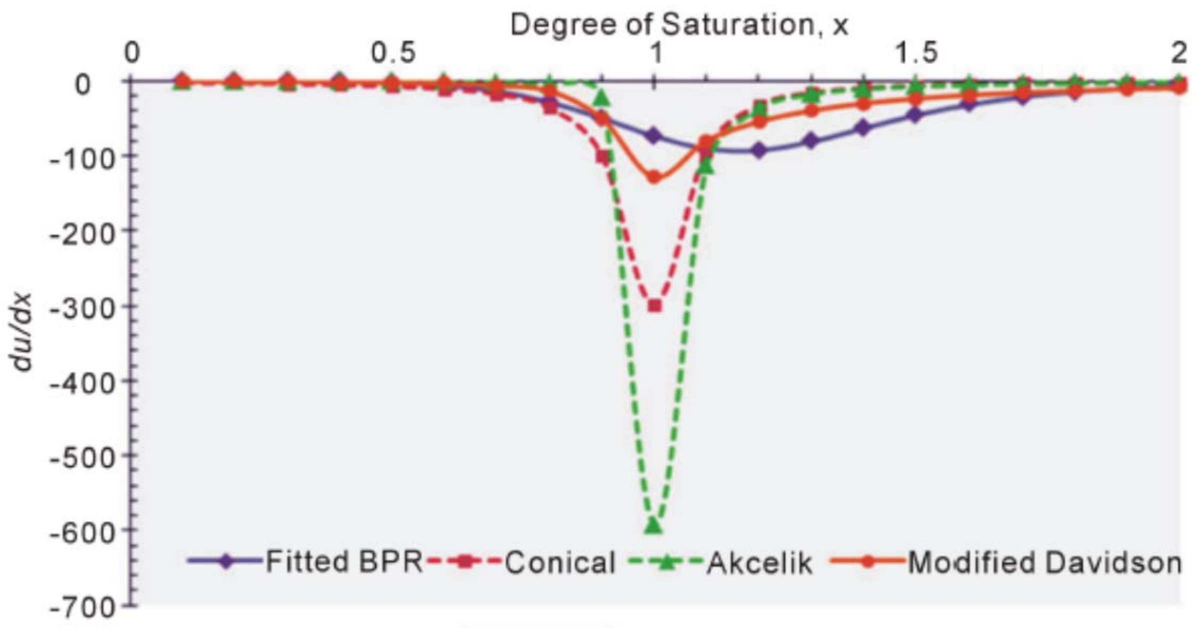
		Facility and Area Type									
		Freeways/Expressways			Toll Roads			HOV/HOT Lanes		Signalized Arterials	
Function	**Parameters**	1	2	3	1	2	3	1	2	1	2
Fitted BPR	α	0.263	0.286	0.15	0.162	0.25	0.32	0.32	0.33	0.24	0.26
	β	6.869	5.091	5.61	6.34	7.9	6.71	8.4	8.6	7.50	8.20
Conical	β	18.390	18.39	15.06	18.39	15.064	15.064	18.55	18.7	18.8	18.8
	α	1.029	1.029	1.04	1.029	1.036	1.036	1.028	1.028	1.03	1.03
Modified Davidson	J	0.009	0.0092	0.0099	0.008	0.0099	0.0099	0.009	0.0089	0.01	0.01
	μ	0.950	0.949	0.951	0.94	0.952	0.940	0.95	0.947	0.95	0.95
Akcelik	τ	0.100	0.101	0.099	0.11	0.098	0.097	0.09	0.08	0.10	0.10

Source: "*Calibration and Evaluation of Link Congestion Functions: Applying Intrinsic Sensitivity of Link Speed as a Practical Consideration to Heterogeneous Facility Types within Urban Network*," Journal of Transportation Technologies, 4, 2014.

Several important findings of the study were:

- VDFs perform differently given different facility types. Therefore, selection of a VDF for a particular facility type and area type needs to be based on an understanding of network performance under different congestion levels and traffic controls.
- The effect of a change in congestion, near or at capacity, will have different impacts on travel speed for a freeway link compared to a signalized arterial link. Speed tends to deteriorate faster on shorter links (urban signalized arterials) than on longer links (uninterrupted flow facilities such as freeways and expressways) when demand is close to capacity.
- The rate of speed change varies by congestion level. When demand is lower than capacity (up to v/c = 0.7), the slopes of the VDFs remain fairly constant, but become steeper as demand approaches capacity. Near capacity, this relationship varies by the type of VDF (see Figure 3).

Figure 3 – Florida Speed Change vs. Congestion Level



Source: "*Calibration and Evaluation of Link Congestion Functions: Applying Intrinsic Sensitivity of Link Speed as a Practical Consideration to Heterogeneous Facility Types within Urban Network*," Journal of Transportation Technologies, 4, 2014.

Representation of Truck Volumes in VDFs

The representation of truck volumes in VDFs can be considered more generally as how to accurately estimate travel speeds for heterogeneous traffic flows. Vehicles exhibit a wide variety of operational and driver characteristics and thus constitute a heterogeneous user population. Based on their physical dimensions, weights, intended uses, and dynamic characteristics, these vehicles can be classified as passenger cars, light trucks, heavy trucks, buses, etc. Trucks, in particular, have very different travel speeds, operational characteristics, sizes, and headways compared to cars. Mixing cars and trucks on the freeway results in larger delays because heterogeneous vehicle types share the same road space. Faster moving cars may experience sight interference and increased lane changing, and trucks slow down the traffic stream because of their limited acceleration and deceleration capabilities. Therefore, the travel time functions of trucks and cars are not identical, and furthermore depend not only on traffic volume, but traffic composition as well.

Commonly-used travel time functions, such as the BPR formula, do not account for heterogeneity in traffic flows. In addition, the technique of converting all vehicle types into a single class using a passenger car unit (PCU) factor does not reflect the operational differences between these types. Thus, there is a need for travel time functions that consider both the volume and proportion of trucks in the traffic stream.

Such functions were developed based on data from a microscopic traffic simulation for a freeway segment in southern California.[5] BPR-type functions were estimated for three vehicle classes: cars, light trucks, and heavy trucks. For the car and light truck functions, a variable reflecting the proportion of the other vehicle classes was included. This variable was not included in the heavy truck function because it was found that the effect of the proportions of the other vehicle classes was not a key determinant of heavy truck travel time. The functions for the car and light truck classes were piecewise, i.e., different functions were applied depending on the proportion of cars in the traffic stream. For conditions where the percentage of cars was above a specific value, the functions containing the proportionality term were applied. In cases where the percentage was below this value, the standard BPR function was applied. This was done because it was found that for traffic streams with a relatively low proportion of cars, the composition of traffic was not as significant variable.

The study revealed that traffic composition plays a significant role in the determination of travel times. Furthermore, the specification of separate functions by vehicle class and the introduction of the proportionality term significantly improved the accuracy of the travel time estimates.

Representation of Network Capacity

The treatment of network capacity varies according to assignment method used. The static, macroscopic assignment method (Method 1 in Section 4.2) is typically used if detailed assignment output is not needed for analysis purposes. An example of this would be the use of

---

[5] Estimating Link Travel Time Functions for Heterogeneous Traffic Flows on Freeways, *Department of Civil and Environmental Engineering, National University of Singapore and Institute of Transportation Studies, University of California, Berkeley,* 2016.

link volumes for estimating the scale of future capacity improvements (number of lanes) for higher-level roadways. For this purpose, network capacity may be represented at the link level only, either in terms of directional capacity or capacity per lane. Typically, different capacities are used by facility type and area type to roughly approximate the varying effects of roadway geometry and traffic operating characteristics on capacity. Intersection capacity at controlled intersections (signals and stop signs) is implicitly represented in the link capacity as a way to include intersection delay in the travel time calculation. If a higher level of accuracy and detail is required in the assignment results, intersection capacity can be reflected separately from link capacity in the type of node-based VDFs described earlier.

Network capacity can be more realistically represented using the dynamic, macroscopic assignment method (Method 2 in Section 4.2). Use of this method would be more appropriate than Method 1 if more accurate, detailed answers to questions about future traffic flows and traffic operations are needed. Some of the advantages of Method 2 are:

- More reasonable response to capacity improvements in highly-congested networks due to a cap on capacity.
- Ability to reflect the effects of smaller-scale capacity improvements (e.g., addition of intersection turn lanes).
- Ability to describe the effects of capacity improvements on both travel time and traffic operations.

The latter two advantages are related to the capability of estimating intersection queuing and queuing delay with Method 2.

The capacities used in Method 2 are more realistic because they are physical capacities rather than abstractions of capacities as in Method 1. Thus, the capacity settings for Method 2 must adhere to the fundamental diagram of traffic flow at the link level and the physical turn/link exit capacities of intersections. This allows the correct calculation and representation of queues in the network. The link capacity is used as a cap for the calculation of link travel time. For link flows less than capacity, travel time is treated as free flow. Once the flow reaches capacity, the travel time is calculated based on a speed-density relationship. The same is true for intersection capacities, where a delay model is used to calculate delays and queues once flows reach capacity.

Dynamic, microscopic/mesoscopic assignment (Method 3 in Section 4.2) provides the most accurate, detailed information about future traffic flows and traffic operations. It is typically used for the analysis of specific corridors or individual locations along corridors rather than system-level analysis due to the large number of detailed inputs required. Because individual vehicle movements are simulated, capacity is an output of the assignment, rather than an input.

Generalized Cost Assignment

Generalized cost assignment attempts to more realistically represent the traveler's path decision-making process by including other factors in addition to travel time. Examples of these factors are tolls, vehicle operating costs, travel time reliability, emissions, and comfort/convenience. Since the generalized cost for a specific link must be expressed as a single value, all of the factors not measured in monetary units must be converted to a constant dollar amount.

An example of this is link travel time, which is converted to dollars using an assumed value of time (VOT). The VOT can be represented in terms of dollars per hour or dollars per minute. In many models, different VOTs are assumed by trip purpose and/or vehicle class. Typically a higher VOT is assigned to the work trip purposes compared to the non-work purposes. Similarly, a higher VOT is assumed for the truck vehicle class than the auto classes. The factors used to convert the non-monetary components of a generalized cost function into dollar amounts can be derived using stated or revealed preference surveys or estimated in the model calibration process.

A simple generalized cost function comprised of travel time, tolls, and operating cost would be expressed as:

*Generalized Cost = Travel Time * VOT + Toll Cost + Vehicle Operating Cost*

Use of Demand Adjustment Procedures

Demand adjustment is a procedure used to update a seed origin-destination matrix using traffic counts. This procedure is most often applied to adjust a base year trip matrix to better fit existing conditions, as reflected in the count data. In the case where the seed matrix is a demand matrix produced by a travel demand forecasting model, the demand matrix can be compared to the adjusted matrix to indicate where potential adjustments may be needed within the model.

Demand adjustment procedures are available within most travel demand modeling software packages, such as Emme and Visum. In Visum, the matrix estimation problem is based on an entropy maximization formulation which incorporates an input count data set and additional constraints that may be selected by the user. In Emme, a gradient method is used which minimizes the differences between link volumes from the model and link counts, while ensuring that the demand matrix is not changed more than necessary.

In applying a demand adjustment procedure, there are some important practical considerations:

1. Based on the general formulation of the O-D estimation problem, it is possible to obtain an infinite number of matrices that will reproduce the counted volumes. As a result, it is recommended that the seed matrix should be from a source that reflects causal travel flow relationships (for example, a travel demand forecasting model), so that the resulting matrix reflects these relationships. Simply using a unit matrix to produce a trip table fitted to observed counts is bad practice and should be avoided.[6]

2. The procedure will adjust the demand matrix to better reflect the observed volumes. It should only be applied if all other data which are used for the assignment have been extensively validated. The procedure will attempt to compensate any remaining error in

[6] Visum Traffic Assignments, PTV, 2017.

the network coding, volume delay functions, or observed volumes by modifying the demand matrix. This, of course, will not correct the error, but just add another error.[7]

3. The selection of count links is very important. The count links should cover the network sufficiently so that most trips will be counted at least once and should not have a large number of local (intrazonal) trips, since these will not be accounted for in the assignment. Care should also be taken when the count links are close to important centroid connectors, since centroids are aggregated abstract trip ends which do not represent true origins and destinations.

4. The matrix adjustment seeks to minimize the error rather than eliminate it. As a result, the procedure should not be used to over-fit the demand matrix to observed counts to obtain a perfect or near-perfect fit. It should be recognized that there are always possible inconsistencies in the counts since they may have been taken on different days or may be non-representative due to traffic incidents, upstream bottlenecks, etc.

5. After the adjustment, the demand matrix and adjusted matrix should be carefully compared in order to identify possible distortions. Typical checks include:

    - Comparing the matrix totals;
    - Plotting before/after trip length frequency distributions;
    - Comparing before/after matrix row and column totals; and
    - Examination of scatter plots to identify matrix cells with large before/after trip differences

    In general, the changes in demand should be small and unsystematic. Large or systematic changes almost always indicate a problem in the input data.

[7] DEMANDJ: A Macro for Demand Matrix Adjustment Using Observed Volumes, Inro, 1990.

5.2 Dynamic, Macroscopic Assignment Topics

Calibration of DTA Models

DTA model results are influenced primarily by the model network, input demand, and the type of queuing model used. Once it has been determined that the input demand is a reasonable representation of trip making in the model area, calibration of the model typically comprises the following steps:

1. In the initial assignment, link capacities only are used, reflecting relatively unconstrained network capacity conditions.
2. Initially, vertical queuing only is applied; i.e., spillback queuing is not modeled.
3. Volume flow plots by v/c level are created for each time interval.
4. The plots are reviewed to determine the reasonableness of traffic flows and congestion points in the network.
5. For locations having unreasonable bottlenecks, the capacities, number of lanes, and speeds are adjusted as needed.
6. Once reasonable link flows are achieved using link capacities only in the model, other capacity constraints are added to the network, such as link exit capacities, followed by turn capacities where needed.
7. The assignment is rerun and volume flow plots are produced.
8. The plots are reviewed for reasonableness and the added capacity constraints are adjusted as needed.
9. Once reasonable link flows are achieved with the added constraints, the spillback queuing model feature is implemented.

Level of Network Disaggregation

DTA model networks are generally more data-intensive than static model networks. For example, although both types of models work on an areawide network, DTA requires more network detail, including the number of lanes on each link, the presence of acceleration–deceleration lanes and turn bays, and lane connectivity.

The network can be based on an existing static model network, GIS files, online maps, or aerial photos. If an existing static model network is used, it must be upgraded to include at least the basic DTA requirements. Such an upgrade can be time-consuming, depending on the spatial extent and density of the network and the level of detail in the static model network. This may also include refining the network topology to better reflect the true alignment of the roadways. Links may be further divided into segments to capture variations in roadway cross-section geometry. Additional work may be involved in defining all allowed and prohibited lane movements at link and segment boundaries.

In contrast to some static models, the geometry and flow characteristics of zone connectors have more significance in DTA models, and should therefore be modeled as real physical roadways. In particular, connectors should not be located close to major intersections as is sometimes the case in static models, and if this is true, they should be moved to mid-block locations or distributed on the link in a manner that corresponds to actual network access/egress locations.

Level of Time Resolution

One of the main advantages of DTA models compared to static models is that the higher level of temporal resolution allows a more realistic representation of network response to congestion, in terms of delay. Theoretically, the higher the level of resolution, the better the calibrated model will represent real-world conditions. Therefore, relatively short time periods are used, typically ranging from five to 15 minutes. Another factor to be considered in defining the time period length is the particular purpose that the model output will be used for. Analyses for which more accurate, detailed estimates of travel times, speeds, and network operating conditions are needed will require higher levels of temporal resolution. This need, however, must be balanced against the computational efficiency of the model. Depending on the spatial extent of the network, the run time for DTA assignments can be significantly longer than that for static models.

Time-dependent trip tables are typically used as the demand inputs to DTA models. Trip patterns can vary across origins, destinations, and departure times. The most common method for capturing these variations is through a series of trip tables, each containing information about the trip departures within a relatively short time interval. The most convenient source of existing trip tables are those produced by travel forecasting models. Most planning agencies have O-D tables for different periods in the day (e.g., a.m. peak, p.m. peak, and off-peak). If hourly factors are available for the time period of interest, these can be used to derive a temporal profile to disaggregate the existing tables into finer time resolutions (e.g., 15-minute tables).

5.3 Dynamic, Microscopic/Mesoscopic Assignment Topics

Calibration of Microsimulation Models

Following the development of a base microsimulation model, calibration is necessary so that it will accurately predict traffic performance. This involves the adjustment of parameters to improve the model's ability to reproduce local driver behavior and traffic performance characteristics. Calibration is performed on various components of the overall model.

Every microsimulation software program comes with a set of user-adjustable parameters for model calibration. The objective of calibration is to find the set of parameter values that best reproduces local traffic conditions.

For convenience, software developers provide suggested default values for the model parameters. It is unlikely, however, that a model will be able to produce accurate results for a local area using only the default values. Therefore, calibration tests should always be performed.

The *Traffic Analysis Toolbox Volume III*[8] recommends dividing the parameters into two basic categories:

- Parameters that the user is certain about and does not wish to adjust; and
- Parameters that the user is less certain about and willing to adjust.

This is done because there are potentially hundreds of model parameters, each of which impacts the simulation results in a manner that is often highly correlated with the others.

The adjustable parameters can be further subdivided into those that directly impact capacity (such as mean headway) and those that directly impact route choice. The capacity parameters are calibrated first, followed by the route choice parameters. The parameters can also be subdivided into those that affect the simulation on a global basis vs. those that have a more localized effect. The global parameters are calibrated first, and then the local link-specific parameters are used to fine-tune the results.

The *Traffic Analysis Toolbox Volume III* also recommends that the model should be calibrated in the following order:

1. Capacity calibration: An initial calibration is performed to identify the values of the capacity parameters that cause the model to best reproduce observed traffic capacities in the field. A global calibration is performed first, followed by link-specific fine tuning.

2. Route choice calibration: If route choice is an option within modeled network, then route choice calibration will be important. In this case, a second calibration process is performed, but this time with the route choice parameters. A global calibration is performed first, followed by link-specific fine-tuning.

3. System performance calibration: Overall model estimates of system performance (travel times and queues) are compared to field-measured travel times and queues.

[8] Traffic Analysis Toolbox Volume III: Guidelines for Applying Microsimulation Modeling Software, Federal Highway Administration, 2004.

Fine-tuning adjustments are then made to enable the model to better match the field measurements.

As an example, the capacity calibration step would be performed as follows:

1. Collect field measurements of capacity, such as queue discharge rates for non-signalized facilities and saturation flow rates for signalized intersections.
2. Obtain model estimates of capacity.
3. Select the calibration parameters, such as mean following headway and driver reaction time for freeways and startup lost time and queue discharge headway for signalized intersections.
4. Set the calibration objective function (e.g., mean square error) for measurement of the difference between observed and modeled capacities.
5. Perform a search for the optimal parameter values that minimize the objective function.
6. Fine-tune the calibration once the optimal global capacity parameter values have been identified.

Size of the Modeling Problem

Among other questions related to the appropriate type of model for a particular type of analysis is the practical scope of the modeling problem. For microsimulation modeling, the answer to this question has significant implications for level of effort required in developing, applying, and maintaining the model. For example, in a study of best practices in microsimulation by the American Association of State Highway and Transportation Officials (AASHTO), it was estimated that mesoscopic simulation models tend to cost an order of magnitude (i.e., ten times) more to develop than macroscopic models.[9] On a similar scale, microscopic simulation models tend to cost an additional order of magnitude more to develop than mesoscopic models on a per-link basis.

[9] Best Practices in the Use of Micro Simulation Models, American Association of State Highway and Transportation Officials (AASHTO), 2010.

This tends to limit the use of microsimulation models to a subregional level. In a national survey of microsimulation practitioners conducted in the same study, it was found that a majority (64%) of the simulation studies were conducted at the corridor or subarea level. Only 27% of the projects were conducted at the regional level. This highlights the difficulties in applying a microscopic simulation tool at a regional level.

Thus, microsimulation is helpful in modeling travel in corridors, but may be less so for regional studies. The size of the network, temporal scale, and travel demand load determine to a large extent the class of simulation models that can produce adequate results. Network size is based on the number of links, nodes, and O-D pairings. The AASHTO best practices study indicates that mesoscopic and DTA models are better equipped to handle large-scale projects, generally those with 15,000 links, 5,000 nodes, 1,000 O-D pairs, and 1,000,000 vehicles. Conversely, microscopic simulation models, to be cost-effective, are typically confined to an area significantly less than regional in size. This is generally on a scale of 50 to 200 nodes and tens of thousands of vehicles, although multi-threading and parallel computing can stretch the simulation model's area of analysis much larger. Table 6 contains information from the study summarizing the applicability of various simulation approaches, including microsimulation, with regard to the geographic and network sizes of the modeling area, length of the analysis time period, and demand level.

Table 6 – General Applicability of Simulation Approaches

Criteria		Applicability		
		Macroscopic Simulation	Mesoscopic Simulation	Microscopic Simulation
Geographic Size	Regional	Yes	Possibly	Not common
	Corridor	Yes	Yes	Possibly
	Subarea	Possibly	Yes	Yes
Network Size*	Large	Yes	Possibly	Not common
	Medium	Yes	Yes	Possibly
	Small	Yes	Yes	Yes
Time Period Length	24 hours	Yes	Possibly	Not common
	Six hours	Yes	Yes	Possibly
	Peak period	Yes	Yes	Yes
	Peak hour	Possibly	Yes	Yes
Demand Level**	Large	Yes	Possibly	Not common
	Medium	Yes	Yes	Possibly
	Small	Yes	Yes	Yes

* Large: > 10,000 links, > 3,000 nodes, > 1,000 zones
Small: < 1,000 links, < 400 nodes, < 100 zones
** Large: >1,000,000 vehicles
Small: < 200,000 vehicles

Source: "*Best Practices in the Use of Micro Simulation Models",* American Association of State Highway and Transportation Officials (AASHTO), 2010.

5.4 General Assignment Topics

Development of Multi-Resolution Modeling Networks

While current macroscopic, mesoscopic, and microscopic modeling approaches have proven their value in analyzing and planning traffic infrastructure and control, they have also shown limitations in their applicability, most of which are inherent in the nature of the models. Microscopic models have proven to be difficult and time consuming to calibrate and difficult to apply because of their richness in parameters and their dependency on large sets of fine grained input data. Macroscopic models are more geared to long-term planning but do not capture the temporal and spatial distribution of traffic during peak hours including daily operational traffic management strategies. Mesoscopic models have shown their ability to accurately model dynamics in traffic demand, but still lack the fidelity to analyze individual vehicles or corridors on a lane by lane basis. Multi-resolution modeling (MRM) is the integration of macroscopic, mesoscopic, and microscopic models for the purpose of analyzing transportation projects at different levels of detail by enabling data to be shared across modeling platforms.[10]

The networks used at each modeling level differ in their scope, level of detail, and types of input data. Typically, the scope or size of the modeling area decreases from the macro level to the micro level, while the level of network detail increases. Corresponding to the greater detail, additional data is needed for the mesoscopic and microscopic model networks. This includes information about roadway geometry and physical characteristics, as well as signal locations, timings, and control.

The objectives in the development of networks at each modeling level are maintaining consistency between the networks, accuracy, and minimization of effort. To maintain consistency and minimize the level of effort, most model integrations are done directly using macroscopic regional travel forecasting models as a starting point. Many of the modeling software platforms, such as TransModeler, have capabilities to facilitate this process. This includes translation of the network structure (links, nodes, and TAZs) into the required format, as well as the transfer basic network attributes, such as the number of lanes, free-flow speed, and capacity.

Even with an automated conversion process, however, the resulting model must be checked for accuracy and built upon to produce the final model. For example, the original network coded in the macroscopic model may have errors and inconsistencies that do not affect that model's results, but could lead to inaccurate results or errors when running more detailed models. Examples of this are ramp locations and lengths, centroid connector locations, and capacities.

[10] Multi-Resolution Model Integration, Center for International Intelligent Transportation Research, Texas Transportation Institute, 2010.

These errors and inconsistencies need to be resolved before using the network as an input to the more detailed models.

In addition, more detailed network attributes and other parameters need to be added when converting a network from a macroscopic model to a mesoscopic or microscopic simulation tool. Real-world sources of this data include data collected using ITS devices, third party vendors, conventional data collection, surveys, and data from agencies responsible for traffic signal control. Further fine-tuning can be done by running the new model after conversion.

Consideration of Travel Time Reliability in Traffic Assignment

Typically in the traffic assignment process, it is assumed that travelers are risk-neutral and only consider average travel time when making route choice decisions. Further, it is assumed that travelers always select the route with minimum travel time between specified origins and destinations, regardless of how variable travel times may be.

However, many empirical studies have shown that travelers also take travel time reliability into consideration when making trip decisions. In fact, under some circumstances, they may place more weight on their knowledge of travel time variation, which is gradually built up based on their past experiences. As a result, the identified optimal paths from traditional models may fail to represent most travelers' risk averse behaviors.

There is no consensus on how reliability should be reflected in deterministic assignment models. One measure is based on the ratio of mean travel time per unit of distance to the standard deviation of mean travel time per unit of distance, with a higher ratio indicating a less reliable trip. A second approach measures reliability as a proportion of success or failure against pre-established thresholds, such as the proportion of trips with a delay less than a predefined threshold.

Another approach was developed in a study in which stated and revealed preference survey data was used to associate travel time reliability with the distribution of travel time. It was assumed that travelers will pay to reduce entropy, which was calculated as a function of the mean and standard deviation of the travel time distribution. The value of entropy was represented in dollars per unit of entropy.

There is greater potential for reflecting reliability in traffic assignment using dynamic rather than static assignment methods. This is because DTA and traffic microsimulation tools explicitly include travel time variability, whereas static assignment can only predict average travel times.

Cost Effectiveness of Dynamic vs. Static Assignment Methods

In determining whether the development of a dynamic traffic assignment model is a worthwhile investment for an urban area or particular project application, the cost and benefits of the model must be considered. As described earlier, one of the largest benefits of dynamic models compared to static models is the more realistic representation of traffic operating conditions, such as travel times, speeds, and intersection queue lengths. Depending on the size of the modeling area, however, the costs of developing and maintaining these models can be substantial, particularly with regard to data preparation, network development, and model calibration.

One of the key factors to be considered in weighing the benefits vs. costs of a dynamic assignment model is the expected level of congestion in the network. This is particularly important if there is interested in studying the effects of congestion-mitigating measures. If congestion can be expected to be low then there is little added value in accounting for it in the model system. This in turn means that detailed representation of congestion in the network assignment is not important. Given that without congestion there is only limited physical connection between the network conditions of different time slices, static network assignment may be fully adequate. If congestion is not expected to be negligible, however, a network assignment method that captures spatio-temporal congestion dynamics may be needed.

An example of the need for a dynamic model is in the examination of the effects of an ITS measure. Since the introduction of this type of measure primarily comes with the intention to provide congestion relief, the network assignment must be able to describe the build-up and dissipation of congestion. Beyond this, the benefits of ITS are strongly dependent on information availability (e.g., who receives the real-time congestion information), technical equipment (who will then follow a recommended path), representation of time (where are travelers at the moment of an incident), and individual traveler characteristics (who is willing and/or capable to at all react to a congestion warning). Apart from time and congestion, a detailed representation of vehicle types, vehicle equipment, and drivers may become necessary. Examples of traffic control measures that fall under the ITS umbrella are traffic responsive signals, dynamic allocation of HOV lanes, and variable speed limits. Vehicle-to-vehicle and vehicle-to-infrastructure communication may also need to be accounted for. These measures require a representation of vehicles and infrastructure at the level of detailed vehicle movements and therefore require a disaggregate representation of network flows.[11]

[11] Evaluation Methods for Calculating Traffic Assignment and Travel Times in Congested Urban Areas with Strategic Transport Models, Institute of Transport Economics, Norwegian Centre for Transport Research, 2014.

6.0 Conclusions

Three alternative assignment methods were evaluated for potential future use in ODOT TPAU's transport model's and those of its OMSC partner agencies – static/macroscopic (Method 1), dynamic/macroscopic (Method 2), and dynamic/microscopic or mesoscopic (Method 3). The methods were evaluated using a set of objectives and evaluation criteria defining the desired properties and capabilities of the assignment methods as well as the types of applications the methods can be used for. The evaluation was not intended to rank the methods to identify a "best" method, but rather to to establish a general framework for considering the advantages and disadvantages of the methods.

Overall, Methods 2 and 3 were rated more highly than Method 1. This is primarily related to the more realistic representation of network response to congestion (in terms of delay), the higher level of temporal resolution and, in the case of Method 3, the higher network resolution. The higher rating for Method 3 compared to Method 2 is also largely accounted for by the greater advantages of Method 3 in these areas. Methods 2 and 3 were rated lower than Method 1 for several of the criteria, however, primarily due to their greater complexity.

Several general criteria recommended in the *Traffic Analysis Toolbox Volume I: Traffic Analysis Tools Primer*[12] can be used to guide decision-making about the most appropriate assignment method to use for a particular application or study. These are:

1. Ability to analyze the appropriate geographic scope or study area for the analysis, such as an isolated intersection, single roadway, corridor, or network.

2. Capability of modeling various facility types, such as freeways, HOV lanes, ramps, arterials, toll plazas, etc.

3. Ability to analyze various vehicle types (e.g. autos vs. trucks).

---

[12] Traffic Analysis Toolbox Volume I: Traffic Analysis Tools Primer, FHWA, 2004.

4. Ability to analyze various traffic management strategies and applications, such as ramp metering, signal coordination, incident management, etc.

5. Ability to estimate traveler responses to traffic management strategies, such as route diversion and departure time choice.

6. Ability to directly produce and output performance measures, such as efficiency (throughput and volumes) and mobility (travel times, speeds, and queue lengths).

7. Cost-effectiveness, mainly from a management or operational perspective, including software cost, level of effort required, ease of use, hardware requirements, data requirements, etc.

Specific topics related to each of the assignment methods were investigated, as well as several general assignment-related topics. Because it is likely that ODOT and its partner agencies will continue to use static assignment in their models over the next several years, several of the static assignment topics that could be further investigated in the near future are:

- Alternative forms of VDFs
- Incorporation of node-based delay in VDFs
- Calibration of parameter values of VDFs
- Representation of truck volumes in VDFs
- Generalized cost assignment

As the need for dynamic assignment grows in the future, several of the dynamic assignment topics that could be further investigated are:

- Calibration of DTA and microsimulation models
- Level of network disaggregation
- Level of time resolution

General assignment topics for additional investigation include:

- Development of multi-resolution networks

- Consideration of travel time reliability in assignment
- Contacting other agencies about the assignment methods they are using

Glossary

Acyclic network	A network that does not contain loops or cycles.
BPR function	A volume-delay function originally developed by the federal Bureau of Public Roads. It is one of the oldest and widely used volume-delay functions.
Bush network	A subnetwork of a larger network rooted at a given origin.
DTA	Dynamic traffic assignment
Fundamental diagram	A diagram that describes the relationship between traffic flow, density, and speed.
Generalized cost	In traffic assignment, the total monetized value of the various factors considered by travelers in route choice.
Mesoscopic model	A hybrid of macroscopic and microscopic models.
Multi-resolution modeling	The integration of macroscopic, mesoscopic, and microscopic models for the purpose of analyzing transportation projects at different levels of detail by enabling data to be shared across modeling platforms.
Network flow model	A component of all traffic assignment models used to define how network links and nodes perform under congested traffic conditions. In static network flow models, the assignment is based on travel times computed using volume-delay functions.
Node-based delay	Delay that occurs at intersections due to traffic control and conflicting traffic volumes
OMSC	Oregon Modeling Steering Committee
Route choice model	A component of all traffic assignment models used to determine the trip-maker’s path selection between origin and destination zones.

Traffic equilibrium	A network state in which travel time on all alternative routes is the same, and route switching would cause an increase in travel time.
Trip end	In traffic assignment, a trip origin or destination. By definition, every trip has two trip ends.
TSMO	Transportation Systems Management and Operations
VDF	Volume delay function. Used in static traffic assignment methods for the estimation of uncongested and congested travel times.
Vertical queue	A traffic queue represented as a queue without any physical length.
VOT	Value of time

Appendix A

Assignment Method Evaluation Results

Table A-1 – Assignment Method Evaluation Results

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	Notes
I. Desired Properties and Capabilities				
A. General Properties and Capabilities				
Objective 1: Accurate estimation of link traffic flows				
Criteria:				
a) Accurate estimation of link traffic flows by vehicle class, facility type, and V/C level	Low	Medium	High	1. In general, the main determinant of assignment accuracy is the accuracy of the demand matrix, not the assignment method used. 2. Accurate estimation of traffic flows is more difficult with congested networks. Method 1 performs more poorly in this case than Methods 2 or 3. 3. While capacity is an input for the Methods 1 and 2, it is an output of Method 3; i.e., maximum flow rates are calculated based on the physical network characteristics and demand. 4. Method 2 does not fully consider the effects of signal timing and opposing traffic flows at intersections, so estimates of link traffic flows are not as accurate as with Method 3.
Objective 2: Reasonable representation of travel times/speeds				
Criteria:				
a) Reasonable representation of link travel times/speeds by vehicle class, facility type, and V/C level	Low	Medium	Medium	1. Method 2 produces better estimates of link travel times/speeds than Method 1 because traffic flows are propagated through the network based on the fundamental diagram and shock wave theory. With these, the effects of downstream congestion can be accounted for. 2. The same is true for Method 3, which uses a car following model to simulate traffic propagation. With this method, physical link features and traffic flow characteristics determine capacity. 3. Method 1 is only a snapshot of traffic flow, with no reflection of dynamic traffic flow.
b) Reasonable representation of zone-to-zone travel times by vehicle class and time period	Low	Medium	Medium	See notes for Criterion a)
Objective 3: Reasonable model run times				
Criteria:				
a) Minimization of run time, assuming a representative network[13]	High	Low	Medium	1. Methods 2 and 3 are generally both slower than Method 1.

[13] Roughly 20,000 links and 1,500 zones.

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	Notes
Objective 4: Reasonable level of effort for implementation and calibration				
Criteria:				
a) Minimization of implementation time, assuming a representative network	Medium	Medium	Low	1. No additional coding time is required for Method 2 compared to Method 1, but coding must be more accurate, so the implementation time is slightly higher. 2. Although link coding with Method 3 is similar to Methods 1 and 2, it requires the coding of geometric details and signal timings for intersections, so the time requirement is higher.
b) Minimization of calibration time, assuming a representative network	Medium	High	High	1. Methods 2 and 3 have slightly lower calibration time requirements compared to Method 1, because there is very little to calibrate. 2. Speed-density relationships can be calibrated with Methods 2 and 3, but this is not typically done because speed-density data for local modeling areas is difficult to obtain. 3. If the network coding is accurate, the speed-density relationships with Methods 2 and 3 generally work well. 4. Method 1 takes longer to calibrate because capacities are abstract approximations, which require the adjustment of VDF parameters. 5. With Method 3, network capacities are an output, not an input, as with Methods 1 and 2. 6. In general, the amount of calibration time varies with level of detail required.
Objective 5: Reasonable input data requirements				
Criteria:				
a) All data readily available from existing sources (e.g., ODOT or local agencies' files, Google Earth, etc.)	High	High	High	1. The required input data for each method are available from existing sources.
b) Minimization of data collection time for initial implementation of method, assuming a representative network	High	Medium	Low	1. Method 2 has a higher time requirement than Method 1 because information on intersection green splits is needed. 2. Method 3 has a higher time requirement than Method 2 because more intersection data is required, such as geometry, phasing plans, cycle lengths, etc.
Objective 6: Reasonable level of staff skills and staff time required for application and maintenance				
Criteria:				

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	Notes
a) Method can be applied by entry-level modeling staff (1-2 years of experience)[14]	High	Low	Medium	1. A higher level of staff experience is required for Method 2, because the user should have some understanding of the fundamental diagram and shock wave theory. 2. Method 3 requires less staff experience than Method 2, because there is not as much theory – it is simply the simulation of individual vehicles on the network.
b) Minimization of time required for typical application, assuming a representative network[15]	High	High	High	1. Method 2 generally takes the longest to run, followed by Method 3. The run time requirement for all methods is relatively quick, however, so this is not a major issue.
c) Minimization of annual maintenance time requirement, assuming a representative network	High	Medium	Low	1. Annual maintenance time differences are related to the same factors as the initial implementation time requirements - see comments for Criterion 5.b).
Objective 7: Robust outputs				
Criteria:				
a) Ability to reflect uncertainty (e.g., natural variability of travel times)	Low	Medium	Medium	1. Method 2 may be slightly more robust than Method 1 if it contains a probit calculation to reflect travel time variance and varying perceptions of the value of travel time. 2. Method 3 is slightly more robust than Method 1 because random seeds can be used to do multiple assignments, with the averaging of results.
b) Reasonable marginal impact of input variable values (e.g., value of time) on assignment results	Low	Medium	Medium	1. Methods 2 and 3 respond more reasonably than Method 1 because V/C ratios cannot exceed 1.0. 2. This assumes that reasonable values are used for wave speed, car following parameters, etc. with Methods 2 and 3.
Objective 8: Transparency/understandability of method				
Criteria:				
a) Ability to intuitively understand assignment method processes	High	Low	Medium	See notes for Criteria 6.a).
b) Ability to interpret cause-effect relationships in assignment outputs	High	Low	Low	1. Interpretation of assignment outputs with Methods 2 and 3 may be more complicated than with Method 1 because of the time dimension and use of "hard" capacities. 2. Methods 2 and 3 are more closely linked with real traffic operations cause-effect relationships, which are more complex than the VDFs used with Method 1.
Objective 9: Flexibility and extendibility of method				
Criteria:				
a) Ability to adapt method to changing future conditions that may	Low	Medium	High	1. Method 3 is more adaptable than Methods 1 or 2 because of the more detailed network

[14] Application includes data collection, network coding, assignment method application, and interpretation and reporting of results.
[15] A typical application would be an assignment to reflect minor network modifications.

Assignment Method Objectives and Evaluation Criteria	Method 1 Static, Macro	Method 2 Dynamic, Macro	Method 3 Dynamic, Micro/Meso	Notes
affect travel behavior or transportation system operations				representation (the higher level of network abstraction with Methods 1 and 2 results in less flexibility). 2. Comparison of the methods using this criterion also depends on the nature of changing future conditions.
B. Specific Properties and Capabilities				
Objective 1: Convergence of network flows				
Criteria:				
a) Degree of convergence	High	Medium	Medium	1. Method 1 achieves the best degree of convergence, because it is based on smooth VDFs. 2. With Methods 2 and 3, capacity is the physical capacity of the roadway, with traffic spilling over to adjacent links and time periods if capacity is exceeded. So the degree of convergence isn't as close. 3. The degree of convergence with Method 3 is also lower because whole vehicles are required, while with Methods 1 and 2, assignments can be done with fractional vehicles.
b) Rate of convergence	Medium	Low	Low	1. The rate of convergence is slowest with Method 2 if MSA is used for volume balancing between iterations. 2. With Method 3, cost proportional balancing can be used if available, which is a faster than MSA.
Objective 2: Compatibility with applicable model form				
Criteria:				
a) Consistency with applicable model and potential to enhance usefulness of model	Low	Low	High	1. For activity-based models, Method 3's dynamic skimming can take advantage time-of-day capabilities. This distinction is only meaningful, however, if there are significant levels of congestion in the network. While current congestion levels may be relatively low in the modeling area, future congestion will likely be higher. 2. For activity-based models, the heterogeneity of traveler characteristics represented in the trip lists activity-based models is lost with the aggregation of demand with Methods 1 and 2. This can be preserved with Method 3 because trip assignment is agent-based.
Objective 3: Realistic estimation of intersection delay				
Criteria:				
a) Accuracy of estimated delays by traffic movement	Low	Medium	High	1. Method 2 produces better estimates of delay than Method 1 because it includes queuing delays. It does not consider opposing traffic volumes, however.

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	**Notes**
				2. Delay estimates are the best with Method 3 because both queuing delays and opposing volumes are reflected.
b) Accuracy of estimated delays for all V/C ranges	Low	Medium	High	1. The accuracy of estimated delays for higher V/C ranges is better with Methods 2 and 3, because there is a cap on capacity. Method 1 does not have cap, so delay estimates for high V/C ratios are not realistic.
Objective 4: Multiple levels of output resolution				
Criteria:				
a) Levels of temporal resolution	Low	Medium	High	1. Method 1 has no temporal resolution within a given time period (e.g., PM peak hour). 2. Varying levels of resolution can be represented with Method 2, depending on the user's preference. 3. Method 3 is an agent-based approach with no fixed time intervals, so it has the highest level of resolution.
b) Levels of network resolution	Low	Low	High	1. Methods 1 and 2 have the same level of network resolution. 2. Method 3 has more intersection detail, reflecting both geometry and signal timing.
Objective 5: Representation of new technologies (e.g., shared mobility)				
Criteria:				
(Criterion same as objective)	Low	Medium	High	See notes for Criterion I.A.9.a).
Objective 6: Representation of traffic operations characteristics (e.g., intersection spillback, queuing, and lane overflows)				
Criteria:				
(Criterion same as objective)	Low	Low	High	1. Method 2 considers spillback, but not finer details, such as signal timing or opposing traffic flows. There is also no accounting of lane-to-lane traffic movements. 2. Method 3 performs complete intersection simulation.
Objective 7: Representation of peak spreading				
Criteria:				
(Criterion same as objective)	Low	Medium	High	1. Method 1 is based on fixed demand per time period and individual link capacities. 2. Method 2 considers the entire network capacity as a constraint, resulting in peak spreading for

Assignment Method Objectives and Evaluation Criteria	Method 1 Static, Macro	Method 2 Dynamic, Macro	Method 3 Dynamic, Micro/Meso	Notes
				high V/C levels. 3. User-coded turning movement capacities are not used with Method 3. Capacities are calculated and treated as outputs rather than inputs.
II. Applications				
A. Scenario Testing				
Objective 1: Road capacity improvements				
Criteria:				
a) Reasonableness of response to capacity improvements	Low	Medium	Medium	1. Method 1 can have an exaggerated response to capacity improvements in highly congested networks due to the lack of a cap on capacity, which results in V/C ratios of greater than one and unrealistically high travel times. 2. Methods 1 and 2 avoid this problem by having a cap on capacity.
b) Ability to reflect effects of small-scale capacity improvements (e.g., addition of intersection turn lanes)	Low	Medium	High	1. Method 2 reflects the effects of intersection improvements better than Method 1 because it includes queuing delay. 2. Method 3 allows the smallest scale improvements to be tested, because the simulation of individual vehicles results in more accurate travel time estimation.
c) Ability to represent effects of capacity improvements on both travel time and traffic operations	Low	Medium	High	1. Method 2 reflects the effects of improvements on intersection queuing; Method 1 does not. 2. Method 3 provides the most complete representation of travel time and traffic operations through consideration of signal timing, offsets, and coordination.
Objective 2: Road pricing schemes				
Criteria:				
a) Reasonableness of response to pricing schemes	Medium	Medium	High	1. Generalized cost can be represented with all of the methods and the value of time is an exogenous input. 2. Method 3 can take advantage of information on individual traveler characteristics produced by ABMs and other agent-based travel models, such as income level and vehicle type, which may affect traveler response to different pricing schemes.
b) Range of pricing schemes that can be represented	Low	Low	Low	1. Method 1 can support/generate input toll matrices (to travel demand models), while Methods 2 and 3 cannot. 2. Time-varying tolls can be represented with Method 2, but the tolls are fixed - i.e., toll levels do not respond to the level of delay, as with managed lanes.
c) Ability to support representation of pricing effects in travel demand model	Low	Medium	Medium	1. Method 2 can include a departure time choice model, which allows time-varying tolls to be input to the peak spreading component of the travel demand model.

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	**Notes**
				2. Method 3 can feed dynamic skims back into travel demand models, unlike Method 2. Therefore, Method 3 is more flexible for time-of-day modeling, which is a component of many ABMs.
Objective 3: TSMO strategies				
Criteria:				
a) Ability to reflect effects of small-scale TSMO strategies	Low	Medium	High	See comments for Criteria 1.b)
b) Ability to represent effects of TSMO strategies on both travel time and traffic operations	Low	Medium	High	See comments for Criteria 1.c)
B. Planning and Analysis Support				
Objective 1: GHG reduction and air quality analysis				
Criteria:				
a) Accuracy of outputs used in GHG reduction/AQ analysis	Low	Medium	Medium	1. Speed is the primary assignment model output used in GHG reduction/air quality analysis. 2. Method 1 doesn't produce real speed estimates, but shadow speeds, because they are not based on real capacities. 3. Methods 2 and 3 provide better speed estimates than Method 1, but neither method accounts for acceleration or deceleration.
b) Number of assignment outputs that can be used for GHG reduction/AQ analysis	Medium	Medium	Medium	Speeds are the primary output available from all methods.
Objective 2: Regional scenario planning				
Criteria:				
a) Benefit/cost of implementing/applying assignment method for regional scenario planning	Medium	Medium	Low	1. The main contribution of assignment models for regional scenario planning is peak spreading modeling. 2. Peak spreading modeling using Method 2 can be done with the same network coding as for Method 1. 3. Method 3 requires more coding time if intersections are involved, so it is probably not worth the effort for regional scenario planning.
Objective 3: Performance measure calculation				
Criteria:				

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	**Notes**
a) Ability to produce performance measure outputs for large, medium, and small-scale improvements	Low	Low	Medium	1. The range of improvements is slightly larger for Method 2 than Method 1. 2. Improvements such as signal coordination can be better tested with Method 3.
Objective 4: Objective: Project selection				
Criteria:				
a) Ability to support project selection	Low	Medium	High	1. A more complete representation of project impacts is possible with Methods 2 and 3 compared to Method 1, allowing for better project evaluation. An example of this is reflecting the effects of intersection improvements on upstream locations, which cannot be done with Method 1. 2. More accurate speed/travel time and queuing estimates are possible with Methods 2 and 3 than Method 1.
Objective 5: Subarea planning				
Criteria:				
a) Minimization of effort to implement/apply method for subarea planning	Medium	Medium	Low	
b) Ability to reflect both capacity and operational effects of improvements for subarea planning	Low	Medium	High	
c) Ability to reflect effects of large, medium, and small-scale improvements for subarea planning	Low	Medium	High	
Objective 6: Policy analysis (e.g., related to TPR)				
Criteria:				
a) Benefit/cost of implementing/applying assignment method for policy analysis	Medium	Medium	Low	
b) Range of policies that can be represented	Medium	Medium	Medium	
Objective 7: Benefit/cost analysis of transportation improvements (large, medium, and small-scale)				
Criteria:				
a) Number and accuracy of outputs that can be used for	Low	Medium	Medium	1. The travel time savings (benefits) of improvements related to queuing can be reflected in

Assignment Method Objectives and Evaluation Criteria	**Method 1** Static, Macro	**Method 2** Dynamic, Macro	**Method 3** Dynamic, Micro/Meso	**Notes**
estimating benefits (travel time savings, traffic operations benefits)				Methods 2 and 3, but not Method 1.
b) Number and accuracy of outputs that can be used for cost estimation (i.e., travel delay, facility sizing)	Low	Medium	Medium	1. Facility sizing requires the use of "hard" capacities in assignment, such as with Methods 2 and 3, to reflect queuing storage requirements.

Appendix B

Evaluation of Assignment Methods for Southern Oregon ABM

Three assignment methods presented by PTV at training sessions held on December 6-7, 2016 were selected for evaluation by TPAU using a set of criteria corresponding to the assignment method objectives developed by the study working group. The evaluation was done to identify an assignment method to use in TPAU's Southern Oregon Activity-Based Model (SOABM).

The Visum assignment methods presented by PTV fall under two main categories – equilibrium and non-equilibrium. Three types of equilibrium methods were presented in the training session. These are:

- Static assignment
- Macroscopic dynamic assignment
- Simulation-based dynamic assignment

Visum contains three static assignment procedures – link-based Loshe, path-based, and bush (origin)-based LUCE. For the evaluation, the bush-based LUCE procedure was selected for evaluation because it has a faster running time than the other static methods. A variation of this method was included, which estimates delay for both links and nodes, rather than links only. To represent the complete range of methods in the evaluation, the macroscopic dynamic (DUE) method and simulation-based dynamic (SBA) method were also selected.

The same objectives and criteria described in Section 4.1 were used for the evaluation. To rank the alternative methods, a methodology was applied in which raw scores were assigned to each method for each of the criteria. The scores reflect the degree of positive or negative difference between the alternative method and the "base" method, which is the static link-based assignment method with BPR VDFs currently used by TPAU. The differences are expressed numerically on a scale of -10 to +10, with a score of zero representing no difference and scores of -10 or +10 representing the maximum degree of negative or positive difference.

Two sets of weights were applied to the raw scores to calculate weighted total scores for each alternative. Weights for the objectives were developed to establish the relative importance of each objective. Criterion weights were developed to reflect the importance of one criterion vs. another in cases where there was more than one criterion per objective. The sum of the objective weights was 100. The sum of the criterion weights was 10 for each objective. A weighted score for each criterion was calculated as:

Weighted Criterion Score = Raw Criterion Score*Objective Weight*(Criterion Weight/10)

A total weighted score for each alternative was then calculated as the sum of the weighted criterion scores.

A summary of the evaluation results are shown Table B-1 below. Detailed results are shown in Table B-2.

Table B-1
Summary of Assignment Methods Evaluation for Southern Oregon ABM

Method	Total Raw Score		Total Weighted Score	
	Score	Rank	Score	Rank
SBA	76	1	183	1
DUE	53	2	129	2
Static - Luce w/ node and link-based delay	21	3	48	3
Static – Luce w/ link-based delay only	0	4	0	4

Table B-2 – Results of Assignment Method Evaluation for Southern Oregon ABM

Evaluation Criteria	Raw Scores (Relative to BPR method)				Weights		Weighted Scores			
	LUCE	Mid-block & node	DUE	SBA	Objective Weights	Criterion Weights	LUCE	Mid-block & node	DUE	SBA
Answer	**0**	**21**	**53**	**76**	**100**		**0**	**48**	**129**	**183**
I. Desired Properties and Capabilities										
A. General Properties and Capabilities										
1. Objective: Accurate estimation of link traffic flows					4.13436693					
Criteria:										
a) Accurate estimation of link traffic flows by vehicle class, facility type, and V/C level	0	1	2	4		10	0	4	8	17
2. Objective: Reasonable representation of travel times/speeds					6.45994832					
Criteria:										
a) Reasonable representation of link travel times/speeds by vehicle class, facility type, and V/C level	0	2	4	4		8	0	10	21	21
b) Reasonable representation of zone-to-zone travel times by vehicle class and time period	0	2	4	4		2	0	3	5	5
3. Objective: Reasonable model run times					1.80878553					
Criteria:										
a) Run time for SOABM network[1]	0	-1	-5	-3		10	0	-2	-9	-5
4. Objective: Reasonable level of effort for implementation and calibration					5.29715762					
Criteria:										
a) Implementation time for SOABM network	0	-2	-4	-6		5	0	-5	-11	-16
b) Calibration time for SOABM network	0	1	1	1		5	0	3	3	3
5. Objective: Reasonable input data requirements					5.94315245					
Criteria:										
a) All data readily available from existing sources (e.g., ODOT or local jurisdictions' files, Google Earth, etc.)	0	0	0	0		0	0	0	0	0

Evaluation Criteria	Raw Scores (Relative to BPR method)				Weights		Weighted Scores			
	LUCE	Mid-block & node	DUE	SBA	Objective Weights	Criterion Weights	LUCE	Mid-block & node	DUE	SBA
b) Data collection time for initial implementation of method for SOABM network	0	-2	-5	-8		10	0	-12	-30	-48
6. Objective: Reasonable level of staff skills and staff time required for application and maintenance					4.65116279					
Criteria:										
a) Method can be applied by entry-level modeling staff (1-2 years of experience)[2]	0	-1	-5	-3		6	0	-3	-14	-8
b) Time requirement for typical application of SOABM[3]	0	0	-1	-1		3	0	0	-1	-1
c) Annual maintenance time requirement for SOABM network	0	-1	-2	-5		1	0	0	-1	-2
7. Objective: Robust outputs					3.61757106					
Criteria:										
a) Ability to reflect uncertainty (e.g., natural variability of travel times)	0	0	1	1		4	0	0	1	1
b) Reasonable marginal impact of input variable values (e.g., value of time) on assignment results	0	0	2	2		6	0	0	4	4
8. Objective: Transparency/understandability of method					2.97157623					
Criteria:										
a) Ability to intuitively understand assignment method processes	0	0	-5	-3		5	0	0	-7	-4
b) Ability to interpret cause-effect relationships in assignment outputs	0	0	-3	-4		5	0	0	-4	-6
9. Objective: Flexibility and extendibility of method					2.3255814					
Criteria:										
a) Ability to adapt method to changing future conditions that may affect travel behavior or transportation system operations	0	5	0	3		10	0	12	0	7
B. Specific Properties and Capabilities										
1. Objective: Convergence of network flows					5.29715762					
Criteria:										
a) Degree of convergence	0	0	-3	-5		10	0	0	-16	-26
b) Rate of convergence	0	0	-2	-1		0	0	0	0	0
2. Objective: Compatibility with SOABM model form					0.7751938					

Evaluation Criteria	Raw Scores (Relative to BPR method)				Weights		Weighted Scores			
	LUCE	Mid-block & node	DUE	SBA	Objective Weights	Criterion Weights	LUCE	Mid-block & node	DUE	SBA
Criteria:										
a) Consistency with SOABM and potential to enhance usefulness of model	0	0	0	0		10	0	0	0	0
3. Objective: Realistic estimation of intersection delay					4.65116279					
Criteria:										
a) Accuracy of estimated delays by traffic movement	0	2	3	6		5	0	5	7	14
b) Accuracy of estimated delays for all V/C ranges	0	2	3	6		5	0	5	7	14
4. Objective: Multiple levels of output resolution					5.94315245					
Criteria:										
a) Levels of temporal resolution	0	0	4	7		7	0	0	17	29
b) Levels of network resolution	0	0	0	2		3	0	0	0	4
5. Objective: Representation of new technologies (e.g., shared mobility) – see Objective I.A.9	N/A	N/A	N/A	N/A						
6. Objective: Representation of traffic operations characteristics (e.g., intersection spillback, queuing, and lane overflows)					3.48837209					
Criteria:										
(Criterion same as objective)	0	1	2	7		10	0	3	7	24
7. Objective: Representation of peak spreading					4.65116279					
Criteria:										
(Criterion same as objective)	0	0	7	9		10	0	0	33	42
II. Applications										
A. Testing of:										
1. Objective: Road capacity improvements					6.45994832					
Criteria:										

Evaluation Criteria	Raw Scores (Relative to BPR method)				Weights		Weighted Scores			
	LUCE	Mid-block & node	DUE	SBA	Objective Weights	Criterion Weights	LUCE	Mid-block & node	DUE	SBA
a) Reasonableness of response to capacity improvements	0	2	5	6		4	0	5	13	16
b) Ability to reflect effects of small-scale capacity improvements (e.g., addition of intersection turn lanes)	0	0	5	6		3	0	0	10	12
c) Ability to represent effects of capacity improvements on both travel time and traffic operations	0	1	5	7		3	0	2	10	14
2. Objective: Road pricing schemes					1.80878553					
Criteria:										
a) Reasonableness of response to pricing schemes	0	0	0	0		4	0	0	0	0
b) Range of pricing schemes that can be represented	0	0	-2	-2		2	0	0	-1	-1
c) Ability to support representation of pricing effects in travel demand model	0	0	5	3		4	0	0	4	2
3. Objective: TSMO strategies					4.78036176					
Criteria:										
a) Ability to reflect effects of small-scale TSMO strategies	0	0	5	6		5	0	0	12	14
b) Ability to represent effects of TSMO strategies on both travel time and traffic operations	0	1	5	7		5	0	2	12	17
B. Support for:										
1. Objective: GHG reduction and air quality analysis					1.80878553					
Criteria:										
a) Accuracy of outputs used in GHG reduction/AQ analysis	0	2	2	3		6	0	2	2	3
b) Number of assignment outputs that can be used for GHG reduction/AQ analysis	0	0	0	0		4	0	0	0	0
2. Objective: Regional scenario planning					3.61757106					
Criteria:										
a) B/C of implementing/applying method for regional scenario planning	0	1	2	-2		10	0	4	7	-7
3. Objective: Performance measure calculation					4.26356589					
Criteria:										

Evaluation Criteria	Raw Scores (Relative to BPR method)				Weights		Weighted Scores			
	LUCE	Mid-block & node	DUE	SBA	Objective Weights	Criterion Weights	LUCE	Mid-block & node	DUE	SBA
a) Ability to produce performance measure outputs for large, medium, and small-scale improvements	0	0	1	3		10	0	0	4	13
4. Objective: Project selection					2.84237726					
Criteria:										
a) Ability to support project selection	0	1	3	4		10	0	3	9	11
5. Objective: Subarea planning					6.45994832					
Criteria:										
a) Level of effort to implement/apply method for subarea planning	0	0	0	-3		5	0	0	0	-10
b) Ability to reflect both capacity and operational effects of improvements for subarea planning	0	1	5	6		2	0	1	6	8
c) Ability to reflect effects of large, medium, and small-scale improvements for subarea planning	0	1	5	7		3	0	2	10	14
6. Objective: Policy analysis (e.g., related to TPR)					4.13436693					
Criteria:										
a) B/C of implementing/applying method for policy analysis	0	1	1	0		7	0	3	3	0
b) Range of policies that can be represented	0	0	2	2		3	0	0	2	2
7. Objective: B/C analysis of transportation improvements (large, medium, and small-scale)					1.80878553					
Criteria:										
a) Number and accuracy of outputs that can be used for estimating benefits (travel time savings, traffic operations benefits)	0	0	3	3		5	0	0	3	3
b) Number and accuracy of outputs that can be used for cost estimation (i.e., travel delay, facility sizing)	0	1	3	3		5	0	1	3	3
	0	**21**	**53**	**76**	**100**		**0**	**48**	**129**	**183**

[1] ~20,000 links, 1,500 zones.

[2] Application includes data collection, network coding, assignment method application, and interpretation and reporting of results.

[3] Typical application would be an assignment to reflect minor network modifications.

NETWORK CAPACITY CALCULATION FOR AREA TYPE

June 1997

Prepared for:

Oregon Department of Transportation

Prepared by:

Kimley-Horn and Associates, Inc.

TABLE OF CONTENTS

INTRODUCTION

The Oregon Department of Transportation (ODOT) and Metropolitan Planning Organizations (MPO's) are cooperating in a program to improve travel demand modeling methods. One component of this program is the development of a standard methodology for estimating network capacity. Parsons Brinkerhoff Quade & Douglas, Inc. has developed a methodology for ODOT and the MPO's[1]. ODOT is planning to implement this methodology in the travel demand model for the Salem-Keizer urbanized area and in the congestion management system. Before the implementation can be completed, analysis is needed to calibrate the methodology based on "area type". The following is an excerpt from the methodology describing area type and its significance:

> *Area type is an important piece of information which may further characterize the links and affect the value specified for link capacity. Area type may be thought of as an additional dimension by which link capacity may be identified. The most appropriate way to introduce area type is by having a separate service flow rate for each area type. A service flow rate would be "looked up" based on area type of link. Similarly, area type may be used to stratify the green to cycle length values if these values differ by area*
>
> *Area type is one objective way of determining whether links are located in dense activity centers or in remote areas. By stratifying the service flow rates by area type, effects of pedestrian interaction, transit vehicle interaction, intersection spill-back, etc. can be accounted for on links in dense areas. Likewise, effects of significant interaction with vehicles leaving and entering driveways and parking lots in suburban area can be accounted for.*

The purpose of this analysis is to perform field measurement of service flow rates on urban arterial roadways in a variety of area types and correlate these values with area type data. The result will be area type adjustment factors that can be applied to the calculated service flow rates in order to calibrate network capacity. The hypothesis is that area type can be defined primarily by employment and population density. Other area specific conditions such as driveway density, pedestrian activity, and transit activity are also considered.

[1] Parsons Brinkerhoff Quade & Douglas, Inc. "Highway Network Capacity Specification, Draft Methodology". January, 1995.

FIELD DATA COLLECTION

Data Collection Methodology

Data required to calculate actual saturation flow was collected at six signalized arterial roadway intersections in the Salem-Keizer metropolitan area. At each location, two lanes on one of the arterial approaches were examined (an exclusive through lane and a shared through/turn lane). The six survey locations were chosen in order to obtain saturation flows in a variety of area types. Locations included both one-way and two-way arterial streets. Data was collected during both the AM and PM peak periods. Other data recorded at each location included signal timing, transit activity, adjacent driveway spacing, and the number of pedestrians crossing the receiving leg of turn movements which conflict with traffic on the subject approach. The locations included in this analysis are summarized in **Table 1**.

Table 1. Summary of Data Collection Locations

Number	Intersection	Roadway Type	Approach	Lanes	Date	Time Period
1	Commercial St. at Kuebler Rd.	Two-Way	Southbound	Exclusive Through, Shared Through/Right	Tuesday, 4/15/97	PM Peak
2	Commercial St. at Madrona Ave.	Two-Way	Northbound	Exclusive Through, Shared Through/Right	Wednesday, 4/16/97	AM Peak
3	Commercial St. at Owens St.	One-Way	Southbound	Exclusive Through, Shared Through/Left	Wednesday, 4/16/97	PM Peak
4	Commercial St. at Hoyt St.	Two-Way	Northbound	Exclusive Through, Shared Through/Right	Thursday, 4/17/97	AM Peak
5	Ferry St. at Liberty St.	One-Way	Westbound	Exclusive Through, Shared Through/Right	Thursday, 4/17/97	PM Peak
6	Mission St. at 25th St.	Two-Way	Eastbound	Exclusive Through, Shared Through/Right	Wednesday, 4/23/97	PM Peak

Saturation flow is defined as the maximum discharge rate during the green time. For operational calculations, saturation flows have units of passenger cars per hour of green time per lane (pcphgpl). When measured in the field, saturation flows have units of vehicles per hour of green time per lane (vphgpl). Each time a movement is started, a "start-up lost time" is experienced. Saturation flow is usually achieved after the fourth vehicle enters the intersection from a standing queue.

The data required to calculate saturation flow was collected based on the procedure described in the Highway Capacity Manual (HCM)[2]. The data collection procedure is described below: Using a stop watch, time was started at the beginning of green for the subject movement. Time was recorded when the rear axle of the fourth vehicle crossed the stop bar (t_4) and when the last vehicle in the queue at the beginning of green crossed the stop bar (t_n). The total number of vehicles stopped in the queue at the beginning of green (n) was recorded.

[2] Transportation Research Board. Highway Capacity Manual, Special Report 209. October 1994. Appendix 9-IV.

As described in the HCM, to obtain a statistically significant value for saturation flow, it is necessary to record data for a minimum of 15 cycles with more than 8 vehicles in the initial queue. Data for cycles that did not have an initial queue of 8 or more vehicles was not included in the calculation of prevailing saturation flows. Field data sheets for each location are included in the **Appendix** of this report.

SATURATION FLOW RESULTS

Saturation Flow Calculation

Saturation flows were calculated for each approach at each study intersection using the methodology below.

The saturation flow is the inverse of the average time headway per vehicle. Adjusting for units, this relationship is shown below in **equation (1)**.

$$s = \frac{3600}{h} \tag{1}$$

where

s = saturation flow (vphgpl)
h = average time headway (sec/veh)

The average time headway per vehicle is obtained directly from the data collected. **Equation (2)** yields the average time headway for one cycle.

$$h = \frac{(t_n - t_4)}{(n - 4)} \tag{2}$$

where

t_n = time when the rear axle of the last vehicle queued at the beginning of green crosses the stop bar (sec)
t_4 = time when the rear axle of the 4th vehicle queued at the beginning of green crosses the stop bar (sec)
n = number of vehicles queued at the beginning of green

Lost Time Calculation

After a signal turns green, the first several vehicles in the queue experience start-up losses that result in their movement at less than the saturation flow rate. It was assumed that saturation flow occurred after the fourth vehicle.

Summary of Saturation Flow Results

Resulting saturation flows, green time to cycle length ratios (g/C), number of cycles observed, and headways for each location are summarized in **Table 2**.

Table 2. Summary of Saturation Flow Results

Location	Lane Type	g/C	Number of Cycles	Average Time Headway (sec/veh)	Saturation Flow (vphgpl)
1. Southbound Commercial Street at Kuebler Road					
	Exclusive Through	0.36	20	1.73	2077
	Shared Through/Right Turn	0.36	16	1.93	1861
2. Northbound Commercial Street at Madrona Avenue					
	Exclusive Through	0.52	19	1.95	1843
	Shared Through/Right Turn	0.52	19	1.82	1981
3. Southbound Commercial Street at Owens Street					
	Exclusive Through	0.68	15	2.00	1802
	Shared Through/Left Turn	0.68	14	1.98	1817
4. Northbound Commercial Street at Hoyt Street					
	Exclusive Through	0.76	18	1.71	2103
	Shared Through/Right Turn	0.76	17	1.89	1900
5. Westbound Ferry Street at Liberty Street					
	Exclusive Through	0.45	18	2.09	1721
	Shared Through/Right Turn	0.45	22	2.21	1629
6. Eastbound Mission Street and 25th Street					
	Exclusive Through	0.46	16	1.80	1998
	Shared Through/Right Turn	0.46	17	1.97	1826

Study results show saturation flows ranging from 1629 vphgpl to 1981 vphgpl for shared through/turn lanes with an average saturation flow rate of 1835 vphgpl. The saturation flow rates for exclusive through lanes ranged from 1721 vphgpl to 2103 vphgpl with an average rate of 1924 vphgpl.

AREA TYPE AND SATURATION FLOW RELATIONSHIP

Observed saturation flow rates were compared to area type characteristics to determine any correlation. Area type in this analysis was assumed to be defined primarily by employment and population density.

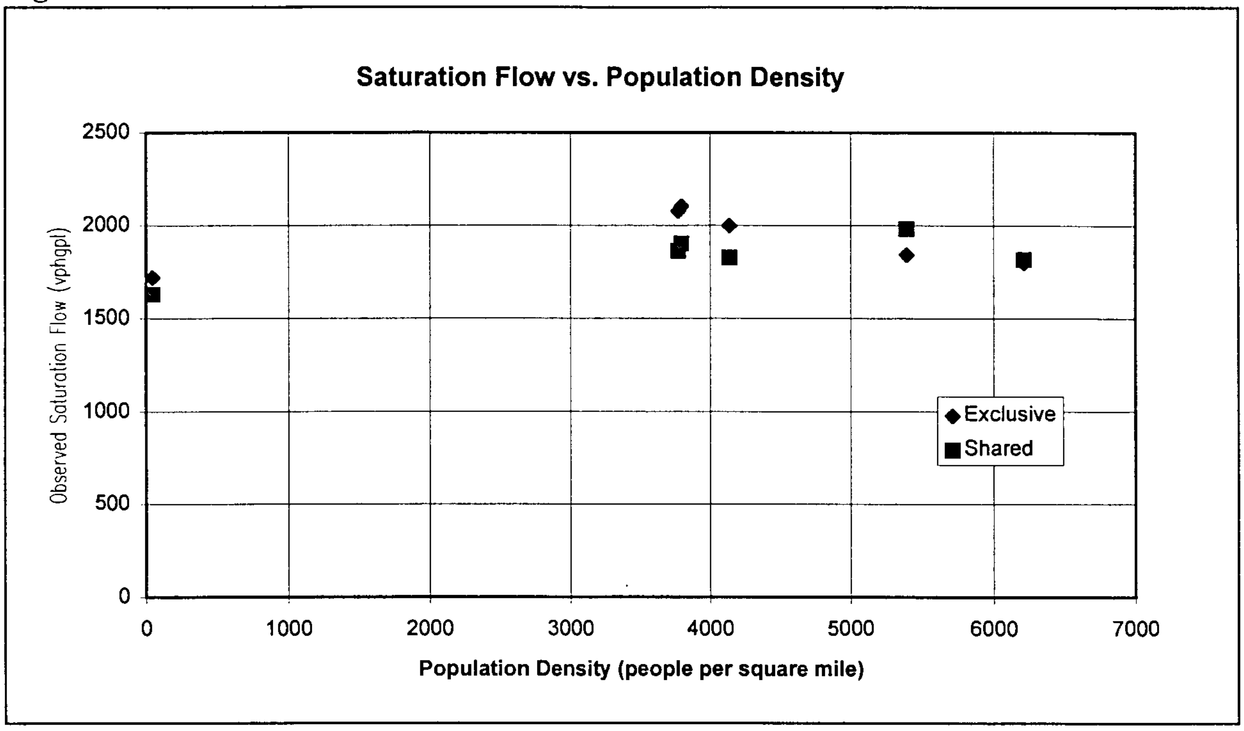
Population and Employment Density

The observed saturation flow results were compared to population and employment density to determine if a correlation exists. Density for a specific location was determined by calculating the weighted average of population, employment, and combined population and employment densities based on the area of adjacent Transportation Analysis Zones (TAZ's). **Table 4** shows the average densities for each intersection. The average densities were plotted versus saturation flow rates in **Figures 1-3**. The densities for the TAZ's adjacent to each intersection, as well as the average density calculations are included in the **Appendix** of this report.

Table 4. Summary of Population, Employment, Combined Densities

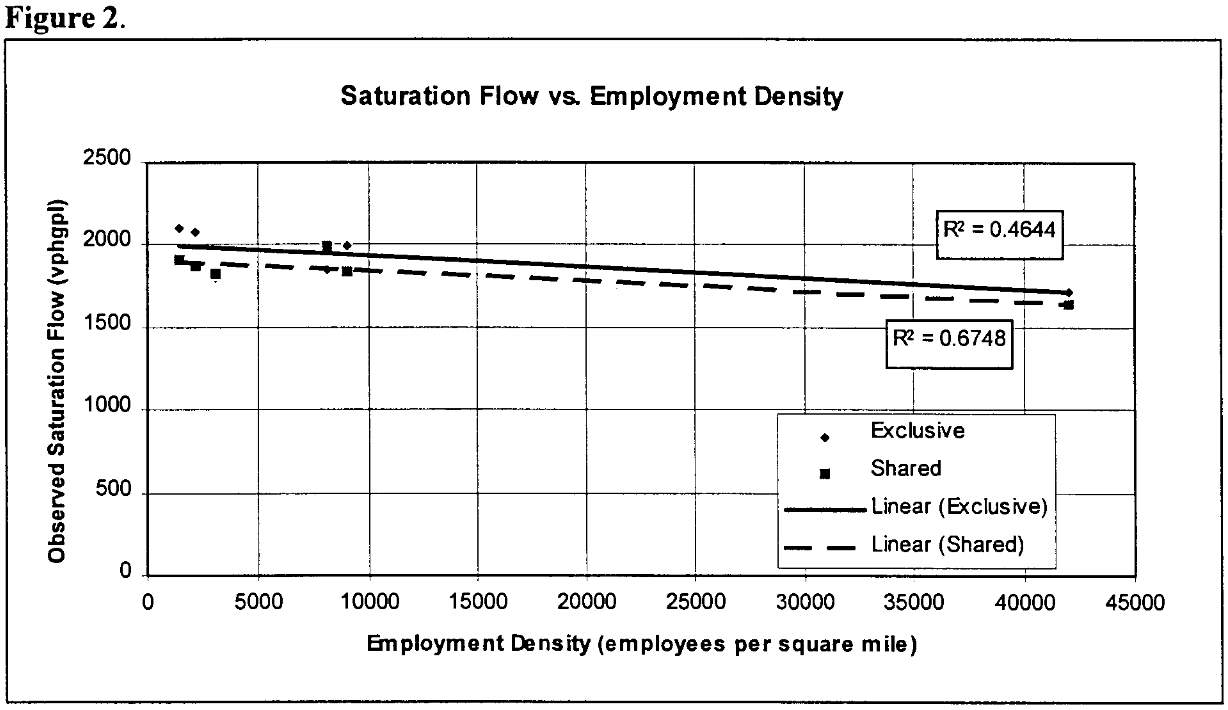
Location	Population Density (people per sq. mi.)	Employment Density (employees per sq. mi.)	Combined Pop. + Emp. Density
1. Southbound Commercial St. at Kuebler Rd.	3774	2171	5945
2. Northbound Commercial St. at Madrona Ave.	5393	8162	13555
3. Southbound Commercial St. at Owens St.	6214	3120	9334
4. Northbound Commercial St. at Hoyt St.	3797	1422	5218
5. Westbound Ferry St. at Liberty St.	45	42033	42078
6. Eastbound Mission St. and 25th St.	4853	4138	8991

Figure 1.



No clear relationship was observed between population density and saturation flow, as illustrated in **Figure 1.**

Figure 2.



As shown in **Figure 2**, saturation flows generally reduced with increased employment density. In addition, saturation flow in shared lanes was in most cases lower than saturation flow in exclusive through lanes. This was expected as traffic in shared lanes is often slowed by turning vehicles.

Figure 3.

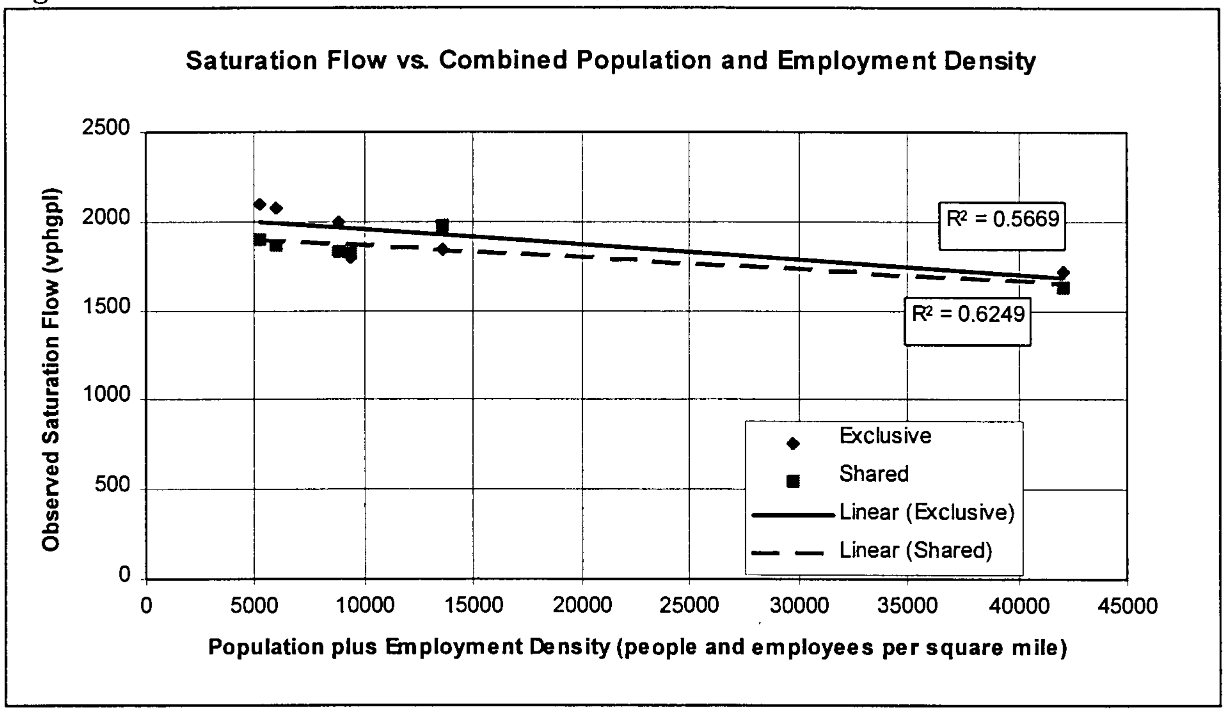


Figure 3 illustrates results similar to **Figure 2**. Although a slight downward trend did occur in saturation flow as combined employment and population density increased, it is apparent that more surveys would need to be performed in order to obtain a more accurate correlation. A wider range of area types should also be examined.

Other Characteristics

Characteristics other than density which can affect saturation flow rates are driveway spacing and volume, pedestrian activity, and transit activity. Although these characteristics are likely to be directly related to density, each was compared independently to the saturation flow data to determine if any strong correlation exists.

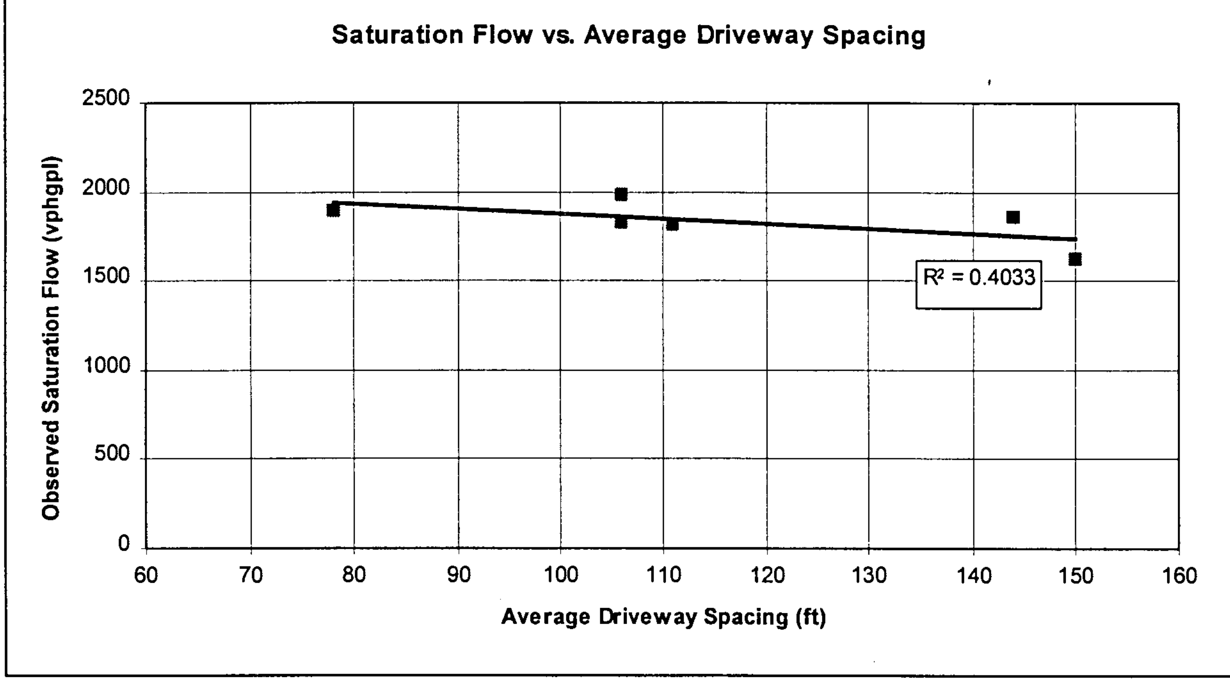
Driveways

The distance from the stop bar to each driveway along the subject approach was measured at each study location to determine the average driveway spacing. **Table 5** summarizes the average driveway spacing for each location. The average spacing was plotted against the observed saturation flow rates for the shared through/turn lanes in **Figure 4** to determine if a relationship exists.

Table 5. Summary of Average Driveway Spacing and Pedestrian Activity

Location	Saturation Flow (Exclusive) (vphgpl)	Saturation Flow (Shared) (vphgpl)	Average Driveway Spacing (feet)	Pedestrian Activity (peds/hr)
1. Southbound Commercial St. at Kuebler Rd.	2077	1861	144	5
2. Northbound Commercial St. at Madrona Ave.	1843	1981	106	0
3. Southbound Commercial St. at Owens St.	1802	1817	111	4
4. Northbound Commercial St. at Hoyt St.	2103	1900	78	3
5. Westbound Ferry St. at Liberty St.	1721	1629	150	36
6. Eastbound Mission St. and 25th St.	1998	1826	106	0

Figure 4.

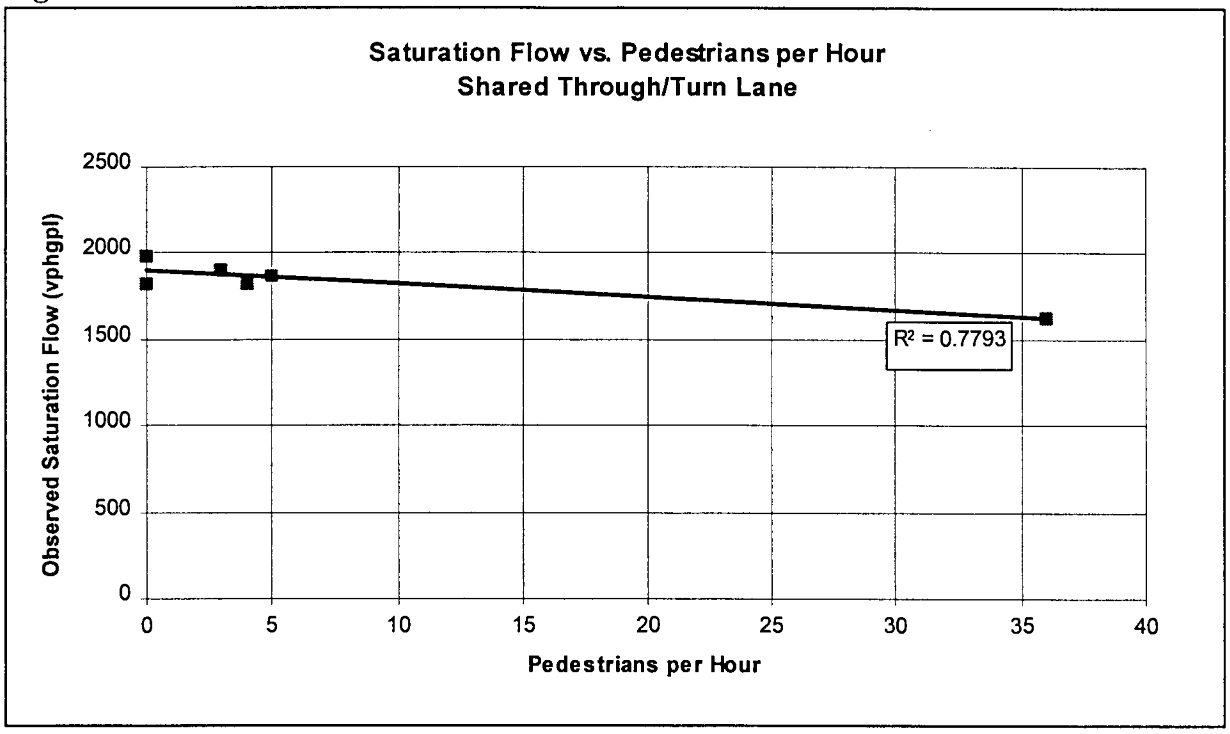


It was expected that with a decrease in driveway spacing, there would be a decrease in the saturation flow rate due to interference by driveway activity. However, this trend was not observed at the locations included in this study.

Pedestrian Activity

Pedestrians were counted at each location during the data collection period. Pedestrians which would impede a turning movement were counted (those which crossed adjacent to the shared through/turn lane). An hourly pedestrian rate was calculated for each location and was compared to the saturation flow for the shared lane. The calculated pedestrian rates for each location were previously shown in **Table 5**. **Figure 5** shows a graphical comparison of the observed saturation flow rate for the shared lane and the pedestrian rate.

Figure 5.



As seen in **Figure 5**, the general trend was for saturation flow in the shared through/turn lane to decrease as pedestrian traffic increased. This relationship was expected due to the fact that pedestrians can impede turning movements, thus reducing the flow rate for that approach. Pedestrian volumes had the strongest correlation with saturation flow of all of the area type variables collected. However, using pedestrian activity as a method to calculate an area factor would be difficult to implement, given the required pedestrian data. More locations should be surveyed to more accurately determine the specific correlation between pedestrian activity and saturation flow.

Transit Activity

Transit vehicles were counted for each location during the data collection period. Each location surveyed had 1-3 vehicles per hour, and therefore a range was not available to examine any correlation between saturation flow and transit activity. It would be necessary to study intersections with a wide range of transit activity to determine if a correlation exists.

CAPACITY CALCULATIONS

Capacities for the through lanes on each subject approach were calculated using three different analysis techniques:

1. **PB Methodology.** Capacity was calculated using **equation 3** below, from the "Highway Network Capacity Specification, Draft Methodology". Left turn capacity was not considered in this analysis.

$$C = L \cdot (g / C) \cdot 1900 \tag{3}$$

where

C = capacity (vph)
L = number of through lanes
g/C = green to cycle ratio

2. **Highway Capacity Manual Methodology.** The HCM methodology for capacity analysis at signalized intersections was used to determine the capacity of the lane group. Several characteristics of the intersections where assumed in order to allow calculation of capacity, including the heavy vehicle percentage and the proportion of right turns. Highway Capacity Software was used to simplify this analysis.

3. **Observed Capacity.** Capacity was calculated by applying g/C to the observed saturation flows for each lane. This is represented by **equation 4**:

$$C = (g / C) \cdot \left[\sum s_i\right] \tag{4}$$

where

C = capacity (vph)
g/C = green to cycle ratio
s_i = observed saturation flow for ith lane (vphgpl)

The resulting capacities for each location are summarized in **Table 5**. Calculations are included in the appendix of this report.

Table 5. Summary of Capacity Results

Location	Capacity (vph)		
	PB Method	HCM Method	Observed
1. Southbound Commercial Street at Kuebler Road	1368	1347	1418
2. Northbound Commercial Street at Madrona Avenue	1976	1929	1988
3. Southbound Commercial Street at Owens Street	2584	2551	2461
4. Northbound Commercial Street at Hoyt Street	2888	2803	3042
5. Westbound Ferry Street at Liberty Street	1710	1502	1508
6. Eastbound Mission Street and 25th Street	1748	1711	1759

Observed capacities were generally slightly higher than those calculated using the PB methodology. The exception is the downtown intersection (Location 5. WB Ferry St. at Liberty St.), where the observed capacity was significantly lower than the PB capacity. This is likely due to the impact of the area type on capacity. The HCM capacities were slightly lower than the PB capacities due to the additional saturation flow reduction factors for heavy vehicles and right turns. At Location 5, a further reduction was made in the HCM capacity because the intersection is located in the Central Business District.

AREA TYPE ADJUSTMENT FACTOR

In order to capture the impact area type has on network capacity in travel demand models, an adjustment factor was developed which can be applied to the service flow rates calculated in the PB model. This area type factor, f_{area}, would be added to **equation 3** as shown below to create **equation 5.**

$$C = f_{area} \cdot L \cdot (g / C) \cdot 1900 \tag{5}$$

where

C = capacity (vph)
$farea$ = area type adjustment factor
L = number of through lanes
g/C = green to cycle ratio

Values of f_{area}

The following methodology was used to calculate values for f_{area} as a function of both combined employment and population density and the number of through lanes:

The area type factor, f_{area}, relates the capacity calculated using the PB methodology shown in **equation 3** (C_{pb}) to the capacity calculated based on the observed saturation flows (C_{obs}). This relationship is shown in **equation 6.**

$$C_{obs} = f_{area} \cdot C_{pb} \tag{6}$$

where

C_{obs} = capacity based on observed saturation flows (vph)
$farea$ = area type adjustment factor
C_{pb} = capacity calculated using PB methodology (vph)

C_{obs} was calculated for a range of densities by obtaining saturation flows from the fitted lines in **Figure 3**. C_{obs} was also calculated for approaches of one, two, three, and four through lanes by summing the appropriate combinations of saturation flows for exclusive and shared lanes. C_{pb} was calculated from **equation 3** for one, two, three, and four through lanes. The values for area type factor, f_{area}, were determined by inserted the calculated values for C_{obs} and C_{pb} into **equation (6)** and solving for f_{area}. The resulting values for f_{area} are summarized in **Table 6.**

Table 6. Values of Area Type Adjustment Factor, f_{area}

Combined Population and Employment Density (residents and employees per sq. mi.)	Number of Through and Shared Through/Right Turn Lanes			
	1 Lane	2 Lanes	3 Lanes	4 Lanes
5,000	1.00	1.03	1.04	1.04
10,000	0.98	1.01	1.02	1.02
15,000	0.96	0.99	0.99	1.00
20,000	0.95	0.97	0.97	0.98
25,000	0.93	0.95	0.95	0.96
30,000	0.91	0.93	0.93	0.93
35,000	0.89	0.91	0.91	0.91
40,000	0.88	0.89	0.89	0.89
45,000	0.86	0.87	0.87	0.87

* May include shared through/left turn lanes on one-way streets.

The values in **Table 6** reflect the reduction in capacity observed as density increased. Factors were less than one for all locations with combined population and employment densities greater than 20,000 per square mile.

In determining these factors, it was assumed that the right-hand lane was always a shared through/right turn lane. In locations with exclusive right turn lanes, the area type adjustment factors would be slightly lower. The factors in **Table 6** provide a conservative estimate of capacity.

Capacity of exclusive left turn lanes was not included in this analysis. The factors in **Table 6** apply only to capacity for through and right turn movements.

Capacity of shared through/left turn lanes on two-way streets was not examined in this analysis. It is expected that roadways which allow permitted left turns from a shared through lane would have a further reduction in capacity.

The values in **Table 6** are based on a limited number of samples and should be used with caution. Additional data collection should be completed at locations ranging in density (15,000 to 40,000 persons and employees per square mile) to verify the relationship between density and saturation flow illustrated in **Figure 3.** Values for f_{area} shown in **Table 6** should be updated if additional data collection is completed.

CONCLUSIONS

An area type factor, f_{area}, was developed which can be applied to the capacities calculated in the PB model for through and shared through/right turn lanes. The purpose of this factor is to model the impact area type has on network capacity in travel demand models. Values of f_{area} as a function of combined population and employment density and the number of through lanes are presented in **Table 6**.

The values for f_{area} were determined by correlating saturation flows in through lanes and shared through/right turn lanes to density. Actual saturation flows were determined on approaches to six arterial intersection in the Salem/Keizer metropolitan area. The observed saturation flows were plotted against the density. Two trends were observed:

1. Saturation flow generally decreased as combined population and employment density increased.
2. Saturation flows for shared through/turn lanes were generally lower than saturation flows for exclusive through lanes.

While it was possible to calculate values of f_{area} from the data collected, they are based on a limited number of samples and should be used with caution. Additional data collection should be completed at locations ranging in density density (15,000 to 40,000 persons and employees per square mile) to verify the relationship between density and capacity.

APPENDIX

FIELD DATA AND SUMMARY SHEETS

Salem Capacity Data

Location: SB Commercial Street at Kuebler Road
Lane: Exclusive Thru

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	7.8	19.6	11	7	1.7	11.76
2	7.4	15.9	9	5	1.7	8.53
3	8.8	21.4	12	8	1.6	12.63
4	8.4	28.8	16	12	1.7	20.38
5	8.4	21.8	12	8	1.7	13.41
6	8.6	22.4	11	7	2.0	13.82
7	8.1	22.6	13	9	1.6	14.5
8	9.4	25.3	13	9	1.8	15.9
9	10.2	22.3	11	7	1.7	12.1
10	9.2	19.8	10	6	1.8	10.6
11	7.8	18.1	10	6	1.7	10.3
12	8.8	22.8	12	8	1.8	14
13	9.0	17.4	9	5	1.7	8.4
14	7.5	17.1	10	6	1.6	9.6
15	7.5	14.6	7	3	2.4	7.1
16	9.5	20.6	10	6	1.9	11.1
17	7.0	19.6	11	7	1.8	12.6
18	10.8	18.7	8	4	2.0	7.9
19	9.4	17.5	9	5	1.6	8.1
20	11.3	24.3	12	8	1.6	13
			Total	*136*		*235.73*

Average Time for 1st 4: 9.0

Average Time Headway: 1.73

Saturation Flow: 2077

Average Lost Time/Cycle: 2.05

Salem Capacity Worksheet

Initials: BES

Location: Commercial / Kuebler

Time: 5:05pm

Date: 4.15.97

Approach: SB thru

Cycle Length: 120

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	7.84	19.60	11		
2	7.37	15.97	9		
3	8.77	21.37	12		
4	8.42	28.75	16		
5	8.39	21.84	12		
6	0	16.97	10		
7	8.58	22.38	11		
8	8.17	22.59	13		
9	9.39	25.34	13		
10	10.21	22.28	11		
11	9.17	19.81	10		
12	7.80	18.09	10		
13	8.77	22.78	12		
14	8.99	17.43	9		
15	7.46	17.09	10		
16	7.95	14.59	7		
17	9.52	20.59	10		
18	6.96	19.59	11		
19	10.83	18.66	8		
20	9.37	17.47	9		
21	11.33	24.31	12		
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance
1	
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Salem Capacity Data

Location: SB Commercial Street at Kuebler Road
Lane: Shared Thru/Right

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	8.3	37.8	18	14	2.1	29.47
2	9.3	36.3	19	15	1.8	27
3	10.1	25.8	13	9	1.7	15.7
4	9.1	32.9	15	11	2.2	23.8
5	8.6	28.8	15	11	1.8	20.2
6	8.4	21.6	12	8	1.7	13.2
7	8.3	18.9	10	6	1.8	10.6
8	8.4	19.9	10	6	1.9	11.5
9	10.2	24.9	12	8	1.8	14.7
10	8.3	23.1	11	7	2.1	14.8
11	8.9	22.2	11	7	1.9	13.3
12	8.9	22.3	11	7	1.9	13.4
13	11.8	21.7	10	6	1.7	9.9
14	11.8	26.5	11	7	2.1	14.7
15	9.3	18.6	8	4	2.3	9.3
16	8.9	24.6	11	7	2.2	15.7
			Total	*133*		*257.27*

Average Time for 1st 4: 9.4

Average Time Headway: 1.93

Saturation Flow: 1861

Average Lost Time/Cycle: 1.65

Salem Capacity Worksheet

Initials: WFB

Location: Commercial / Kuebler

Time: 5:05 - 6:00

Date: 4.15.97

Approach: NB Shared thru

Cycle Length: 120

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	8.33	37.8	18	3	1
2	9.3	36.3	19		
3	10.1	25.8	13		
4	9.1	32.9	15		
5	8.6	28.9	15		
6	8.4	21.6	12		
7	8.3	16.9	10		
8	8.4	19.9	10		
9	10.2	24.9	12		
10	12.2	14.15	5	1	
11	8.3	23.1	11		
12	8.9	22.2	11		
13	8.9	22.3	11		
14	7.6	11.5	5	1	
15	11.8	21.7	10		1
16	10.2	13.3	5		
17	12.1	32.8	13		
18	12.9	26.5	11		
19	10.9		4		
20	9.3	18.6	8		
21	8.9	24.6	11		
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance	
1	122	SHARI'S RT. OUT
2	156	SHARI'S RT IN
3	280	BOONE ST.
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Salem Capacity Data

Location: NB Commercial Street at Madrona Avenue
Lane: Exclusive Thru

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	10.3	27.4	11	7	2.4	17.1
2	6.2	25.4	13	9	2.1	19.2
3	8.3	24.8	13	9	1.8	16.5
4	8.2	31.0	15	11	2.1	22.8
5	8.3	19.8	10	6	1.9	11.5
6	8.6	24.6	13	9	1.8	16
7	8.1	25.7	13	9	2.0	17.6
8	7.4	21.6	11	7	2.0	14.2
9	8.6	20.7	11	7	1.7	12.1
10	8.9	26.3	13	9	1.9	17.4
11	6.9	16.3	10	6	1.6	9.4
12	7.7	16.8	7	3	3.0	9.1
13	10.0	20.7	10	6	1.8	10.7
14	9.3	14.6	7	3	1.8	5.3
15	8.1	14.4	7	3	2.1	6.3
16	8.7	15.8	8	4	1.8	7.1
17	9.1	18.3	9	5	1.8	9.2
18	8.8	20.4	10	6	1.9	11.6
19	8.7	13.9	7	3	1.7	5.2
			Total	*122*		*238.3*

Average Time for 1st 4: 8.4

Average Time Headway: 1.95

Saturation Flow: 1843

Average Lost Time/Cycle: 0.62

Salem Capacity Worksheet

Initials: WFB

Location: commercial/madrona Time: 7:45 Date: 4.16.97

Approach: NB thru Cycle Length: ~ 130

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	10.3	27.4	11		
2	6.2	25.4	13		
3	8.3	24.8	13		
4	8.2	31.0	15		
5	8.3	19.8	10		
6	8.6	24.6	13		
7	8.1	25.7	13		
8	7.4	21.6	11		
9	8.6	20.7	11		
10	8.9	26.3	13		
11	6.9	16.3	10		
12	7.7	16.8	7		
13	6.1	8.3	5		
14	10.0	20.7	10		
15	9.3	14.6	7		
16	8.1	14.4	7		
17	8.7	15.8	8		
18	9.1	18.3	9		
19	8.8	20.4	10		
20	8.7	13.9	7		
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing
(From stop bar)

Driveway #	Distance	
1	14	RT. PORK CHOP
2	116	CHEVRON
3	210	CHEVRON
4	258	B.K.
5	370	FURNITURE
6	535	FURNITURE
7	590	P.O.
8	785	P.O.
9	958	HARRIS AVE
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Salem Capacity Data

Location: NB Commercial Street at Madrona Avenue
Lane: Shared Thru/Right with Right Turn Pork Chop

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	9.5	17.4	8	4	2.0	7.91
2	6.9	16.6	10	6	1.6	9.67
3	8.9	19.5	10	6	1.8	10.64
4	7.1	22.1	12	8	1.9	14.98
5	7.1	20.3	12	8	1.7	13.25
6	8.1	22.0	11	7	2.0	13.9
7	8.5	25.9	13	9	1.9	17.4
8	8.9	19.0	10	6	1.7	10.1
9	8.2	20.4	11	7	1.7	12.2
10	7.6	17.0	9	5	1.9	9.4
11	9.8	15.4	8	4	1.4	5.6
12	8.1	25.4	14	10	1.7	17.3
13	4.3	19.4	11	7	2.2	15.1
14	8.4	13.4	7	3	1.7	5
15	7.2	10.5	6	2	1.7	3.3
16	9.3	15.2	7	3	2.0	5.9
17	8.4	14.7	7	3	2.1	6.3
18	8.7	18.0	9	5	1.9	9.3
19	11.0	18.2	8	4	1.8	7.2
			Total	*107*		*194.45*

Average Time for 1st 4: 8.4

Average Time Headway: 1.82

Saturation Flow: 1981

Average Lost Time/Cycle: 1.16

Salem Capacity Worksheet

Initials: BFG

Location: Commercial / Madrona

Time: 7:40-8:20

Date: 4-10-97

Approach: NB thru/Rt

Cycle Length: ~130

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	949	1743	8		
2	6.93	16.62	10		
3	8.86	19.50	10		
4	7.12	22.10	12		
5	7.05	2028	12		
6	8.14	22.03	11		1
7	8.49	25.91	13		
8	8.90	18.97	10		
9	8.05	20.44	11		
10	7.61	17.00	9		
11	9.77	1544	8		1
12	8.11	25.37	14		
13	7.33	19.37	11		
14	8.37	13.35	7		
15	7.15	10.53	6		
16	927	15.18	7		
17	8.40	14.69	7		
18	8.68	18.03	9		
19	10.48	18.16	8		1
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing
(From stop bar)

Driveway #	Distance (WEST)	EAST
1	45'-CAR RENTAL	57' DRYCLEAN
2	158'-COMP DR.	158' PACK & SHIP
3		332' BUSH ST.
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Salem Capacity Data

Location: SB Commercial Street at Owens Street
Lane: Exclusive Thru

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	9.9	28.5	14	10	1.9	18.6
2	9.9	23.8	11	7	2.0	13.9
3	10.4	26.8	12	8	2.1	16.4
4	10.1	23.0	10	6	2.2	12.9
5	8.2	27.1	14	10	1.9	18.9
6	12.4	31.7	14	10	1.9	19.3
7	10.2	22.4	10	6	2.0	12.2
8	10.8	21.1	9	5	2.1	10.3
9	10.0	26.9	12	8	2.1	16.9
10	10.0	22.0	10	6	2.0	12
11	8.2	18.0	9	5	2.0	9.8
12	10.1	28.7	13	9	2.1	18.6
13	8.2	16.4	7	3	2.7	8.2
14	10.2	20.2	9	5	2.0	10
15	8.2	24.0	13	9	1.8	15.8
			Total	*107*		*213.8*

Average Time for 1st 4: 9.8

Average Time Headway: 2.00

Saturation Flow: 1802

Average Lost Time/Cycle: 1.79

Salem Capacity Worksheet

Initials: BES

Location: Commercial/Owens

Time: 5:10

Date: 4-16-97

Approach: SB thru.

Cycle Length: 130

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	9.97	28.54	14		
2	9.92	23.84	11		
3	10.43	26.82	12		
4	10.05	2303	10		
5	8.24	27.10	14		
6	12.37.	31.72	14		
7	10.17	22.40	10		
8	10.77	21.07.	9		
9	9.95	26.87.	12		
10	10.01	22.00.	10		
11	8.21	18.00	9		
12	10.06	28.66.	13		
13	8.24	16.41.	7		
14	10.18	20.16	9		
15	9.17	23.97	13		
16					
17					
18					
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing
(From stop bar)

Driveway #	Distance
1	
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Salem Capacity Data

Location: SB Commercial Street at Owens Street
Lane: Shared Thru/Left

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	9.8	26.7	13	9	1.9	16.9
2	10.5	28.1	13	9	2.0	17.6
3	6.9	22.1	11	7	2.2	15.2
4	10.6	26.2	12	8	2.0	15.6
5	9.6	29.5	14	10	2.0	19.9
6	8.8	33.9	17	13	1.9	25.1
7	10.6	22.0	10	6	1.9	11.4
8	9.2	29.5	13	9	2.3	20.3
9	8.9	22.9	11	7	2.0	14
10	8.6	23.2	11	7	2.1	14.6
11	9.9	24.5	11	7	2.1	14.6
12	13.3	26.7	11	7	1.9	13.4
13	11.0	27.3	13	9	1.8	16.3
14	9.9	18.9	9	5	1.8	9
			Total	*113*		*223.9*

Average Time for 1st 4: 9.8

Average Time Headway: 1.98

Saturation Flow: 1817

Average Lost Time/Cycle: 1.90

Salem Capacity Worksheet

Initials: WFB

Location: Commercial / Owens.

Time: 505

Date: 4.16.97

Approach: SB thru / left.

Cycle Length: 130

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	9.8	26.7	13		
2	10.5	28.1	13		
3	6.9	22.1	11		
4	10.6	26.2	12		
5	9.6	29.5	14		
6	8.8	33.9	17		
7	10.6	22.0	10		
8	9.2	29.5	13		
9	8.9	22.9	11		
10	8.6	23.2	11	1	
11	9.9	24.5	11		
12	10.7	15.2	6		
13	7.6	-	4		
14	13.3	26.7	11		
15	11.0	27.3	13		
16	9.9	18.9	9	3	
17					
18					
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance
1	45.
2	158
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Salem Capacity Data

Location: NB Commercial Street at Hoyt Street
Lane: Exclusive Thru

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	8.2	25.0	14	10	1.7	16.76
2	9.2	26.7	15	11	1.6	17.5
3	10.8	20.7	10	6	1.7	9.9
4	7.8	26.9	15	11	1.7	19.1
5	9.3	25.1	12	8	2.0	15.8
6	11.2	28.7	14	10	1.8	17.5
7	7.3	21.6	12	8	1.8	14.3
8	11.2	26.5	14	10	1.5	15.3
9	9.2	24.4	13	9	1.7	15.2
10	8.2	19.3	10	6	1.9	11.1
11	8.5	15.8	9	5	1.5	7.3
12	8.5	21.6	11	7	1.9	13.1
13	8.9	26.7	14	10	1.8	17.8
14	9.1	23.2	12	8	1.8	14.1
15	9.0	21.0	12	8	1.5	12
16	9.1	19.2	9	5	2.0	10.1
17	9.2	21.5	12	8	1.5	12.3
18	9.7	22.2	11	7	1.8	12.5
			Total	*147*		*251.66*

Average Time for 1st 4: 9.0

Average Time Headway: 1.71

Saturation Flow: 2103

Average Lost Time/Cycle: 2.19

Salem Capacity Worksheet

Initials: BES

Location: Commercial / Hoyt

Time: 7:30

Date: 4.17.97

Approach: NB Thru

Cycle Length: 145

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	9.24	25.00	14		
2	9.17	26.72	15		
3	10.77	20.65	10		
4	7.80	26.91	15		
5	9.27	25.06	12		
6	11.21	28.72	14		
7	7.33	21.56	12		
8	11.21	26.47	14		
9	9.24	24.35	13		
10	8.17	19.31	10		
11	8.46	15.78	9		
12	8.46	21.57	11		
13	8.92	26.72	14		
14	9.08	23.22	12		
15	9.02	20.97	12		
16	9.06	19.19	9		
17	9.21	21.50	12		
18	9.65	22.16	11		
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance	
1	66'	STRIP MALL
2	156'	Q LUBE
3	206'	CAR RENTAL
4	313'	JUDSON ST.
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Salem Capacity Data

Location: NB Commercial Street at Hoyt Street
Lane: Shared Thru/Right

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	8.2	23.1	11	7	2.1	14.9
2	7.6	26.5	14	10	1.9	18.9
3	9.5	27.6	14	10	1.8	18.1
4	11.0	24.6	12	8	1.7	13.6
5	10.1	26.3	14	10	1.6	16.2
6	10.4	18.9	9	5	1.7	8.5
7	11.1	25.3	11	7	2.0	14.2
8	11.1	29.3	13	9	2.0	18.2
9	8.8	23.4	12	8	1.8	14.6
10	10.8	30.1	13	9	2.1	19.3
11	8.6	18.7	10	6	1.7	10.1
12	9.5	26.8	12	8	2.2	17.3
13	9.3	24.8	11	7	2.2	15.5
14	8.6	22.1	12	8	1.7	13.5
15	11.6	25.2	12	8	1.7	13.6
16	9.2	17.3	8	4	2.0	8.1
17	8.9	20.6	10	6	2.0	11.7
			Total	*130*		*246.3*

Average Time for 1st 4: 9.6

Average Time Headway: 1.89

Saturation Flow: 1900

Average Lost Time/Cycle: 2.00

Salem Capacity Worksheet

Initials:

Location: Commercial / Hoyt.

Time: 730 - 830.

Date: 4.17.

Approach: NB Thru / RT.

Cycle Length: 145

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	8.2	23.09	11		
2	7.6	26.5	14		
3	9.5	27.6	14		
4	11.0.	24.6	12		
5	10.1	26.3	14		
6	10.4	18.9	9		
7	11.1.	25.3	11.		
8	11.1	29.3.	13		
9	8.8	23.4	12		
10	10.8	30.1	13		
11	8.6	13.4	7		2
12	8.6.	18.7.	10		
13	8.4.	13.2.	7		
14	9.5.	26.8	12	1	
15	9.3	24.8	11		
16	9.6	22.1	12		
17	11.6	25.2	12		
18	9.2	17.3	8		
19	8.9	20.6.	10.	2	1
20					
21					1
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance
1	66
2	156.
3	206.
4	313
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Salem Capacity Data

Location: WB Ferry Street at Liberty Street
Lane: Exclusive Thru

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	9.7	17.3	8	4	1.9	7.65
2	9.3	19.4	8	4	2.5	10.1
3	12.0	23.8	10	6	2.0	11.8
4	10.6	20.4	10	6	1.6	9.8
5	13.3	24.7	10	6	1.9	11.4
6	9.0	16.6	8	4	1.9	7.6
7	10.4	21.6	9	5	2.2	11.2
8	11.2	20.3	9	5	1.8	9.1
9	11.6	24.8	10	6	2.2	13.2
10	10.1	22.6	10	6	2.1	12.5
11	9.0	21.9	10	6	2.2	12.9
12	7.9	17.7	8	4	2.5	9.8
13	9.0	20.5	9	5	2.3	11.5
14	8.9	19.0	10	6	1.7	10.1
15	9.4	17.9	7	3	2.8	8.5
16	9.2	18.3	8	4	2.3	9.1
17	9.6	21.1	9	5	2.3	11.5
18	9.6	20.1	9	5	2.1	10.5
			Total	*90*		*188.25*

Average Time for 1st 4: 10.0

Average Time Headway: 2.09

Saturation Flow: 1721

Average Lost Time/Cycle: 1.68

Salem Capacity Worksheet

Initials: BES

Location: WB Ferry / Liberty

Time: 4:30.

Date: 4-17-97.

Approach: WB Excl. thru.

Cycle Length: 64

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	9.65	17.25	8		
2	9.33	19.41	8		
3	11.96	23.82	10		
4	10.58	20.43	10		
5	13.30	24.65	10		
6	8.96	16.57	8		
7	10.40	21.57	9		
8	11.20	20.35	9		
9	11.55	24.79	10		
10	10.05	22.56	10		
11	9.07	21.91	10		
12	7.86	17.69	8		
13	8.96	20.54	9		
14	9.55	19.00	10		
15	9.40	17.85	7		
16	9.24	18.79	8		
17	9.64	21.09	9		
18	9.58	20.13	9		
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing
(From stop bar)

Driveway #	Distance
1	MID-BLOCK ALLEY
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Salem Capacity Data

Location: WB Ferry Street at Liberty Street
Lane: Shared Thru/Right

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	12.6	22.0	8	4	2.4	9.4
2	13.5	19.7	7	3	2.1	6.2
3	12.6	21.2	8	4	2.2	8.6
4	8.2	18.6	9	5	2.1	10.4
5	9.9	19.9	8	4	2.5	10
6	8.6	16.2	8	4	1.9	7.6
7	10.1	20.2	9	5	2.0	10.1
8	9.7	24.4	10	6	2.5	14.7
9	11.3	23.8	10	6	2.1	12.5
10	12.1	29.6	12	8	2.2	17.5
11	12.0	23.3	9	5	2.3	11.3
12	9.7	19.6	8	4	2.5	9.9
13	16.1	27.9	9	5	2.4	11.8
14	12.0	20.3	8	4	2.1	8.3
15	11.3	21.5	9	5	2.0	10.2
16	13.4	19.5	7	3	2.0	6.1
17	11.4	22.0	9	5	2.1	10.6
18	11.6	21.9	8	4	2.6	10.3
19	11.6	21.3	8	4	2.4	9.7
20	7.0	17.9	9	5	2.2	10.9
21	12.0	23.9	10	6	2.0	11.9
22	9.5	21.3	9	5	2.4	11.8
			Total	*104*		*229.8*

Average Time for 1st 4: 11.0

Average Time Headway: 2.21

Saturation Flow: 1629

Average Lost Time/Cycle: 2.12

Salem Capacity Worksheet

Initials: WFB

Location: WB Ferry/Liberty

Time: 5:00-5:25

Date:

Approach: WB SHARED THRU/RT.

Cycle Length: 64

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	12.6.	22.	8	4	
2	13.5	19.7.	7	1	
3	11.5	16.3	6	1	
4	12.6	21.2	8		
5	8.2	18.6	9	1	
6	9.9	19.9	8.	2	
7	8.6.	16.2	8		
8	10.1	20.2	9		
9	9.7	24.4	10		
10	11.3	23.9	10		
11	12.1	29.8	12	1	
12	12.0	23.3	9	2	
13	9.7.	19.6	8		1
14	16.1	27.9	9		
15	12.0	20.3	8	1	
16	11.3	21.5	9	1	
17	13.4	19.5	7	2	
18	11.4	22.0	9		1
19	11.6	21.9	8.		
20	11.6	21.3	8.		
21	7.0.	17.9	9		
22	12.0.	23.9	10		
23	9.5	21.3	9	11	
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance
1	MIDBLOCK ALLEY
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Salem Capacity Data

Location: EB Mission and 25th
Lane: Exclusive Thru

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	10.6	23.6	12	8	1.6	13
2	9.9	28.0	13	9	2.0	18.1
3	9.3	29.4	16	12	1.7	20.1
4	9.1	28.0	14	10	1.9	18.9
5	10.7	32.7	17	13	1.7	22
6	8.1	26.7	14	10	1.9	18.6
7	9.5	29.7	15	11	1.8	20.2
8	10.7	28.2	14	10	1.8	17.5
9	11.4	25.0	12	8	1.7	13.6
10	10.5	23.4	11	7	1.8	12.9
11	8.9	26.9	13	9	2.0	18
12	10.6	25.5	12	8	1.9	14.9
13	10.4	25.3	12	8	1.9	14.9
14	10.2	22.4	11	7	1.7	12.2
15	9.8	17.3	8	4	1.9	7.5
16	15.9	25.7	10	6	1.6	9.8
			Total	*140*		*252.2*

Average Time for 1st 4: 10.0

Average Time Headway: 1.80

Saturation Flow: 1998

Average Lost Time/Cycle: 2.77

Salem Capacity Worksheet

Initials: WFB

Location: Mission / 25th

Time: 505 - 550

Date: 4-23

Approach: EB Mission - excl. thru

Cycle Length: ~128

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	10.6	23.6	12		
2	9.9	28.0	13		
3	9.3	29.4	16		
4	9.1	28.0	14		
5	10.7	32.7	17		
6	8.1	26.7	14		
7	9.5	29.7	15		
8	10.7	28.2	14		/
9	11.4	25.0	12		
10	10.5	23.4	11		
11	8.9	26.9	13		
12	10.6	25.5	12		
13	10.4	25.3	12		
14	10.7	22.4	11		
15	9.9	17.3	8		
16	15.9	25.7	10		
17					
18					
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

Approach Driveway Spacing

(From stop bar)

Driveway #	Distance	
1	64	PLAID PANTRY
2	212	K-MART
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Salem Capacity Data

Location: EB Mission and 25th
Lane: Shared Thru/Right

Cycle #	T_4	T_{last}	Total Number	Number-4	Average Time Headway	Headway*Number
1	11.1	24.9	15	11	1.3	13.8
2	9.4	27.6	13	9	2.0	18.2
3	9.2	31.3	14	10	2.2	22.1
4	9.5	28.3	15	11	1.7	18.8
5	7.8	31.5	15	11	2.2	23.7
6	7.4	33.4	17	13	2.0	26
7	9.7	26.2	11	7	2.4	16.5
8	11.7	26.9	11	7	2.2	15.2
9	10.8	33.3	14	10	2.3	22.5
10	13.1	33.3	15	11	1.8	20.2
11	11.1	37.8	16	12	2.2	26.7
12	10.6	23.9	12	8	1.7	13.3
13	12.8	28.4	12	8	2.0	15.6
14	9.3	17.3	8	4	2.0	8
15	10.1	21.3	9	5	2.2	11.2
16	11.3	25.8	12	8	1.8	14.5
17	10.4	31.6	15	11	1.9	21.2
			Total	*156*		*307.5*

Average Time for 1st 4: 10.3

Average Time Headway: 1.97

Saturation Flow: 1826

Average Lost Time/Cycle: 2.43

Salem Capacity Worksheet

Initials: AM

Location: Mission/25th

Time: 505-550

Date: 4.23.97

Approach: WB Mission shared thru/rt

Cycle Length: ~128

Cycle #	T_4	T_{last}	Number	Peds	Busses
1	11.1	24.9	15		
2	9.4	27.6	13		
3	9.2	31.3	14		
4	9.5	28.3	15		
5	7.8	31.5	15		
6	9.4	33.4	17		
7	9.7	26.2	11		
8	11.7	26.9	11		
9	10.8	33.3	14		
10	13.1	33.3	15		
11	11.1	37.8	16		
12	10.6	23.9	12		
13	12.9	29.4	12		
14	9.3	17.3	8		
15	10.1	21.3	9		
16	11.3	25.8	12		
17	10.4	31.6	15		
18					
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					
31					
32					
33					
34					
35					
36					
37					
38					
39					
40					

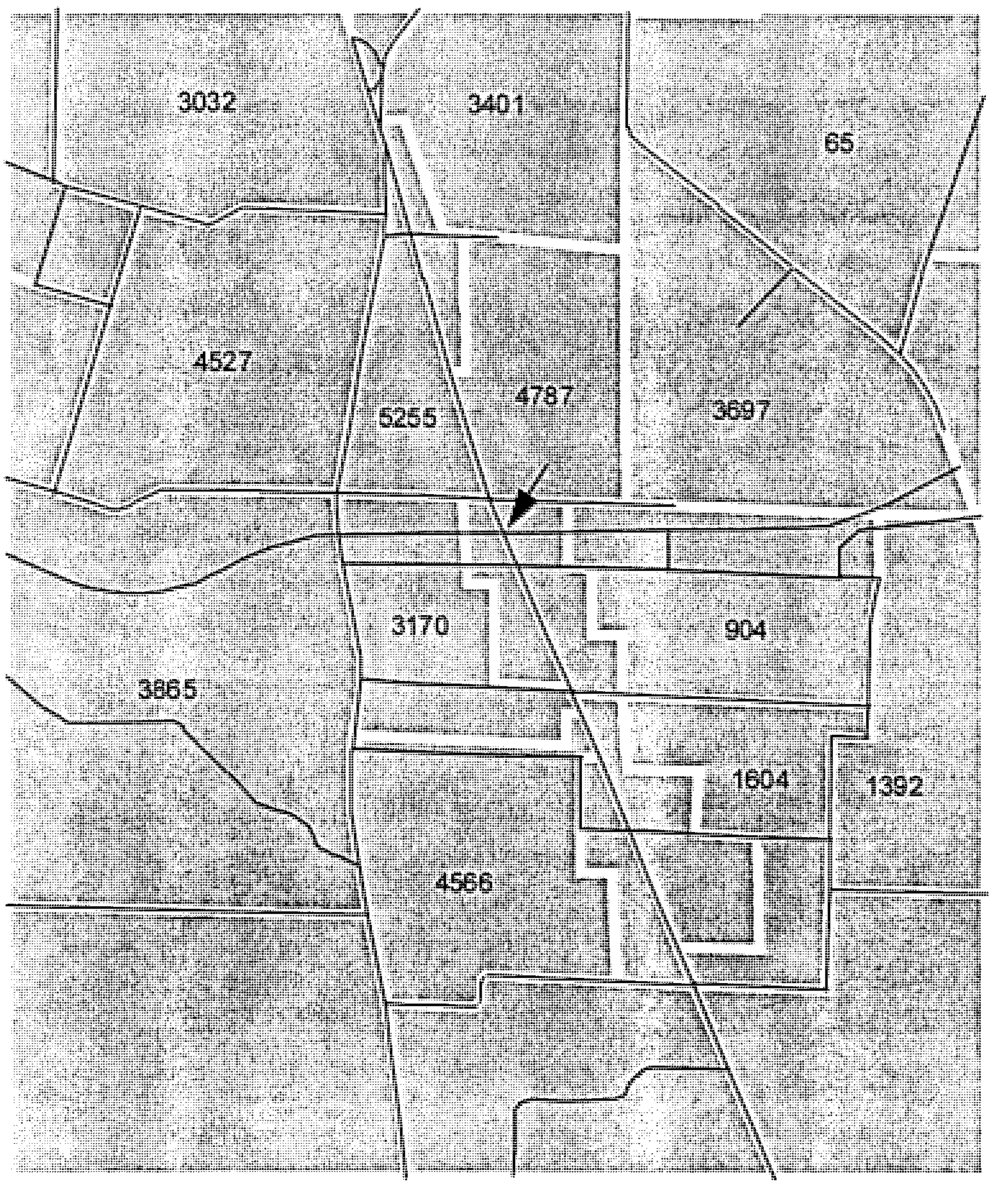
Approach Driveway Spacing

(From stop bar)

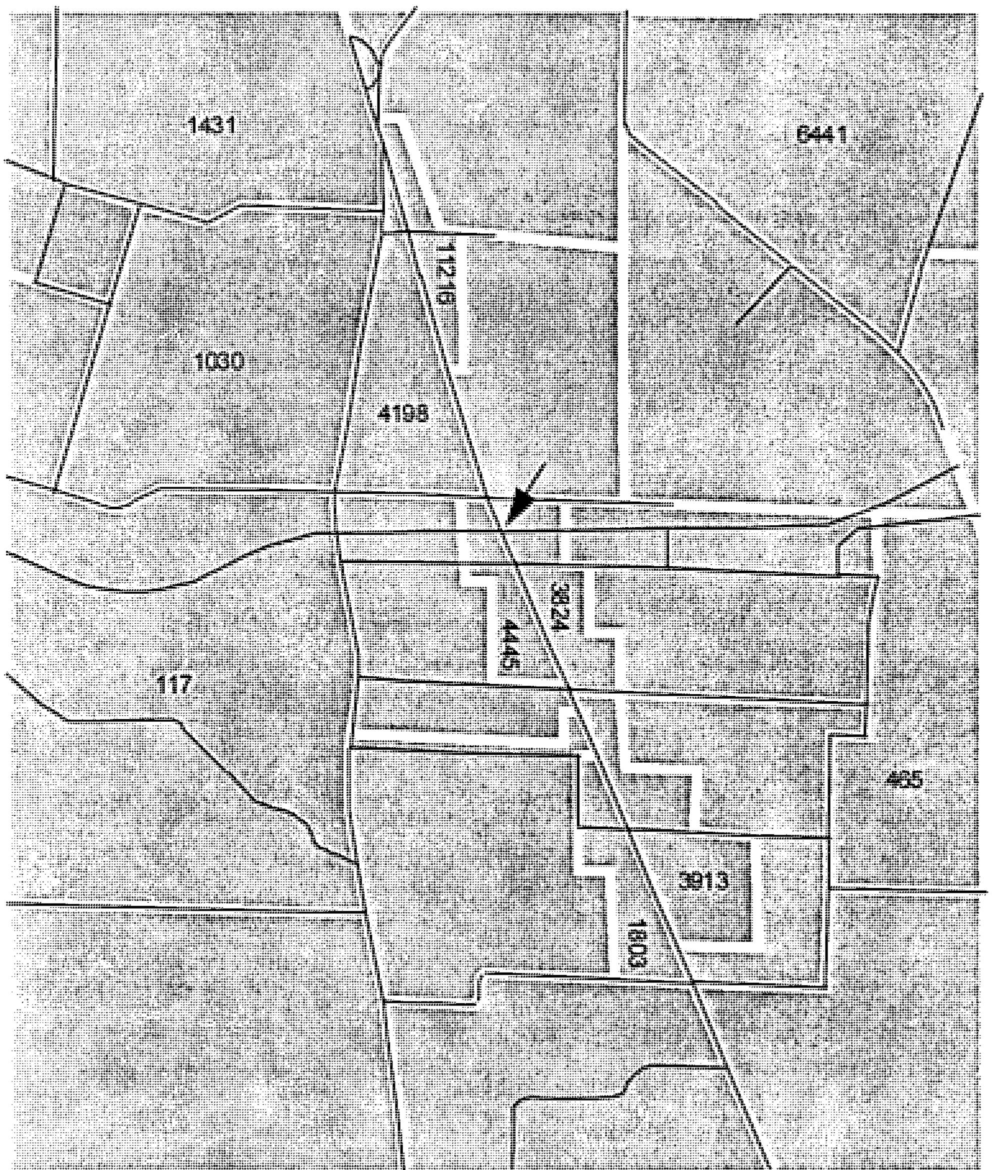
Driveway #	Distance
1	
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

TAZ DENSITY AND AVERAGE DENSITY CALCULATIONS

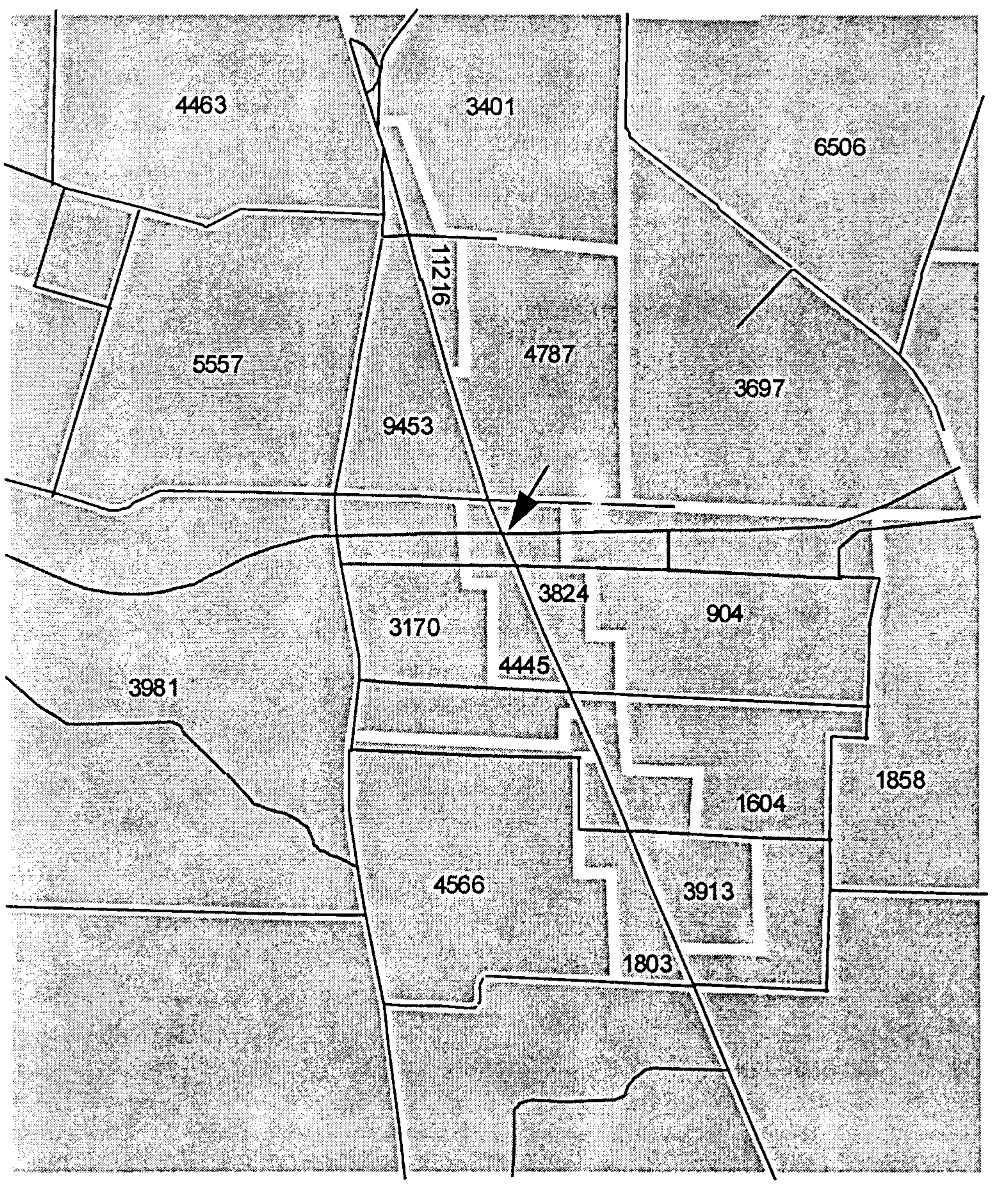
Commercial @ Kuebler
Population Density
(people per square mile)



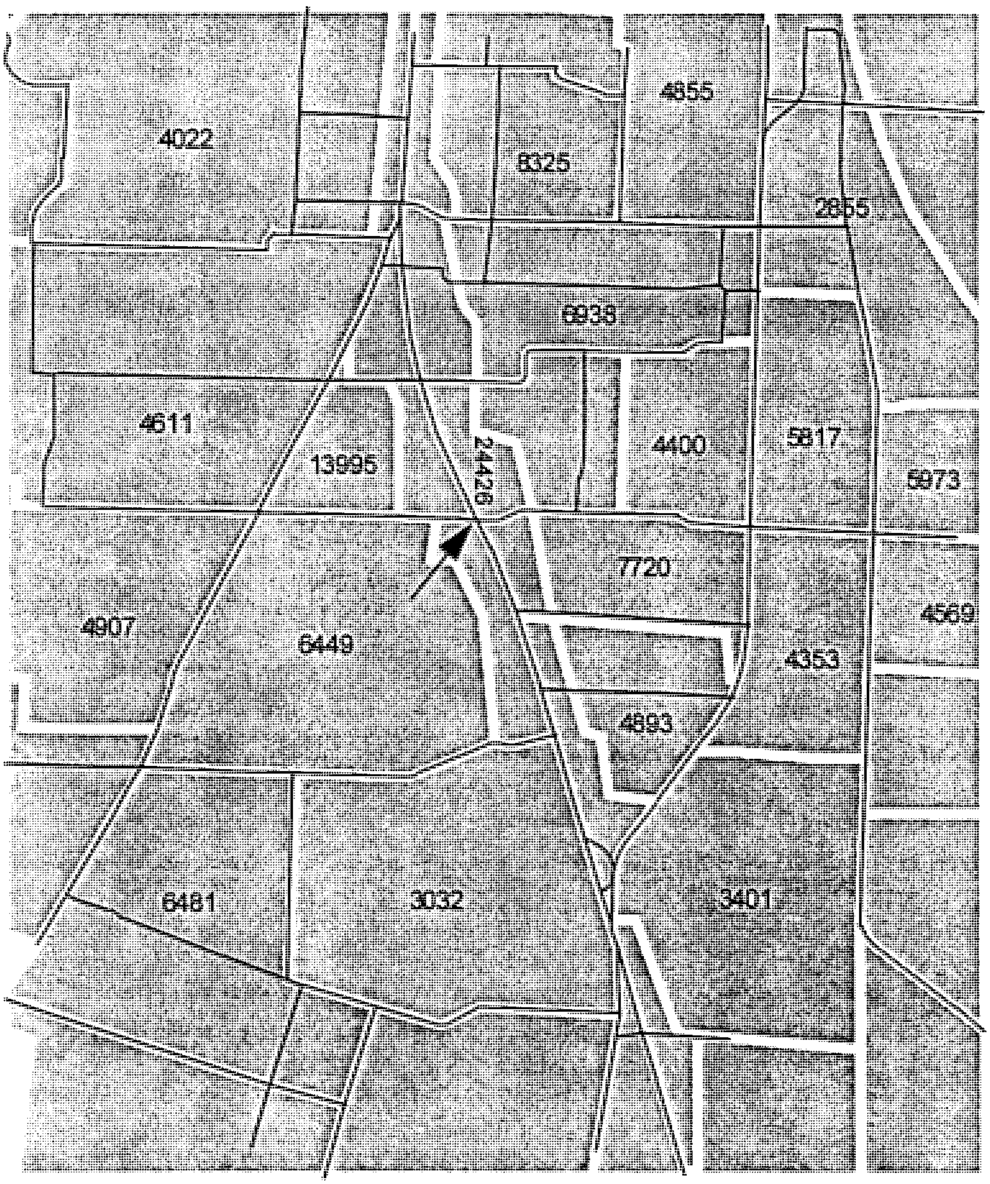
Commercial @ Kuebler
Employment Density
(employees per square mile)



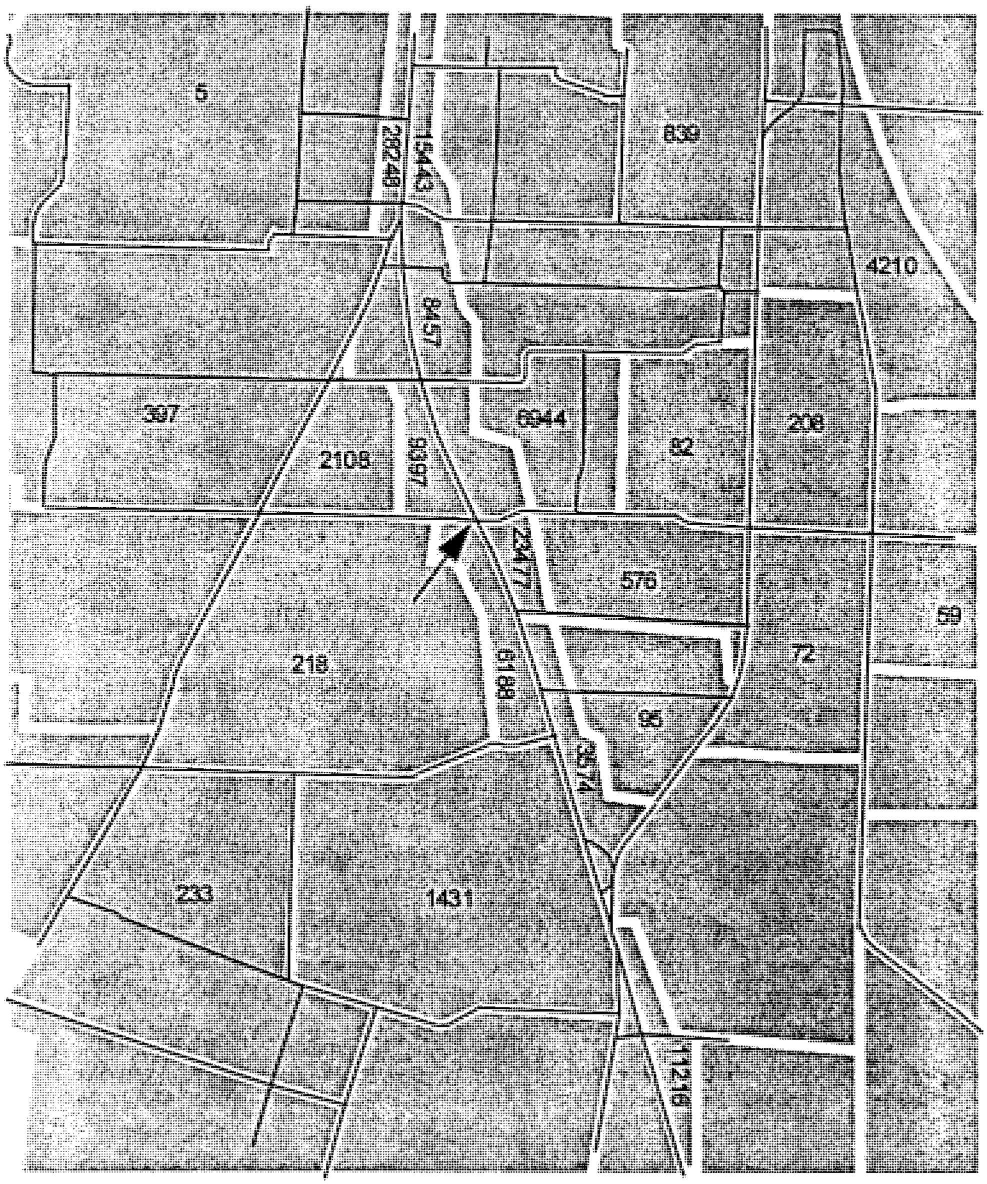
Commercial @ Kuebler
Population and Employment Density
(people and employees per square mile)



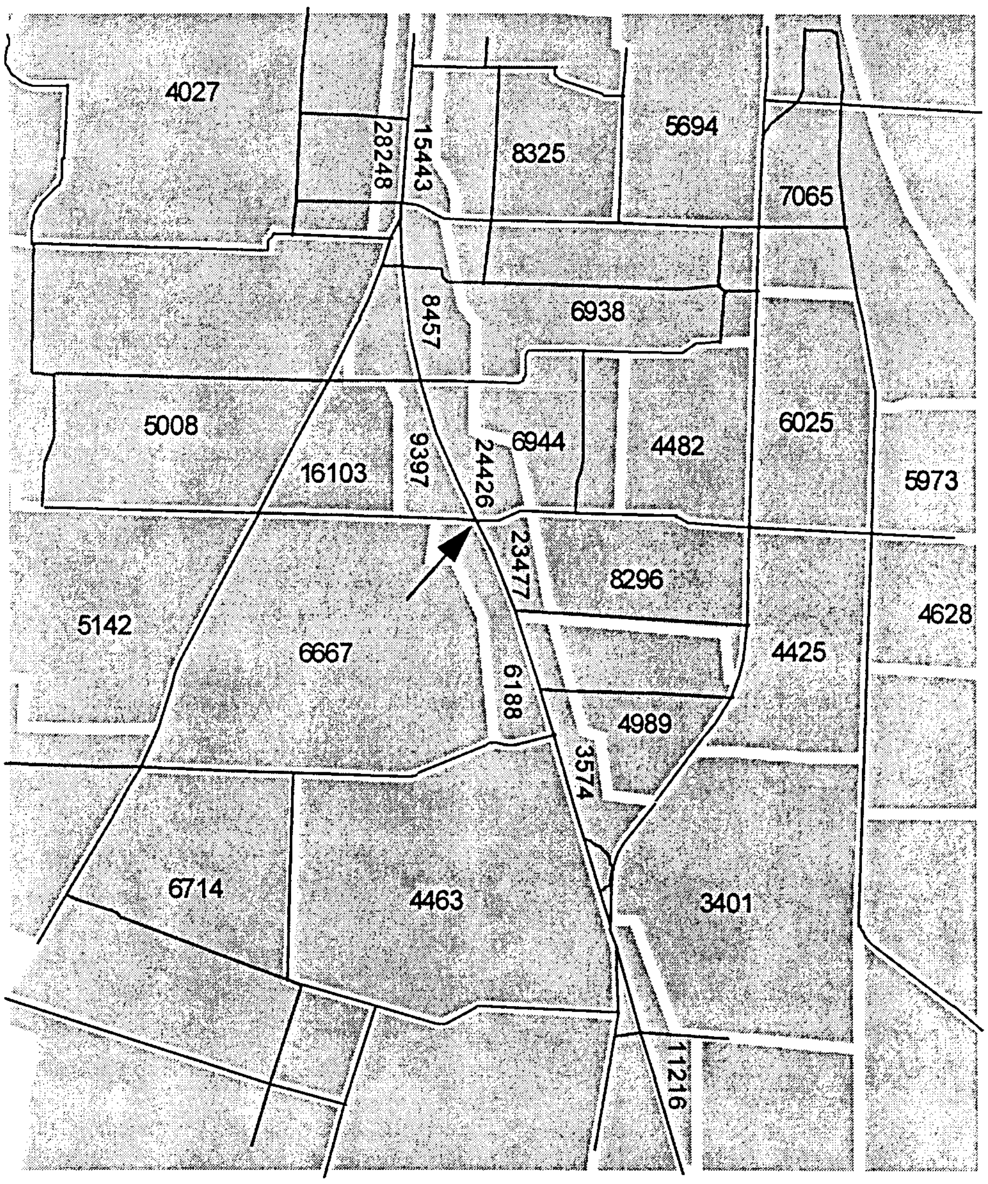
Commercial @ Madrona
Population Density
(people per square mile)



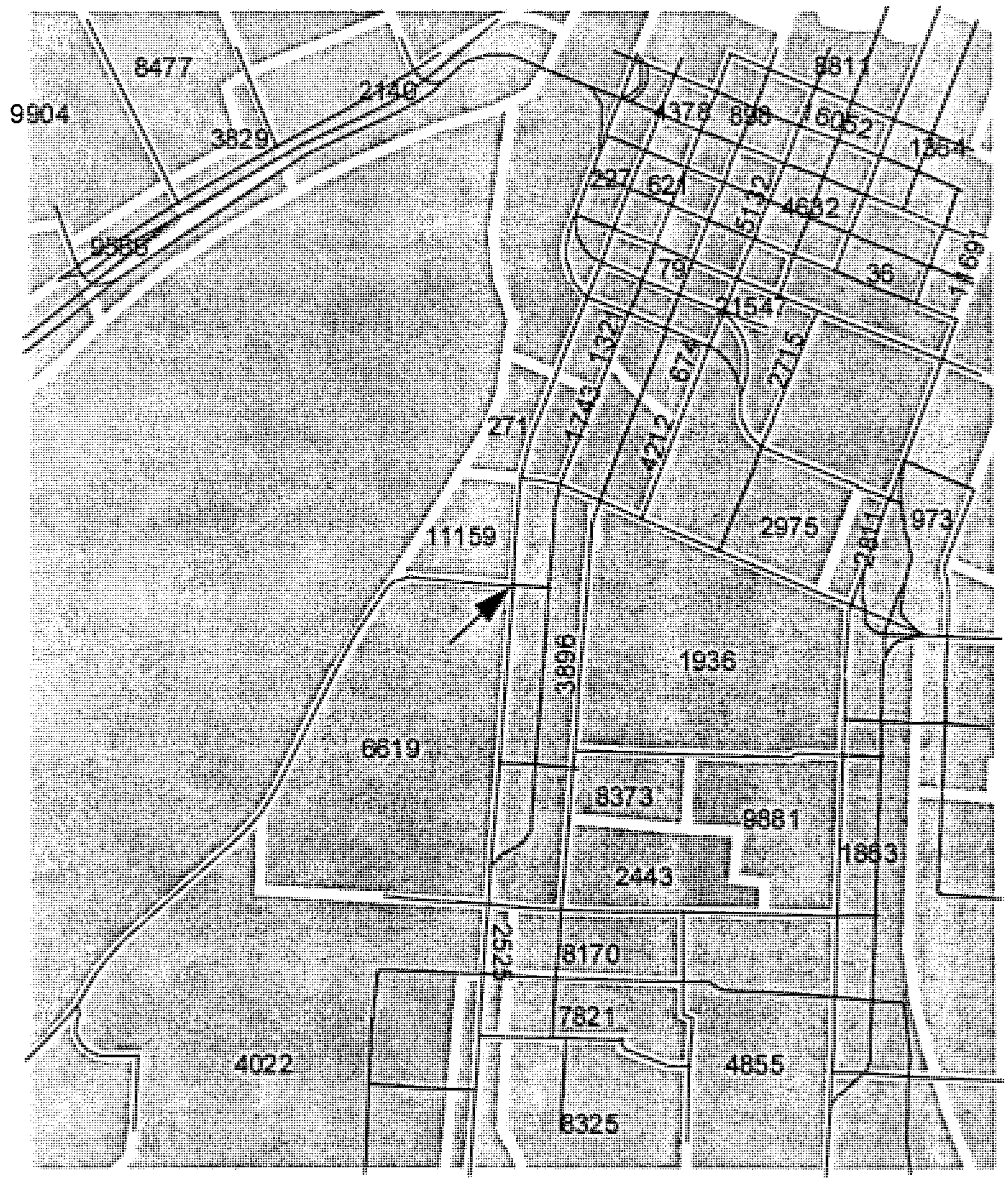
Commercial @ Madrona
Employment Density
(employees per square mile)



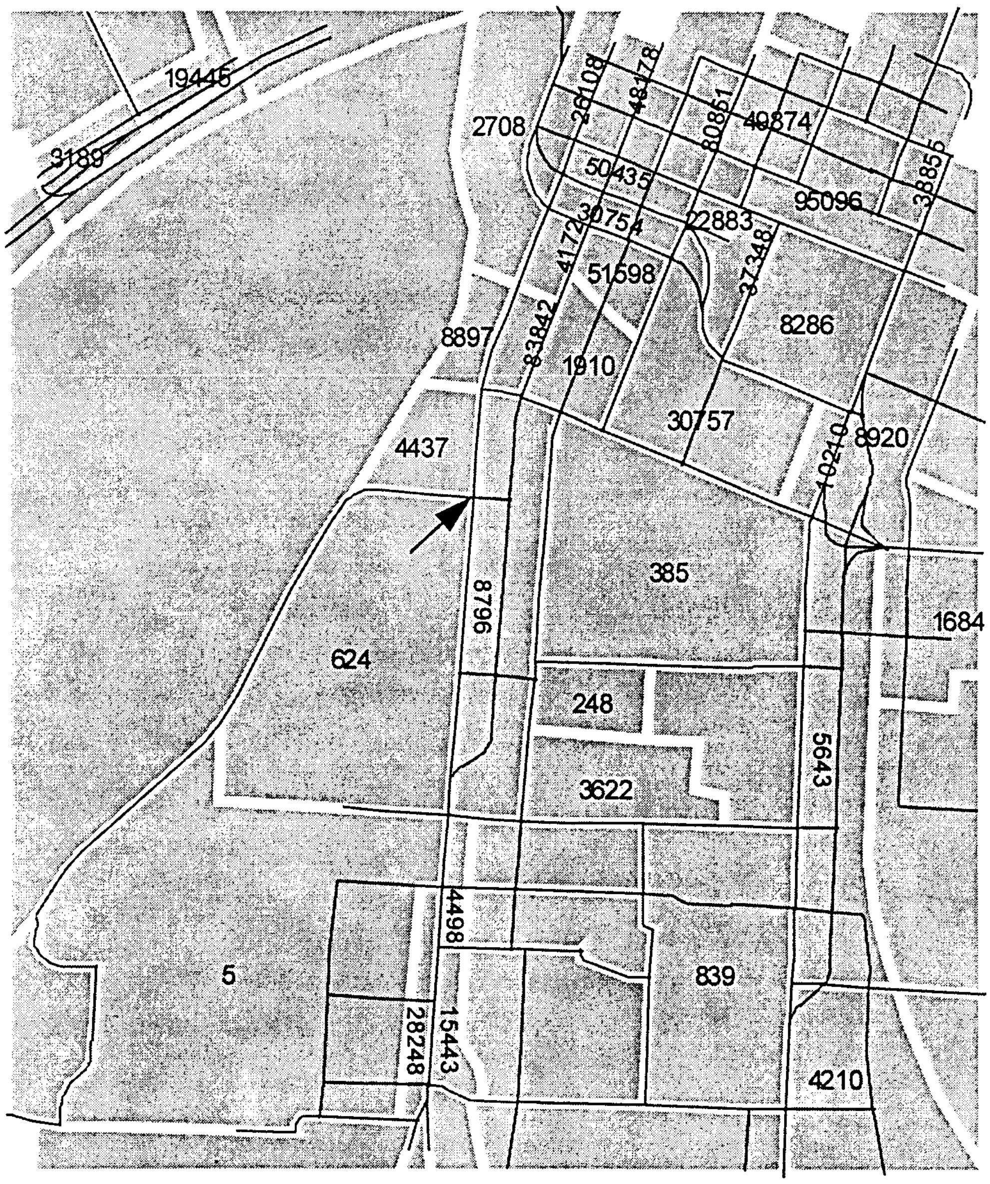
Commercial @ Madrona
Population and Employment Density
(people and employees per square mile)



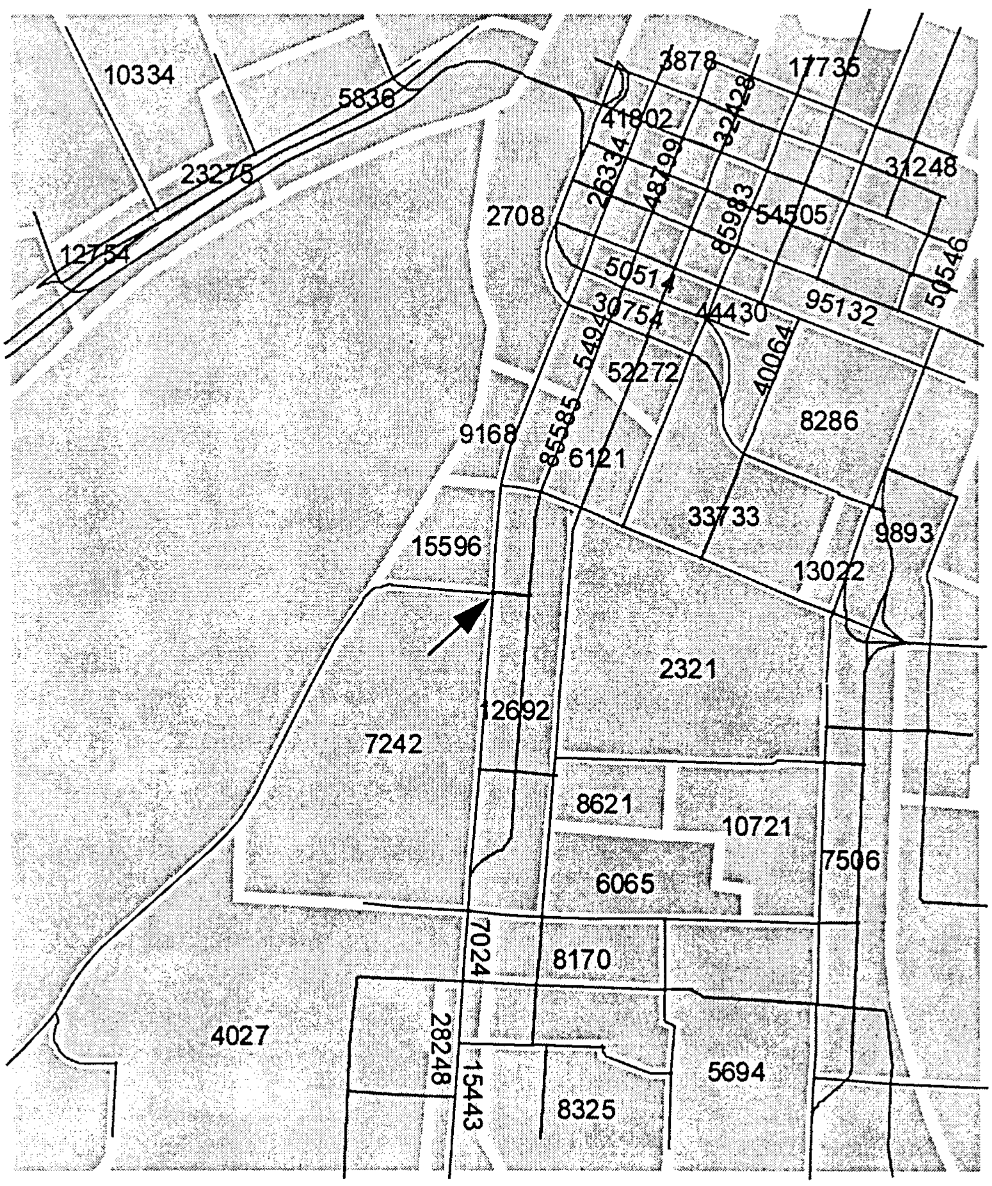
Commercial @ Owens
Population Density
(people per square mile)



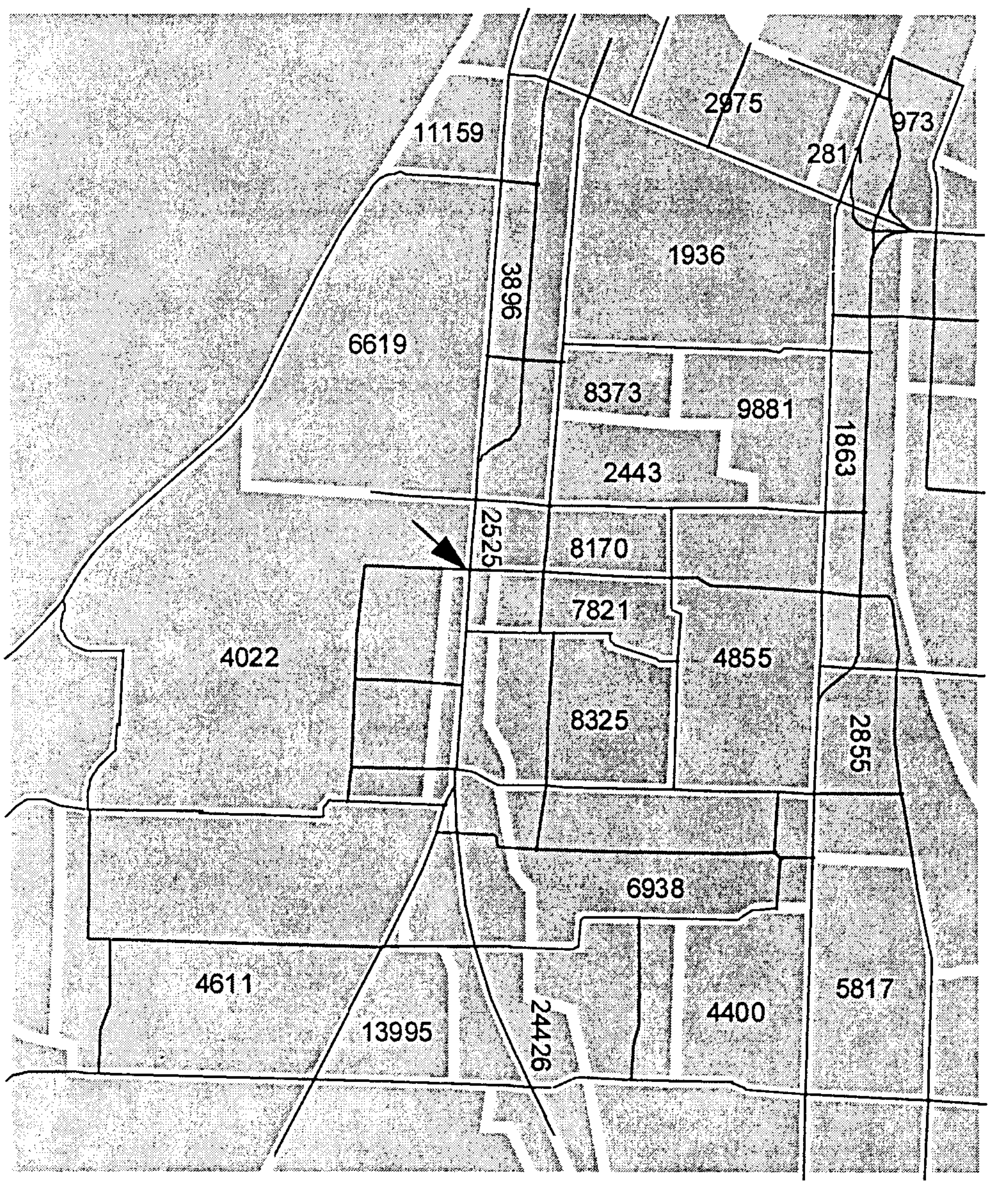
Commercial @ Owens
Employment Density
(employees per square mile)



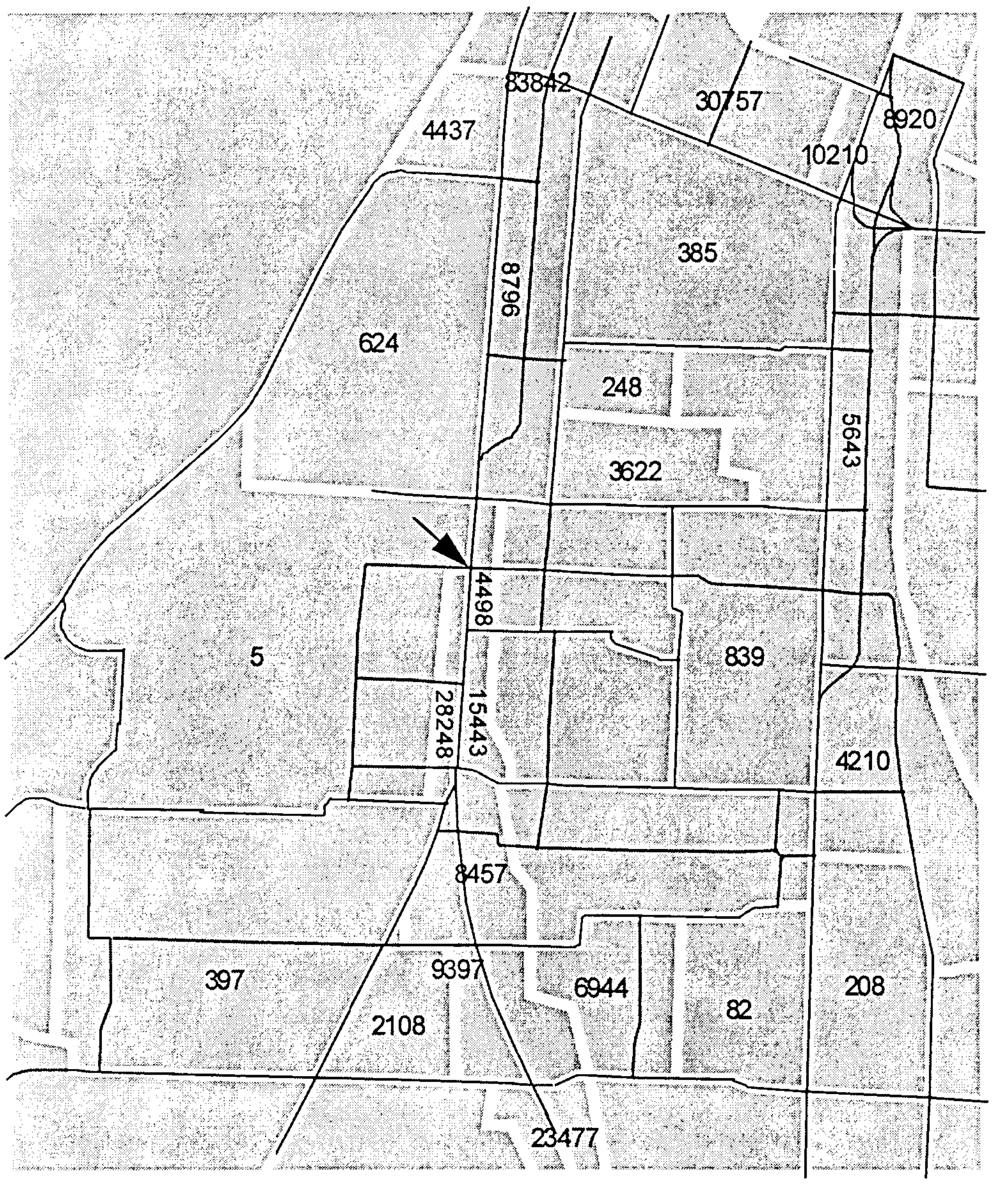
Commercial @ Owens
Population and Employment Density
(people and employees per square mile)



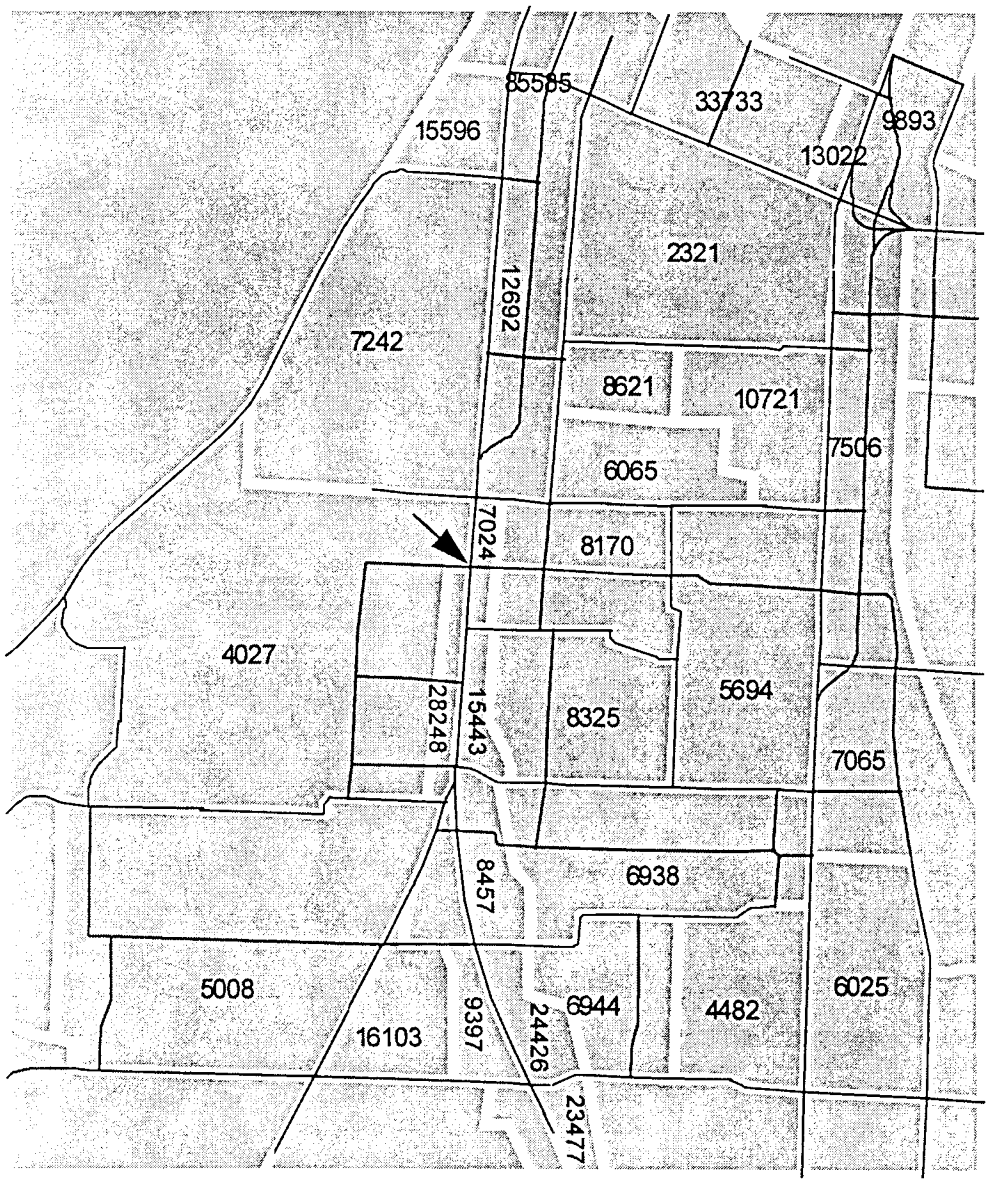
Commercial @ Hoyt
Population Density
(people per square mile)



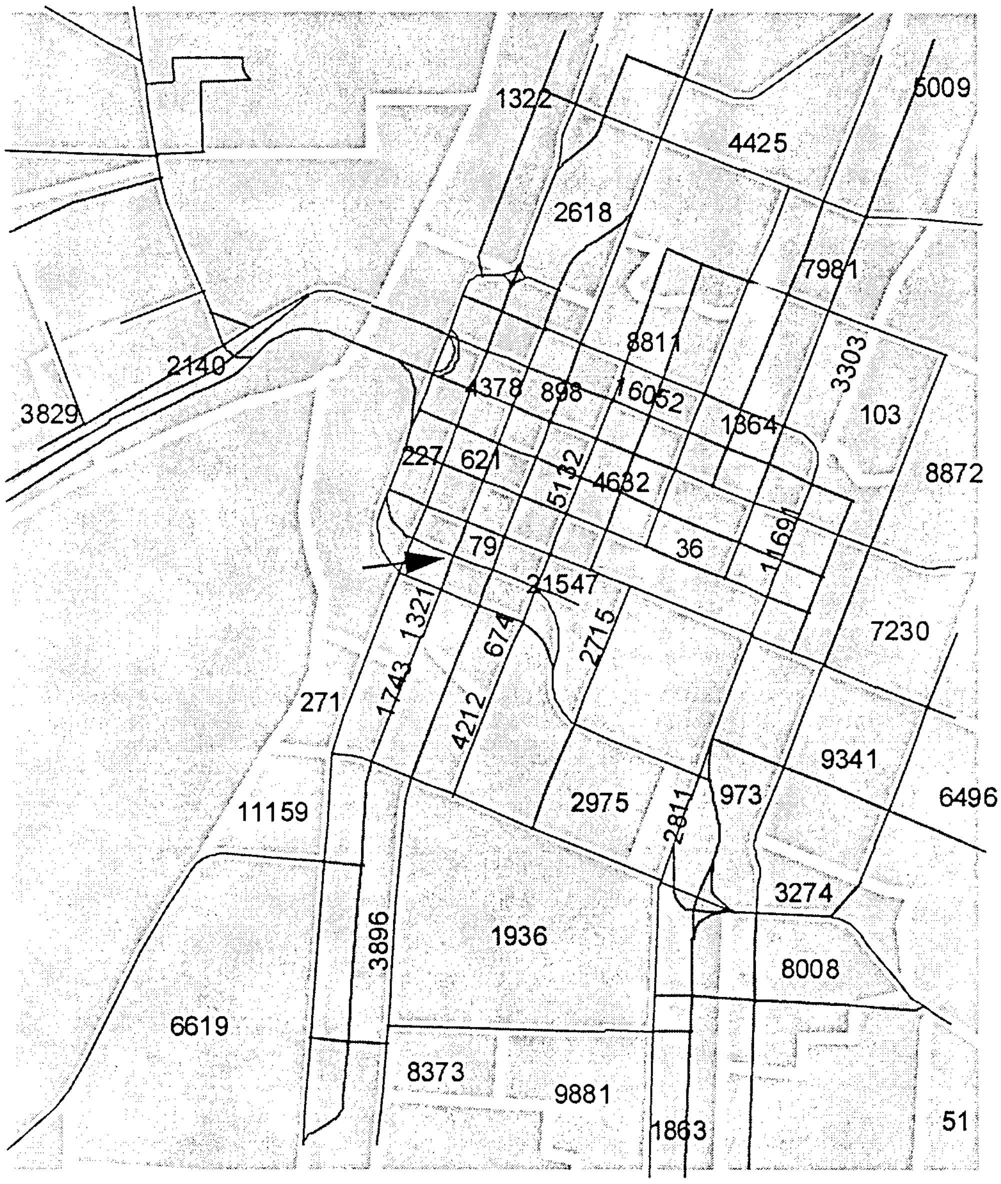
Commercial @ Hoyt
Employment Density
(employees per square mile)



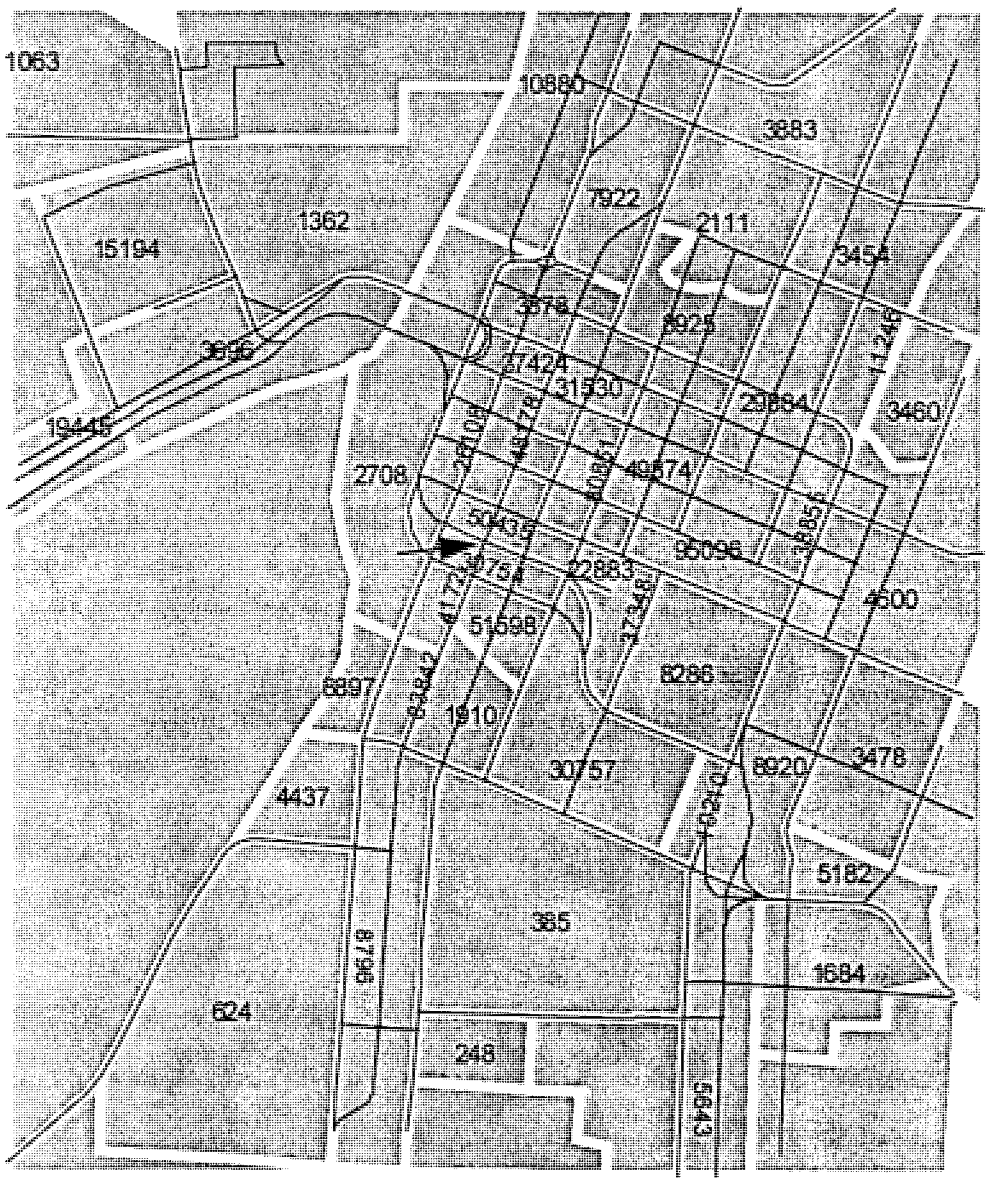
Commercial @ Hoyt
Population and Employment Density
(people and employees per square mile)



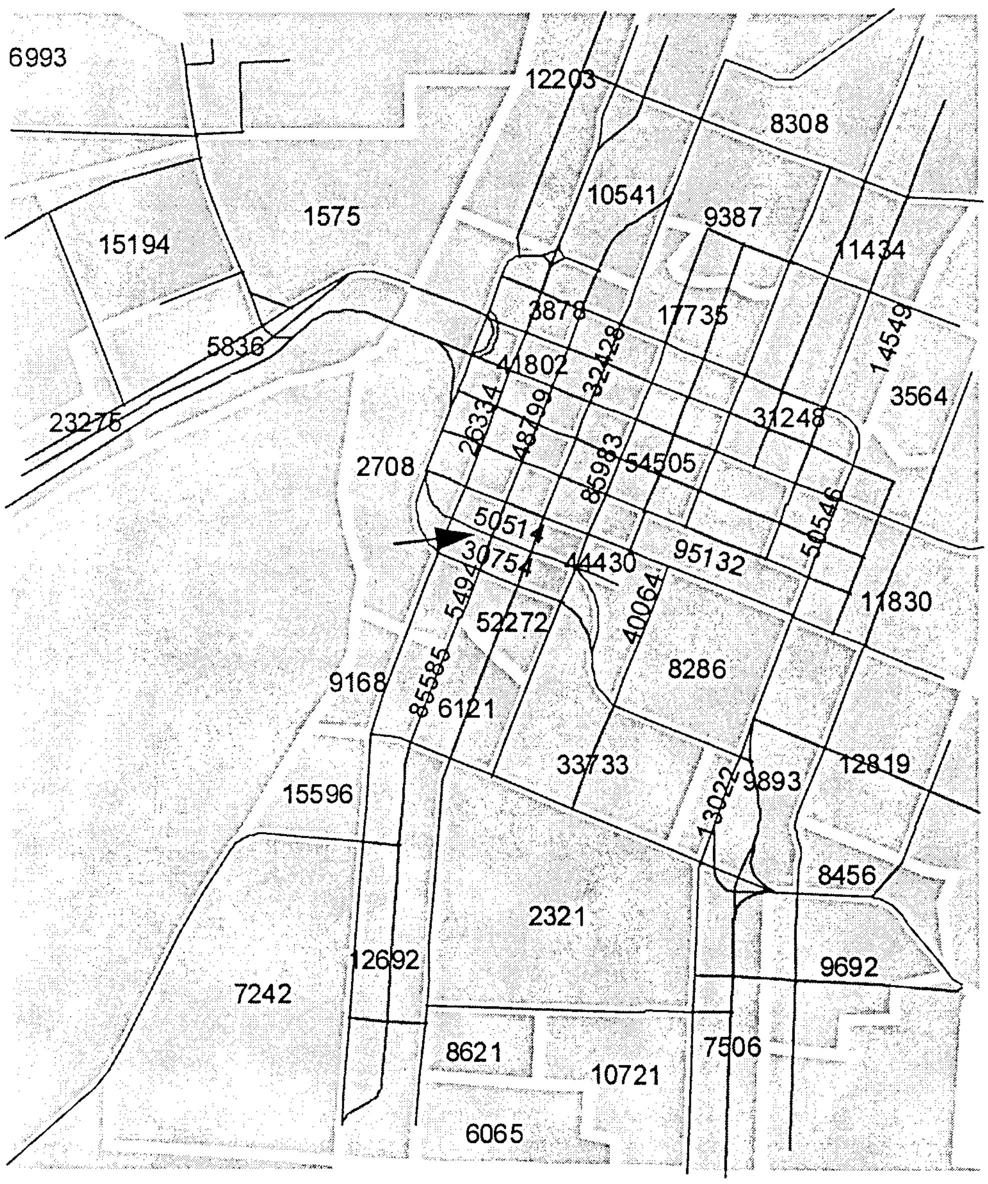
Ferry @ Liberty
Population Density
(people per square mile)



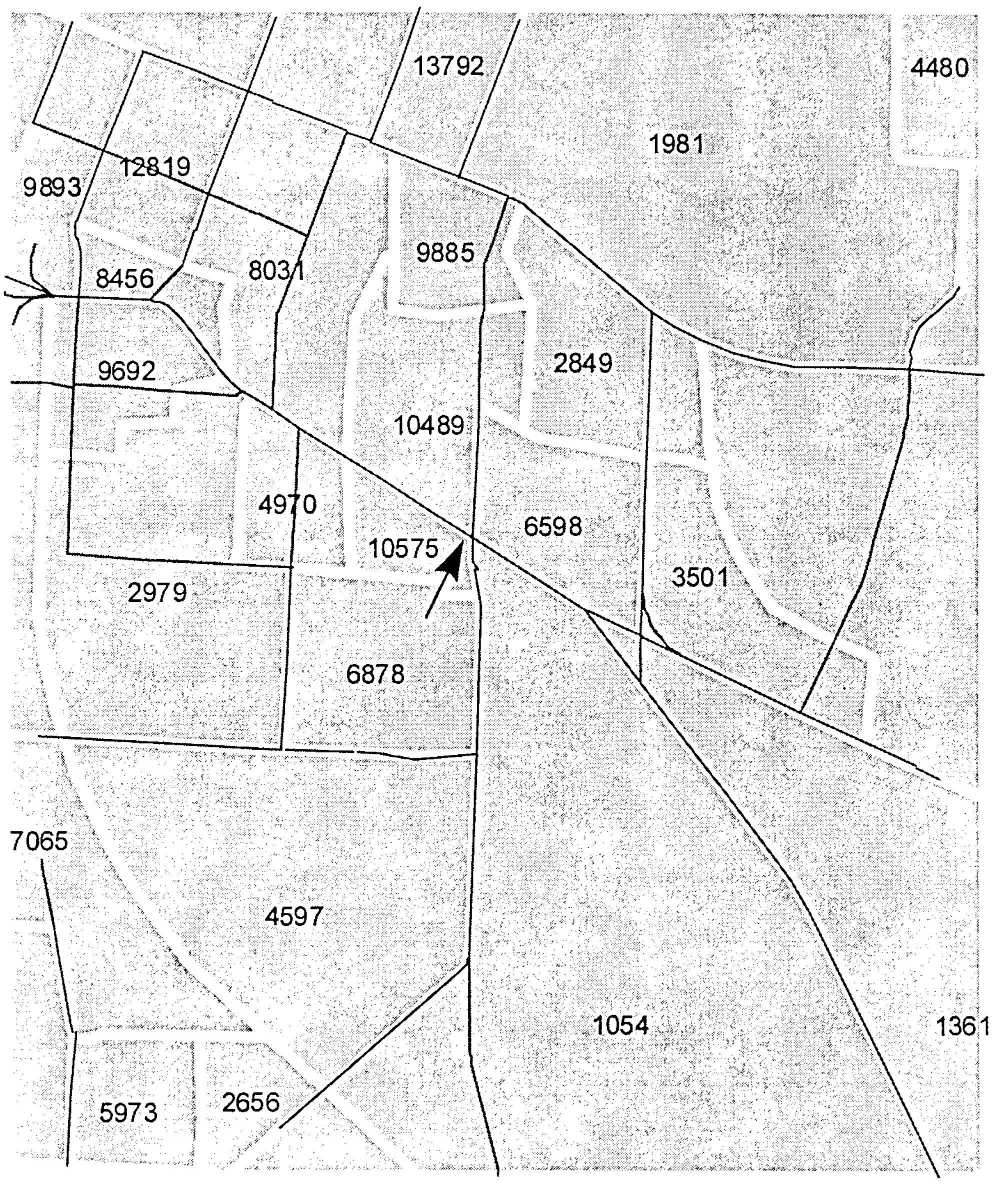
Ferry @ Liberty
Employment Density
(employees per square mile)



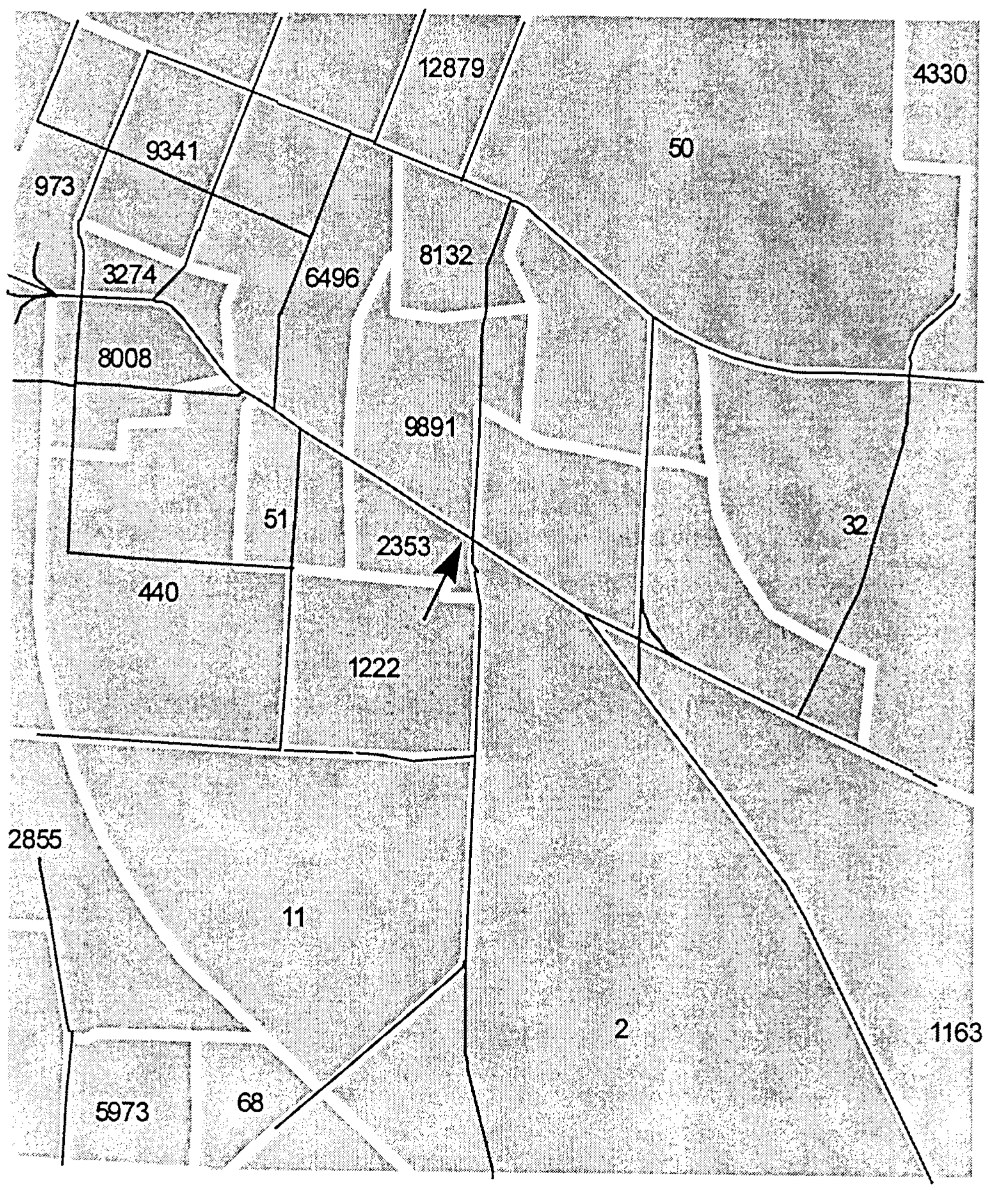
Ferry @ Liberty
Population and Employment Density
(people and employees per square mile)



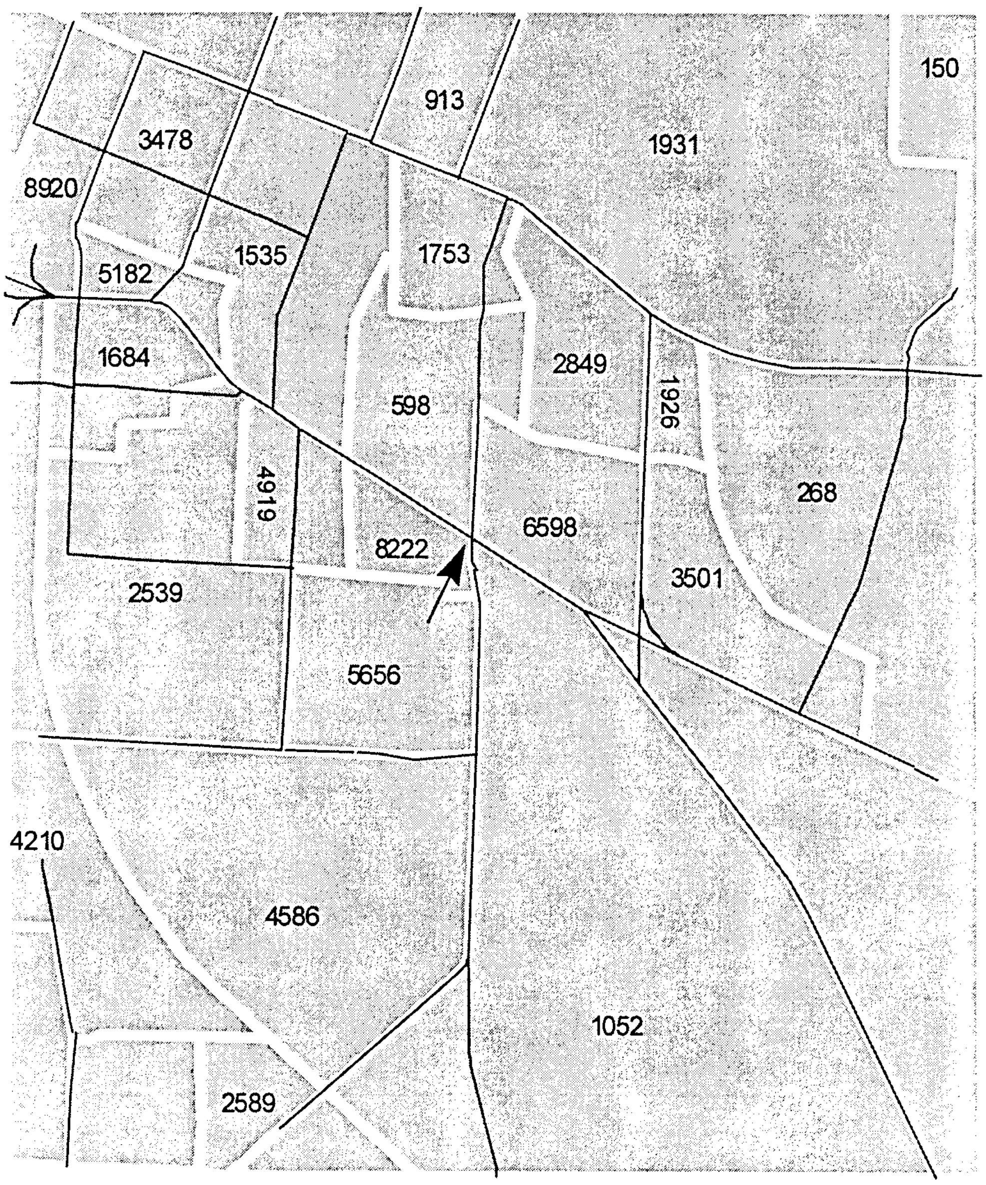
Mission @ 25th
Population and Employment Density
(people and employees per square mile)



Mission @ 25th
Population Density
(people per square mile)



Mission @ 25th
Employment Density
(employees per square mile)



Calculation of Average Densities

For each intersection, average density of the adjacent TAZ's was weighted by TAZ area.

Location 1: SB Commercial at Kuebler

		All density per sq. mile.		
TAZ	**Area (sq. mile)**	**Pop.Density**	**Emp. Density**	**Comb. Density**
NW quad.	0.09362	5255	4198	9453
NE quad.	0.15719	4787	0	4787
SW quad.	0.03397	0	4445	4445
SE quad.	0.04498	0	3824	3824
Weighted Average:		**3774**	**2171**	**5945**

Location 2: NB Commercial at Madrona

		All density per sq. mile.		
TAZ	**Area (sq. mile)**	**Pop.Density**	**Emp. Density**	**Comb. Density**
NW quad.	0.04906	0	9397	9397
NE quad.	0.02997	24426	0	24426
SW quad.	0.03960	0	6188	6188
SE quad.	0.01712	0	23477	23477
Weighted Average:		**5393**	**8162**	**13555**

Location 3: SB Commercial at Owens

		All density per sq. mile.		
TAZ	**Area (sq. mile)**	**Pop.Density**	**Emp. Density**	**Comb. Density**
NW quad.	0.03764	11159	437	11596
East Side	0.12141	3896	8796	12692
SW quad.	0.23570	6619	624	7243
Weighted Average:		**6214**	**3120**	**9334**

Location 4: NB Commercial at Hoyt

		All density per sq. mile.		
TAZ	**Area (sq. mile)**	**Pop.Density**	**Emp. Density**	**Comb. Density**
NW quad.	0.42915	4022	5	4027
East Side	0.01267	2525	4498	7023
SW quad.	0.02121	0	28248	28248
Weighted Average:		**3797**	**1422**	**5218**

Location 5: WB Ferry at Liberty

		All density per sq. mile.		
TAZ	**Area (sq. mile)**	**Pop.Density**	**Emp. Density**	**Comb. Density**
North Side	0.02528	79	50345	50424
South Side	0.01863	0	30754	30754
Weighted Average:		**45**	**42033**	**42078**

Location 4: NB Commercial at Hoyt

		All density per sq. mile.		
TAZ	**Area (sq. mile)**	**Pop.Density**	**Emp. Density**	**Comb. Density**
NW quad.	0.11536	9891	598	10489
NE quad.	0.09897	0	6598	6598
SW quad.	0.04038	2353	8222	10575
Weighted Average:		**4853**	**4138**	**8991**

Note: SE quad. not included. This TAZ is represents the airport, which has relatively low densities and a large area. Inclusion of this zone would skew the average density. The arterial approaches are adjacent to the NE and SW quads, which were included.

HCM CAPACITY CALCULATIONS

```
HCS:  Signalized Intersection    Version 2.4d             05-14-1997  1
=======================================================================
Center For Microcomputers In Transportation
University of Florida
512 Weil Hall
Gainesville, FL   32611-2083       (904) 392-0378
=======================================================================
Streets: (E-W) Kuebler                  (N-S) Commercial
Analyst: WFB                            File Name: LOC1.HC9
Area Type: Other                        5-14-97 PMPEAK
Comment: CAPACITY FOR SB THRUS
=======================================================================

                    Traffic and Roadway Conditions
-----------------------------------------------------------------------
           |   Eastbound    |   Westbound    |   Northbound   |   Southbound
           | L    T    R    | L    T    R    | L    T    R    | L    T    R
           | ---- ---- ---- | ---- ---- ---- | ---- ---- ---- | ---- ---- ----
No. Lanes  |      1         |      1         |      1         |      2  <
Volumes    |      10        |      10        |      10        |      10    2
PHF or PK15|     0.95       |     0.95       |     0.95       |     0.95 0.95
Lane W (ft)|     12.0       |     12.0       |     12.0       |     12.0
Grade      |       0        |       0        |       0        |       0
% Heavy Veh|       2        |       2        |       2        |       2    2
Parking    | (Y/N) N        | (Y/N) N        | (Y/N) N        | (Y/N) N
Bus Stops  |           0    |           0    |           0    |           0
Con. Peds  |           0    |           0    |           0    |           0
Ped Button | (Y/N) N        | (Y/N) N        | (Y/N) N        | (Y/N) N
Arr Type   |       3        |       3        |       3        |       3
RTOR Vols  |           0    |           0    |           0    |           0
Lost Time  |     3.00       |     3.00       |     3.00       |     3.00 3.00
-----------------------------------------------------------------------

                         Signal Operations
-----------------------------------------------------------------------
Phase Combination 1     2     3     4  |             5     6     7     8
EB  Left                               |NB  Left
    Thru          *                    |    Thru     *
    Right                              |    Right
    Peds                               |    Peds

WB  Left                               |SB  Left
    Thru          *                    |    Thru     *
    Right                              |    Right    *
    Peds                               |    Peds

NB  Right                              |EB  Right
SB  Right                              |WB  Right

Green          56.0P                   |Green    36.0P
Yellow/AR       4.0                    |Yellow/AR 4.0

Cycle Length: 100 secs  Phase combination order: #1 #5
```

Streets: (E-W) Kuebler (N-S) Commercial
Analyst: WFB File Name: LOC1.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR SB THRUS

Volume Adjustment Worksheet

Direc-tion/ Mvt	Mvt Vol	PHF	Adj Vol	Lane Grp	Lane Grp Vol	No. Ln	Lane Util Fact	Growth Fact	Adj Grp Vol	Prop LT	Prop RT
EB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
WB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
NB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
SB											
Thru	10	0.95	11	TR	13	2	1.050	1.000	14	0.00	0.15
Right	2	0.95	2								

Saturation Flow Adjustment Worksheet

Direction /LnGrp	Ideal Sat Flow	No. Lns	f W	f HV	f G	f p	f BB	f A	f RT	f LT	Adj Sat Flow
EB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
WB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
NB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
SB											
TR	1900	2	1.00	0.98	1.00	1.00	1.00	1.00	0.98	1.00	3640

Streets: (E-W) Kuebler (N-S) Commercial
Analyst: WFB File Name: LOC1.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR SB THRUS

Capacity Analysis Worksheet

Direction /LnGrp	Adj Flow Rate (v)	Adj Sat Flow Rate (s)	Flow Ratio (v/s)	Green Ratio (g/C)	Lane Group Capacity (c)	v/c Ratio	
EB							
T	11	1863	0.006	0.570	1062	0.010	*
WB							
T	11	1863	0.006	0.570	1062	0.010	
NB							
T	11	1863	0.006	0.370	689	0.016	*
SB							
TR	14	3640	0.004	0.370	$\boxed{1347}$	0.010	

Sum (v/s) critical = 0.012
Lost Time/Cycle, L = 6.0 sec Critical v/c(x) = 0.013

Level of Service Worksheet

Direction /LnGrp	v/c Ratio	g/C Ratio	Delay d 1	Del Adj Fact	Lane Group Cap	Calib d 2	Delay d 2	Lane Grp Del	Lane Grp LOS	Delay By App	LOS By App
EB											
T	0.010	0.570	7.1	1.000	1062	16	0.0	7.1	B	7.1	B
WB											
T	0.010	0.570	7.1	1.000	1062	16	0.0	7.1	B	7.1	B
NB											
T	0.016	0.370	15.2	1.000	689	16	0.0	15.2	C	15.2	C
SB											
TR	0.010	0.370	15.1	1.000	1347	16	0.0	15.1	C	15.1	C

Intersection Delay = 11.4 sec/veh Intersection LOS = B

```
HCS:  Signalized Intersection    Version 2.4d              05-14-1997  1
========================================================================
Center For Microcomputers In Transportation
University of Florida
512 Weil Hall
Gainesville, FL   32611-2083       (904) 392-0378
========================================================================
Streets: (E-W) Madrona                       (N-S) Commercial
Analyst: WFB                                 File Name: LOC2.HC9
Area Type: Other                             5-14-97 PMPEAK
Comment: CAPACITY FOR NB THRUS
========================================================================

                    Traffic and Roadway Conditions
------------------------------------------------------------------------
           |  Eastbound     |  Westbound     |  Northbound    |  Southbound
           | L    T    R    | L    T    R    | L    T    R    | L    T    R
           | ---- ---- ---- | ---- ---- ---- | ---- ---- ---- | ---- ---- ----
No. Lanes  |      1         |      1         |      2  <      |      1
Volumes    |       10       |       10       |       10     2 |       10
PHF or PK15|     0.95       |     0.95       |     0.95 0.95  |     0.95
Lane W (ft)|     12.0       |     12.0       |     12.0       |     12.0
Grade      |        0       |        0       |        0       |        0
% Heavy Veh|        2       |        2       |        2     2 |        2
Parking    | (Y/N) N        | (Y/N) N        | (Y/N) N        | (Y/N) N
Bus Stops  |             0  |             0  |             0  |             0
Con. Peds  |             0  |             0  |             0  |             0
Ped Button | (Y/N) N        | (Y/N) N        | (Y/N) N        | (Y/N) N
Arr Type   |        3       |        3       |        3       |        3
RTOR Vols  |             0  |             0  |             0  |             0
Lost Time  |     3.00       |     3.00       |     3.00 3.00  |     3.00
------------------------------------------------------------------------

                          Signal Operations
------------------------------------------------------------------------
Phase Combination 1     2     3     4 |              5     6     7     8
EB   Left                             |NB   Left
     Thru         *                   |     Thru     *
     Right                            |     Right    *
     Peds                             |     Peds
WB   Left                             |SB   Left
     Thru         *                   |     Thru     *
     Right                            |     Right
     Peds                             |     Peds
NB   Right                            |EB   Right
SB   Right                            |WB   Right
Green          40.0P                  |Green    52.0P
Yellow/AR       4.0                   |Yellow/AR 4.0
Cycle Length: 100 secs  Phase combination order: #1 #5
```

Streets: (E-W) Madrona (N-S) Commercial
Analyst: WFB File Name: LOC2.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR NB THRUS

Volume Adjustment Worksheet

Direction/ Mvt	Mvt Vol	PHF	Adj Vol	Lane Grp	Lane Grp Vol	No. Ln	Lane Util Fact	Growth Fact	Adj Grp Vol	Prop LT	Prop RT
EB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
WB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
NB											
Thru	10	0.95	11	TR	13	2	1.050	1.000	14	0.00	0.15
Right	2	0.95	2								
SB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00

Saturation Flow Adjustment Worksheet

Direction /LnGrp	Ideal Sat Flow	No. Lns	f W	f HV	f G	f p	f BB	f A	f RT	f LT	Adj Sat Flow
EB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
WB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
NB											
TR	1900	2	1.00	0.98	1.00	1.00	1.00	1.00	0.98	1.00	3640
SB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863

HCS: Signalized Intersection Version 2.4d 05-14-1997 3

Streets: (E-W) Madrona (N-S) Commercial
Analyst: WFB File Name: LOC2.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR NB THRUS

Capacity Analysis Worksheet

Direction /LnGrp	Adj Flow Rate (v)	Adj Sat Flow Rate (s)	Flow Ratio (v/s)	Green Ratio (g/C)	Lane Group Capacity (c)	v/c Ratio	
EB							
T	11	1863	0.006	0.410	764	0.014	*
WB							
T	11	1863	0.006	0.410	764	0.014	
NB							
TR	14	3640	0.004	0.530	$\boxed{1929}$	0.007	
SB							
T	11	1863	0.006	0.530	987	0.011	*

Sum (v/s) critical = 0.012
Lost Time/Cycle, L = 6.0 sec Critical v/c(x) = 0.013

Level of Service Worksheet

Direction /LnGrp	v/c Ratio	g/C Ratio	Delay d 1	Del Adj Fact	Lane Group Cap	Calib d 2	Delay d 2	Lane Grp Del	Lane Grp LOS	Delay By App	LOS By App
EB											
T	0.014	0.410	13.3	1.000	764	16	0.0	13.3	B	13.3	B
WB											
T	0.014	0.410	13.3	1.000	764	16	0.0	13.3	B	13.3	B
NB											
TR	0.007	0.530	8.4	1.000	1929	16	0.0	8.4	B	8.4	B
SB											
T	0.011	0.530	8.4	1.000	987	16	0.0	8.4	B	8.4	B

Intersection Delay = 10.7 sec/veh Intersection LOS = B

HCS: Signalized Intersection   Version 2.4d   05-14-1997 1

Center For Microcomputers In Transportation
University of Florida
512 Weil Hall
Gainesville, FL 32611-2083 (904) 392-0378

Streets: (E-W) Owens   (N-S) Commercial
Analyst: WFB   File Name: LOC3.HC9
Area Type: Other   5-14-97 PMPEAK
Comment: CAPACITY FOR SB THRUS

Traffic and Roadway Conditions

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
No. Lanes		1			1						> 2	
Volumes		10			10					2	10	
PHF or PK15		0.95			0.95					0.95	0.95	
Lane W (ft)		12.0			12.0						12.0	
Grade		0			0						0	
% Heavy Veh		2			2					2	2	
Parking	(Y/N)	N		(Y/N)	N					(Y/N)	N	
Bus Stops			0			0						0
Con. Peds			0			0			0			0
Ped Button	(Y/N)	N		(Y/N)	N					(Y/N)	N	
Arr Type		3			3						3	
RTOR Vols			0			0						0
Lost Time		3.00			3.00					3.00	3.00	

Signal Operations

Phase Combination	1	2	3	4		5	6	7	8
EB Left					NB Left				
Thru	*				Thru				
Right					Right				
Peds					Peds				
WB Left					SB Left	*			
Thru	*				Thru	*			
Right					Right				
Peds					Peds				
NB Right					EB Right				
SB Right					WB Right				
Green	24.0P				Green	68.0P			
Yellow/AR	4.0				Yellow/AR	4.0			

Cycle Length: 100 secs Phase combination order: #1 #5

HCS: Signalized Intersection Version 2.4d 05-14-1997 3

Streets: (E-W) Owens (N-S) Commercial
Analyst: WFB File Name: LOC3.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR SB THRUS

Capacity Analysis Worksheet

Direction /LnGrp	Adj Flow Rate (v)	Adj Sat Flow Rate (s)	Flow Ratio (v/s)	Green Ratio (g/C)	Lane Group Capacity (c)	v/c Ratio	
EB							
T	11	1863	0.006	0.250	466	0.024	*
WB							
T	11	1863	0.006	0.250	466	0.024	
NB							
SB							
LT	14	3697	0.004	0.690	$\boxed{2551}$	0.005	*

Sum (v/s) critical = 0.010
Lost Time/Cycle, L = 6.0 sec Critical v/c(x) = 0.010

Level of Service Worksheet

Direction /LnGrp	v/c Ratio	g/C Ratio	Delay d 1	Del Adj Fact	Lane Group Cap	Calib d 2	Delay d 2	Lane Grp Del	Lane Grp LOS	Delay By App	LOS By App
EB											
T	0.024	0.250	21.5	1.000	466	16	0.0	21.5	C	21.5	C
WB											
T	0.024	0.250	21.5	1.000	466	16	0.0	21.5	C	21.5	C
NB											
SB											
LT	0.005	0.690	3.7	1.000	2551	16	0.0	3.7	A	3.7	A

Intersection Delay = 14.6 sec/veh Intersection LOS = B

HCS: Signalized Intersection Version 2.4d 05-14-1997 1

Center For Microcomputers In Transportation
University of Florida
512 Weil Hall
Gainesville, FL 32611-2083 (904) 392-0378

Streets: (E-W) Hoyt (N-S) Commercial
Analyst: WFB File Name: LOC4.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR NB THRUS

Traffic and Roadway Conditions

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
No. Lanes		1			1			2	<		1	
Volumes		10			10			10	2		10	
PHF or PK15		0.95			0.95			0.95	0.95		0.95	
Lane W (ft)		12.0			12.0			12.0			12.0	
Grade		0			0			0			0	
% Heavy Veh		2			2			2	2		2	
Parking	(Y/N)	N		(Y/N)	N		(Y/N)	N		(Y/N)	N	
Bus Stops			0			0			0			0
Con. Peds			0			0			0			0
Ped Button	(Y/N)	N		(Y/N)	N		(Y/N)	N		(Y/N)	N	
Arr Type		3			3			3			3	
RTOR Vols			0			0			0			0
Lost Time		3.00			3.00			3.00	3.00		3.00	

Signal Operations

Phase Combination	1	2	3	4		5	6	7	8
EB Left					NB Left				
Thru	*				Thru	*			
Right					Right	*			
Peds					Peds				
WB Left					SB Left				
Thru	*				Thru	*			
Right					Right				
Peds					Peds				
NB Right					EB Right				
SB Right					WB Right				
Green	16.0P				Green	76.0P			
Yellow/AR	4.0				Yellow/AR	4.0			

Cycle Length: 100 secs Phase combination order: #1 #5

HCS: Signalized Intersection Version 2.4d 05-14-1997 2

Streets: (E-W) Hoyt (N-S) Commercial
Analyst: WFB File Name: LOC4.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR NB THRUS

Volume Adjustment Worksheet

Direction/ Mvt	Mvt Vol	PHF	Adj Vol	Lane Grp	Lane Grp Vol	No. Ln	Lane Util Fact	Growth Fact	Adj Grp Vol	Prop LT	Prop RT
EB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
WB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
NB											
Thru	10	0.95	11	TR	13	2	1.050	1.000	14	0.00	0.15
Right	2	0.95	2								
SB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00

Saturation Flow Adjustment Worksheet

Direction /LnGrp	Ideal Sat Flow	No. Lns	f_W	f_{HV}	f_G	f_p	f_{BB}	f_A	f_{RT}	f_{LT}	Adj Sat Flow
EB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
WB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
NB											
TR	1900	2	1.00	0.98	1.00	1.00	1.00	1.00	0.98	1.00	3640
SB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863

Streets: (E-W) Hoyt   (N-S) Commercial
Analyst: WFB   File Name: LOC4.HC9
Area Type: Other   5-14-97 PMPEAK
Comment: CAPACITY FOR NB THRUS

Capacity Analysis Worksheet

Direction /LnGrp	Adj Flow Rate (v)	Adj Sat Flow Rate (s)	Flow Ratio (v/s)	Green Ratio (g/C)	Lane Group Capacity (c)	v/c Ratio	
EB							
T	11	1863	0.006	0.170	317	0.035	*
WB							
T	11	1863	0.006	0.170	317	0.035	
NB							
TR	14	3640	0.004	0.770	$\boxed{2803}$	0.005	
SB							
T	11	1863	0.006	0.770	1435	0.008	*

Sum (v/s) critical = 0.012

Lost Time/Cycle, L = 6.0 sec   Critical v/c(x) = 0.013

Level of Service Worksheet

Direction /LnGrp	v/c Ratio	g/C Ratio	Delay d 1	Del Adj Fact	Lane Group Cap	Calib d 2	Delay d 2	Lane Grp Del	Lane Grp LOS	Delay By App	LOS By App
EB											
T	0.035	0.170	26.3	1.000	317	16	0.0	26.3	D	26.3	D
WB											
T	0.035	0.170	26.3	1.000	317	16	0.0	26.3	D	26.3	D
NB											
TR	0.005	0.770	2.0	1.000	2803	16	0.0	2.0	A	2.0	A
SB											
T	0.008	0.770	2.0	1.000	1435	16	0.0	2.0	A	2.0	A

Intersection Delay = 13.4 sec/veh Intersection LOS = B

HCS: Signalized Intersection Version 2.4d 05-14-1997 1

Center For Microcomputers In Transportation
University of Florida
512 Weil Hall
Gainesville, FL 32611-2083 (904) 392-0378

Streets: (E-W) Ferry (N-S) Liberty
Analyst: WFB File Name: LOC5.HC9
Area Type: CBD 5-14-97 PMPEAK
Comment: CAPACITY FOR WB THRUS

Traffic and Roadway Conditions

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
No. Lanes					2	<		1				
Volumes					10	2		10				
PHF or PK15					0.95	0.95		0.95				
Lane W (ft)					12.0			12.0				
Grade					0			0				
% Heavy Veh					2	2		2				
Parking				(Y/N)	N		(Y/N)	N				
Bus Stops						0			0			
Con. Peds			0			0			0			0
Ped Button				(Y/N)	N		(Y/N)	N				
Arr Type					3			3				
RTOR Vols						0			0			
Lost Time					3.00	3.00		3.00				

Signal Operations

Phase Combination		1	2	3	4			5	6	7	8
EB	Left					NB	Left				
	Thru						Thru	*			
	Right						Right				
	Peds						Peds				
WB	Left					SB	Left				
	Thru	*					Thru				
	Right	*					Right				
	Peds						Peds				
NB	Right					EB	Right				
SB	Right					WB	Right				
Green		45.0P				Green		47.0P			
Yellow/AR		4.0				Yellow/AR		4.0			

Cycle Length: 100 secs Phase combination order: #1 #5

Streets: (E-W) Ferry (N-S) Liberty
Analyst: WFB File Name: LOC5.HC9
Area Type: CBD 5-14-97 PMPEAK
Comment: CAPACITY FOR WB THRUS

Volume Adjustment Worksheet

Direction/ Mvt	Mvt Vol	PHF	Adj Vol	Lane Grp	Lane Grp Vol	No. Ln	Lane Util Fact	Growth Fact	Adj Grp Vol	Prop LT	Prop RT
WB											
Thru	10	0.95	11	TR	13	2	1.050	1.000	14	0.00	0.15
Right	2	0.95	2								
NB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00

Saturation Flow Adjustment Worksheet

Direction /LnGrp	Ideal Sat Flow	No. Lns	f W	f HV	f G	f p	f BB	f A	f RT	f LT	Adj Sat Flow
WB											
TR	1900	2	1.00	0.98	1.00	1.00	1.00	0.90	0.98	1.00	3276
NB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	0.90	1.00	1.00	1676

HCS: Signalized Intersection Version 2.4d 05-14-1997 3

Streets: (E-W) Ferry (N-S) Liberty
Analyst: WFB File Name: LOC5.HC9
Area Type: CBD 5-14-97 PMPEAK
Comment: CAPACITY FOR WB THRUS

Capacity Analysis Worksheet

Direction /LnGrp	Adj Flow Rate (v)	Adj Sat Flow Rate (s)	Flow Ratio (v/s)	Green Ratio (g/C)	Lane Group Capacity (c)	v/c Ratio
EB						
WB						
TR	14	3276	0.004	0.460	1507	0.009 *
NB						
T	11	1676	0.007	0.480	804	0.014 *
SB						

Sum (v/s) critical = 0.011
Lost Time/Cycle, L = 6.0 sec Critical v/c(x) = 0.012

Level of Service Worksheet

Direction /LnGrp	v/c Ratio	g/C Ratio	Delay d 1	Del Adj Fact	Lane Group Cap	Calib d 2	Delay d 2	Lane Grp Del	Lane Grp LOS	Delay By App	LOS By App
EB											
WB											
TR	0.009	0.460	11.1	1.000	1507	16	0.0	11.1	B	11.1	B
NB											
T	0.014	0.480	10.3	1.000	804	16	0.0	10.3	B	10.3	B
SB											

Intersection Delay = 10.8 sec/veh Intersection LOS = B

HCS: Signalized Intersection Version 2.4d 05-14-1997 1

Center For Microcomputers In Transportation
University of Florida
512 Weil Hall
Gainesville, FL 32611-2083 (904) 392-0378

Streets: (E-W) Mission (N-S) 25th
Analyst: WFB File Name: LOC6.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR EB THRUS

Traffic and Roadway Conditions

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
No. Lanes		2	<		1			1			1	
Volumes		10	2		10			10			10	
PHF or PK15		0.95	0.95		0.95			0.95			0.95	
Lane W (ft)		12.0			12.0			12.0			12.0	
Grade		0			0			0			0	
% Heavy Veh		2	2		2			2			2	
Parking	(Y/N)	N		(Y/N)	N		(Y/N)	N		(Y/N)	N	
Bus Stops			0			0			0			0
Con. Peds			0			0			0			0
Ped Button	(Y/N)	N		(Y/N)	N		(Y/N)	N		(Y/N)	N	
Arr Type		3			3			3			3	
RTOR Vols			0			0			0			0
Lost Time		3.00	3.00		3.00			3.00			3.00	

Signal Operations

Phase Combination		1	2	3	4
EB	Left				
	Thru	*			
	Right	*			
	Peds				
WB	Left				
	Thru	*			
	Right				
	Peds				
NB	Right				
SB	Right				
Green		46.0P			
Yellow/AR		4.0			

		5	6	7	8
NB	Left				
	Thru	*			
	Right				
	Peds				
SB	Left				
	Thru	*			
	Right				
	Peds				
EB	Right				
WB	Right				
Green		46.0P			
Yellow/AR		4.0			

Cycle Length: 100 secs Phase combination order: #1 #5

Streets: (E-W) Mission (N-S) 25th
Analyst: WFB File Name: LOC6.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR EB THRUS

Volume Adjustment Worksheet

Direc-tion/ Mvt	Mvt Vol	PHF	Adj Vol	Lane Grp	Lane Grp Vol	No. Ln	Lane Util Fact	Growth Fact	Adj Grp Vol	Prop LT	Prop RT
EB											
Thru	10	0.95	11	TR	13	2	1.050	1.000	14	0.00	0.15
Right	2	0.95	2								
WB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
NB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00
SB											
Thru	10	0.95	11	T	11	1	1.000	1.000	11	0.00	0.00

Saturation Flow Adjustment Worksheet

Direction /LnGrp	Ideal Sat Flow	No. Lns	f W	f HV	f G	f p	f BB	f A	f RT	f LT	Adj Sat Flow
EB											
TR	1900	2	1.00	0.98	1.00	1.00	1.00	1.00	0.98	1.00	3640
WB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
NB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863
SB											
T	1900	1	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00	1863

Streets: (E-W) Mission (N-S) 25th
Analyst: WFB File Name: LOC6.HC9
Area Type: Other 5-14-97 PMPEAK
Comment: CAPACITY FOR EB THRUS

Capacity Analysis Worksheet

Direction /LnGrp	Adj Flow Rate (v)	Adj Sat Flow Rate (s)	Flow Ratio (v/s)	Green Ratio (g/C)	Lane Group Capacity (c)	v/c Ratio
EB						
TR	14	3640	0.004	0.470	$\boxed{1711}$	0.008
WB						
T	11	1863	0.006	0.470	876	0.013 *
NB						
T	11	1863	0.006	0.470	876	0.013 *
SB						
T	11	1863	0.006	0.470	876	0.013

Sum (v/s) critical = 0.012
Lost Time/Cycle, L = 6.0 sec Critical v/c(x) = 0.013

Level of Service Worksheet

Direction /LnGrp	v/c Ratio	g/C Ratio	Delay d 1	Del Adj Fact	Lane Group Cap	Calib d 2	Delay d 2	Lane Grp Del	Lane Grp LOS	Delay By App	LOS By App
EB											
TR	0.008	0.470	10.7	1.000	1711	16	0.0	10.7	B	10.7	B
WB											
T	0.013	0.470	10.7	1.000	876	16	0.0	10.7	B	10.7	B
NB											
T	0.013	0.470	10.7	1.000	876	16	0.0	10.7	B	10.7	B
SB											
T	0.013	0.470	10.7	1.000	876	16	0.0	10.7	B	10.7	B

Intersection Delay = 10.7 sec/veh Intersection LOS = B

Modeling Performance Indicators on Two-Lane Rural Highways:

The Oregon Experience

Facility Analysis and Simulation Team
Transportation Development Division
Oregon Department of Transportation

December 2010

Table of Contents

Table of Figures

Table of Tables

1 Introduction

Two-lane rural roads exhibit high level of interactions between vehicles traveling in the same and opposite directions. Traffic volume in both directions, configuration of highway geometry, terrain, grades, and presence of heavy vehicles intensifies this interaction. Drivers look for opportunities to pass slower vehicles in order to maintain free flow speeds. Limited passing opportunities may lead to an increase in crash rates as evidenced from crash reports. Also, higher interactions between vehicles forms platoons. In order to study the operational characteristics of two-lane highways, one needs to analyze platoons. The following section briefly describes some common approaches to studying two-lane rural highways

1.1 Literature Review

Several studies have proposed or reported on the use of performance measures on two-lane highways, including those used by the HCM.

1.1.1 Highway Capacity Manual (HCM) method

The 2000 Highway Capacity Manual (HCM) evaluates two-lane highway performance using both average travel speed (ATS) and percent time spent following (PTSF) as performance indicators. The PTSF[1] is defined as "the average percentage of travel time that vehicles must travel in platoons behind slower vehicles because of an inability to pass". But, PTSF is very difficult to measure in the field. The HCM recommends use of a surrogate measure, percent followers, defined as the percentage of vehicles in the traffic stream with time headways smaller than 3 sec.

1.1.2 Luttinen[2]

Luttinen reported a study by Normann[3] who suggested the following performance measures on two-lane highways:

- Proportion of headways less than 9 s,
- Ratio of actual passings to desired passings,
- Average number of passings per vehicle, and
- Speed differences between successive vehicles.

---

[1] Highway Capacity Manual. TRB, National Research Council, Washington, D.C., 2000.

[2] Luttinen, R. T. Percent Time-Spent-Following as Performance Measure for Two-Lane Highways. In Transportation Research Record: Journal of the Transportation Research Board, No. 1776, TRB, National Research Council, Washington, D.C., 2001, pp. 52–59.

[3] Normann, O. K. Results of Highway Capacity Studies. Public Roads,Vol. 23, No. 4, June 1942, pp. 57–81.

1.1.3 *Morrall and Werner*[4]

Morrall and Werner proposed the use of overtaking ratio, which is obtained by dividing the number of passings achieved by the number of passings desired, as a supplementary indicator of LOS on two-lane highways. According to the study, the number of passings achieved is the total number of passings for a given two-lane highway, and the number of passings desired is the total number of passings for a two-lane highway with continuous passing lanes and similar vertical and horizontal geometry.

1.1.4 *Brilon and Weiser*[5]

Brilon and Weiser reported the use of average speed of passenger cars over a longer stretch of highway, averaged over both directions, as a major performance measure on two-lane highways.

1.1.5 *Christo van As*[6]

A South African research project was undertaken to investigate the use of other measures of performance on two-lane highways as part of developing new analytical procedures and a simulation model for two-lane highways, found follower density (number of followers per kilometer) a promising measure of performance on two-lane highways. Among other performance measures considered by the same project are follower flow (followers per hour), percent followers, percent speed reduction due to traffic, total queuing delay, and traffic density.

1.1.6 *Romana and Pérez*[7]

This study suggested a "new LOS scheme" on two-lane highways using the current HCM performance measures, such as average travel speed and percent time spent following.

---

[4]Morrall, J. F., and A. Werner. Measuring Level of Service of Two-Lane Highways by Overtakings. In Transportation Research Record 1287, TRB, National Research Council, Washington D.C., 1990, pp. 62–69.

[5] Brilon, W., and F. Weiser. Two-Lane Rural Highways: The German Experience. In Transportation Research Record: Journal of the TransportationResearch Board, No. 1988, Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 38–47.

[6] Van As, C. The Development of an Analysis Method for the Determination of Level of Service on Two-Lane Undivided Highways in South Africa. Project Summary. South African National Roads Agency, Limited, Pretoria, 2003.

[7]Romana, M. G., and I. Pérez. Measures of Effectiveness for Level-of- Service Assessment of Two-Lane Roads: An Alternative Proposal Using a Threshold Speed. In Transportation Research Record: Journal of the Transportation Research Board, No. 1988, Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 56–62.

1.1.7 Ahmed Al-Kaisy and Sarah Karjala[8]

Six performance indicators were investigated in this study:

- Average travel speed,
- Average travel speed of passenger cars,
- Average travel speed as a percent of free-flow speed,
- Average travel speed of passenger cars as a percent of free-flow speed of passenger cars,
- Percent followers, and
- Follower density.

Field data was collected from four study sites in the state of Montana. The study examined the level of association between the selected performance indicators and major platooning variables, namely, ***traffic flow in the direction of travel, opposing traffic flow, percent heavy vehicles, standard deviation of free flow speeds, and percent no-passing zones***.

This study takes the same performance measures and platooning variables and tries to fit regression models among them based on Oregon data.

1.2 Problem Definition

PTSF, used in current HCM Manual, is based heavily on traffic simulation, which lacks field validation. PTSF is very difficult to measure in the field. HCM estimates of PTSF are far from field observations, according to the studies by Luttinen[9] and Dixon et al[10]. Therefore there is a need for alternative and practically measurable performance measures to study operations on two-lane rural highways.

1.3 Study Objective

The objective of this study is to examine an array of performance measures, proposed by Ahmed Al-Kaisy and Sarah Karjala, in regard to their suitability in describing performance on two-lane rural highways for Oregon conditions.

---

[8] A. Al-Kaisy, and S. Karjala, Indicators of Performance on Two-Lane Rural Highways :Empirical Investigation, Transportation Research Record: Journal of the Transportation Research Board, No. 2071, Transportation Research Board of the National Academies, Washington, D.C., 2008, pp. 87–97.

[9] Luttinen, R. T. Percent Time-Spent-Following as Performance Measure for Two-Lane Highways. In Transportation Research Record:Journal of the Transportation Research Board, No. 1776, TRB, National Research Council, Washington, D.C., 2001, pp. 52–59.

[10] Dixon, M. P., S. S. K. Sarepali, and K. A. Young. Field Evaluation of Highway Capacity Manual 2000 Analysis Procedures for Two-Lane Highways. In Transportation Research Record: Journal of the Transportation Research Board, No. 1802, Transportation Research Board of the National Academies, Washington, D.C., 2002, pp. 125–132.

1.4 Study Methodology

The first step in the study is to identify the performance indicators and platooning variables which explain the operations of traffic on rural two-lane highways. Data collection comes next, and requires prior effort in the form of defining site selection criteria, checking the sample size and location, and determining the season and duration of data collection. After collection, data should be processed to obtain the required inputs. Then, a model is formulated, and statistical analysis of data is performed to predict performance measures. Finally, model validation is conducted to check the model accuracy.

2 Adopted Measures of Performance

The following performance measures are adopted for this study.

- Average travel speed (ATS)
- Average travel speed of passenger cars (ATSPC),
- ATS as a percent of free-flow speed (ATS/FFS),
- ATSPC as a percent of free-flow speed of passenger cars (ATSPC/ FFSPC),
- Percent followers (PTfollowers), and
- Follower density (FLdensity)

2.1 Average Travel Speed (ATS)

ATS was used as one of the two performance measures used in the 2000 version of HCM. Average speed does not consider the variations of geometric and other operational characteristics. Although it is easy to measure in the field, ATS alone may not give accurate picture of traffic performance on two-lane rural highways.

2.2 Average Travel Speed of Passenger Cars (ATSPC)

The average travel speed of passenger cars (ATSPC) is currently used in Germany and Finland as a performance indicator. Average travel speed of passenger cars may more accurately describe speed reduction due to traffic because passenger car speeds are more affected by high traffic volumes than are heavy vehicle speeds. This performance indicator has the same limitations and strengths as those for overall ATS discussed earlier.

2.3 Average Travel Speed as Percentage of Free-Flow Speed

Average travel speed as a percentage of free-flow speed (ATS/FFS) is an indicator of the amount of speed reduction due to traffic. If average travel speed is close to free-flow speed, then the interaction among successive vehicles in the traffic stream is small and a high level of service or performance is expected. A lower percentage indicates a higher interaction between vehicles in the traffic stream and therefore a lower quality of service. ATS as a sole measure of performance is a limitation, although this indicator can easily be measured in the field.

2.4 Average Travel Speed of Passenger Cars as Percentage of Free-Flow Speed of Passenger Cars (ATSPC/ FFSPC)

ATSPC as a percentage of the free-flow speed of passenger cars is similar to the previous performance indicator, except that heavy vehicles are not considered in the speed measurements. The rationale behind this performance indicator is that passenger cars more accurately describe speed reduction due to traffic because their speeds are more affected by high traffic volumes than are heavy vehicle speeds. This performance indicator has the same limitations and strengths as those for ATS/FFS.

2.5 Percent Followers

Percent followers represent the percentage of vehicles with short headways in the traffic stream. This performance indicator can easily be measured in the field by using a headway cutoff value of 3 sec as recommended by the HCM. Moreover, the percentage of short headways in the traffic stream is a function mainly of traffic flow level and speed variation. As flow increases, so do the number of short headways and consequently the percent followers. Also, as speed variation increases, the percent followers increase. The main drawback of using percent followers as a sole performance indicator is that it does not accurately reflect the effect of traffic level, which is an important performance criterion in the HCM quality-of-service concept. Theoretically, low traffic levels could still have high percent followers if speed variation is relatively high and passing opportunities are limited. Therefore, the use of percent followers alone could be misleading, particularly for decision making concerning highway improvements and upgrades.

2.6 Follower Density

Follower density is the number of followers in a directional traffic stream over a unit length, typically one mile stretch of a highway. The argument behind using this performance indicator is that a road with low average daily traffic (ADT) and high PTSF should have a lower LOS than the same road with a higher ADT and equal PTSF. The main advantage of using this performance indicator is that, unlike percent followers, it takes into account the effect of traffic level on performance. Although density is difficult to directly measure in the field, it can be estimated at point locations from percent occupancy or from volume and speed measurements using outputs from permanent or temporary traffic detectors.

3 Data Collection & Analysis

Geographic setting, traffic volumes, and terrain are among the considerations for the selection of study sites. All sites are located in rural areas on roughly straight segments, and far from the influence of traffic signals and driveways. In total, data is collected at 17 sites by using automatic traffic recorders. Two data sets were collected at each study site, one in each direction of travel.

For each vehicle, data was recorded on direction of travel, date and time, number of axles, vehicle class, speed (mph), time gap (sec), headway (sec), acceleration (ft/sec^2), and spacing between axles. The 2010 AADT varies from 850 to 8100 vpd, and percent heavy vehicles varies from 5 to of 21%. Except for three sites, all sites are located in level terrain. All sites operate as two-lane two-way traffic. Traffic data was collected over two consecutive two days. Table A1 in the Appendix describes the data collection sites.

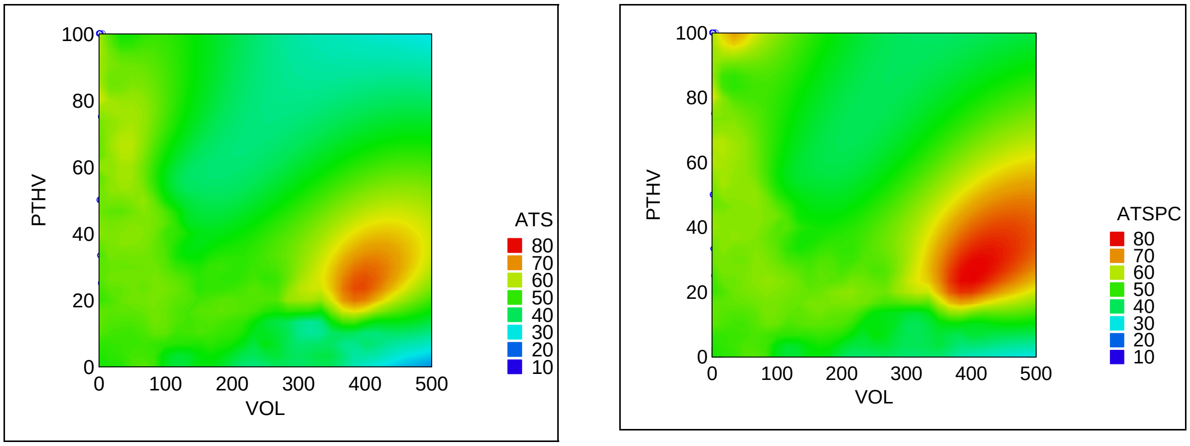
Data from automatic traffic recorders are processed to measure various performance indicators and platooning variables. In the measurement of flow rates for each direction of travel, vehicle counts are aggregated to hourly rates. The percentage of heavy vehicles is found from vehicle classification provided in the recorder output. Free-flow speed is calculated in this analysis by averaging the speed of all vehicles traveling with headways greater than 8 s. Percent followers is calculated using headways less than 3 s. The same headway cutoff value 3 s, is used in determining follower density. Follower density (veh/mil/lane) is calculated as the number of followers (vph) divided by the average follower travel speed (mph). These calculations are performed for each hour at all sites. Site specific hourly data is used to develop site specific models between performance indicators and platooning variables. Later, all site data is aggregated to build the final version of the model.

4 Model Development

Model development is aimed at examining the level of association between performance indicators on two-lane highways and the "platooning" phenomenon through its major contributory factors.

4.1 Data Analysis Results

The analyses involve graphical examination of relationships along with the use of correlation and regression statistical analyses. Site-specific and across-sites examinations are conducted in this study. The relationship between the proposed performance indicators and platooning variables for all the sites combined is plotted first to explore the trends and patterns.



a) Average Travel Speed(ATS)

b) Average Travel Speed of Passenger Cars

Figure 1 Variation of speeds with Volume (VPH) and % Heavy Vehicles (%)

Although as the volume and % heavy vehicles increases ATS decreases, there is no definite pattern observed among sites, as shown in Figure 1. Higher percent of heavy vehicles are observed for lower volumes during the night off peak periods, where heavy vehicle volume usually dominates the traffic flow. Similar trend is observed for ATSPC. Clearly shown in Figure 2, the ratio between ATS and FFS decreases as the volume and % heavy vehicles increases.

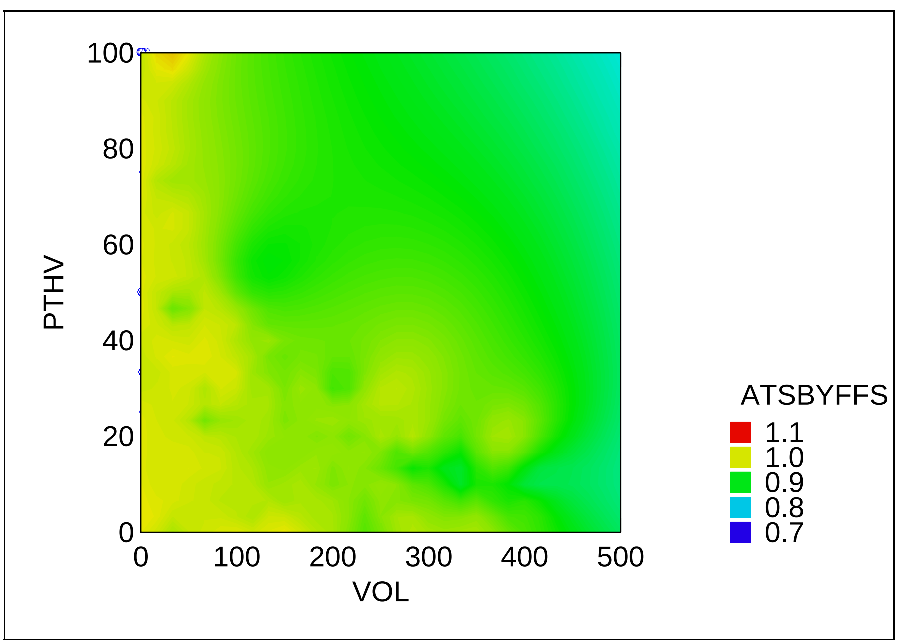


Figure 2 Variation of ratio ATS/FFS with Volume (VPH) and % Heavy Vehicles (%)

Roughly, increases in volume and percent heavy vehicles increases percent vehicles following. This trend is obvious when heavy vehicles are below 25%.

The trends of percent followers with varying VOL and percent heavy vehicles are shown in figure 3.

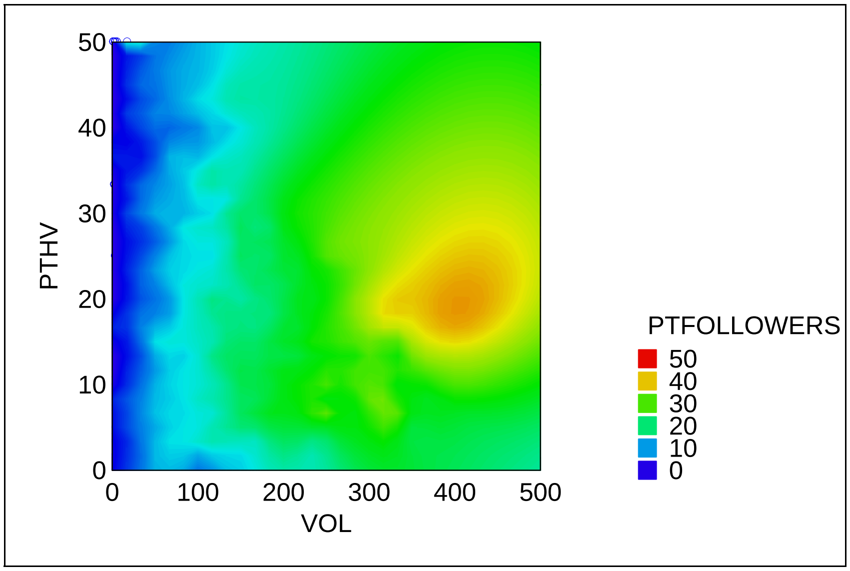


Figure 3 Variation of percent followers with Volume (VPH) and % Heavy Vehicles (%)

Figure 4 shows the bands for follower densities. Follower densities have the value ranging from 0 to 4 vehicles/mile/lane.

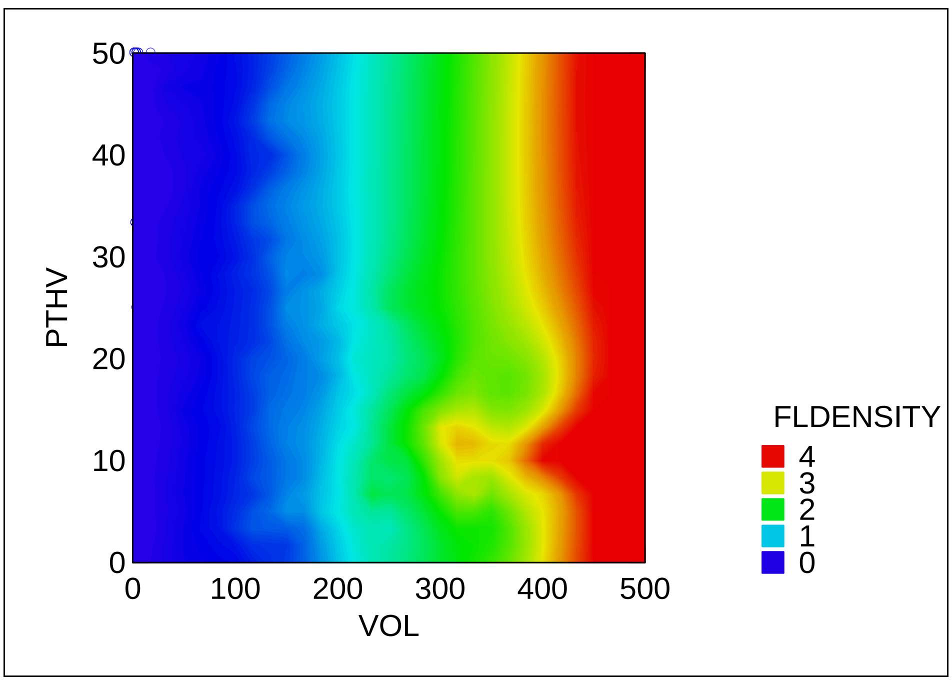


Figure 4 Variation of follower densities with Volume (VPH) and % Heavy Vehicles (%)

As traffic flow increases, average percent followers relatively increases as shown in Figure 5. Similarly, increase in followers reduces average travel speed as shown in Figure 6.

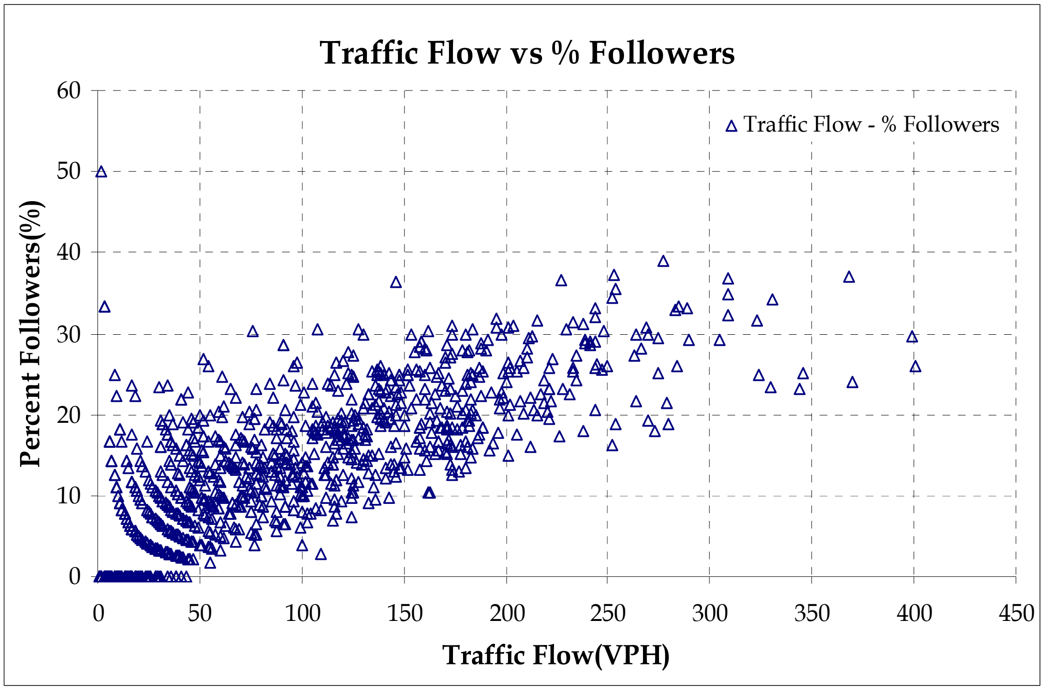


Figure 5 Traffic flow and % followers

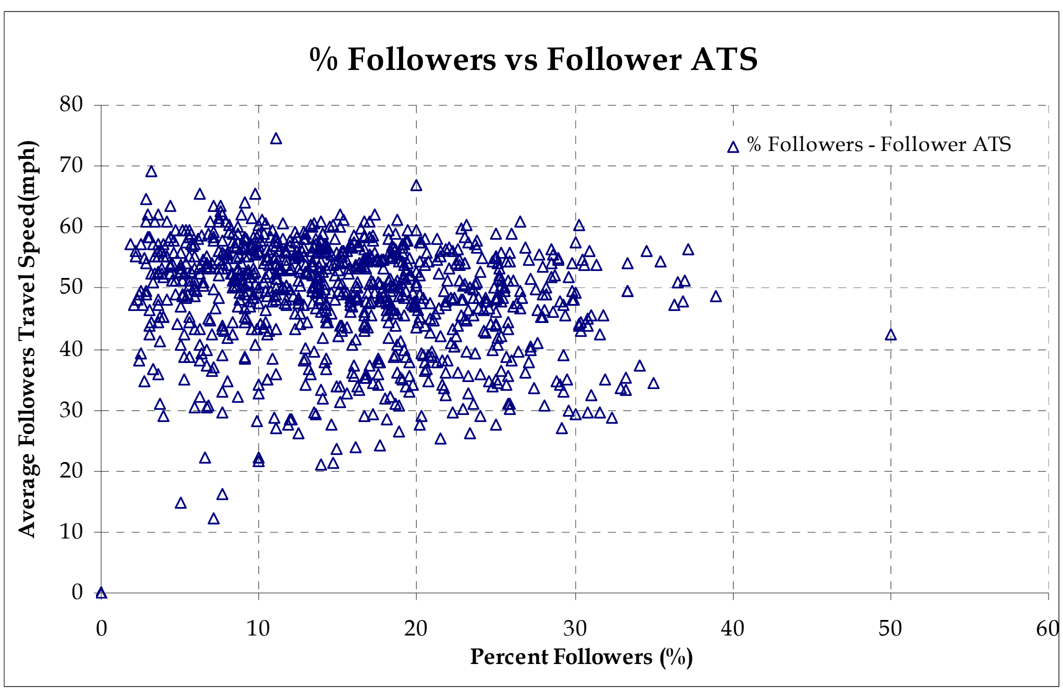


Figure 6 Percent Followers and Average Followers Travel Speed

Figure 7 shows the trend of increasing follower density with increasing percent followers. Figure 8 shows followers average travel speed decreases as the follower density increases.

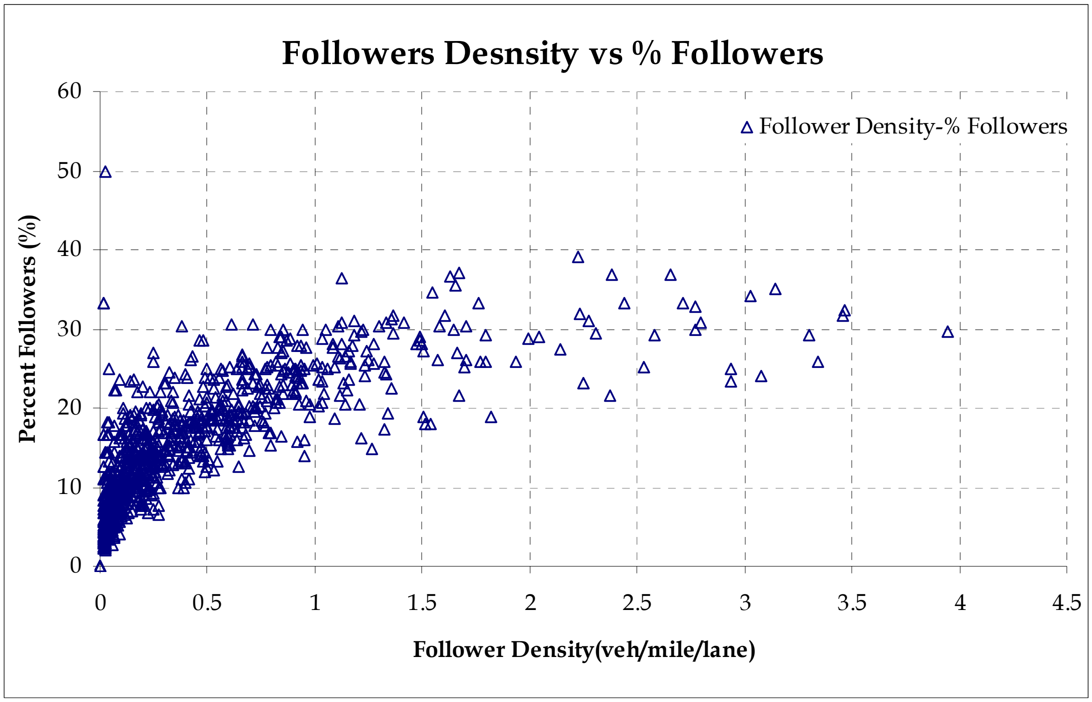


Figure 7 Followers Density and Percent Followers

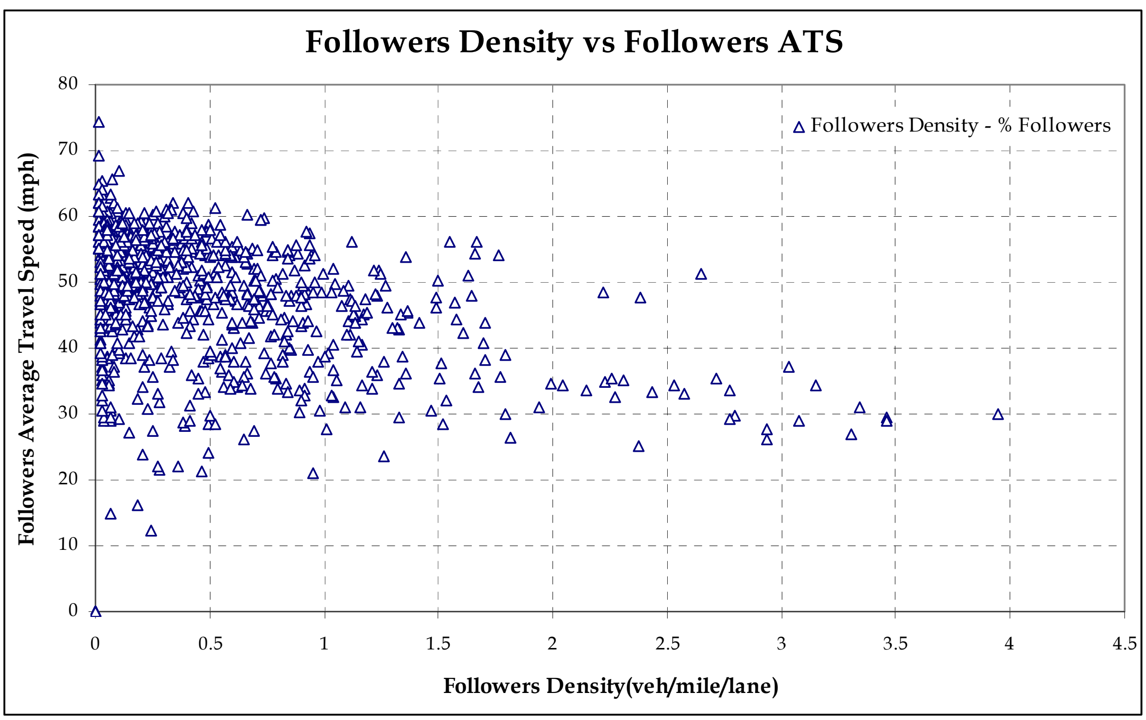


Figure 8 Followers Density and Followers Average Travel Speed

As the volume group and its corresponding opposing volume increases, follower densities increases as distinguished bands, shown in Figure 9.

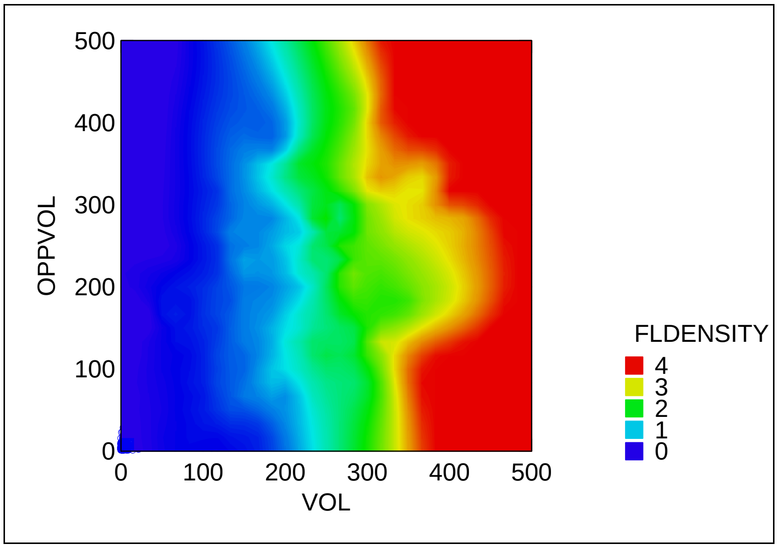


Figure 9 Variation of follower density with volume and opposing volume

Figure 10 shows lower speeds and higher percent followers reflect higher follower densities. Although follower densities show bands, it is very difficult to set LOS intervals based on follower density values for follower ATS and percent followers.

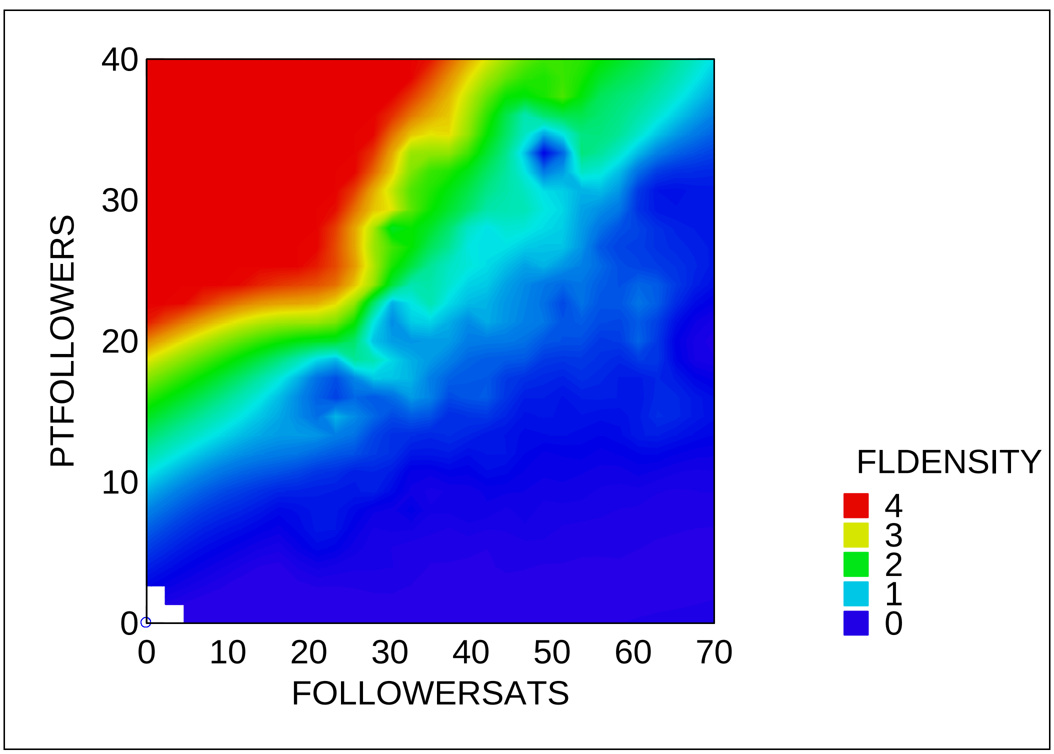


Figure 10 Variation of follower density with volume and opposing volume

4.2 Model Form

The following dependent variables (performance indicators) are considered for the modeling:

- Average travel speed (ATS)
- Average travel speed of passenger cars (ATSPC),
- ATS as a percent of free-flow speed (ATS/FFS),
- ATSPC as a percent of free-flow speed of passenger cars (ATSPC/ FFSPC),
- Percent followers (PTfollowers), and
- Follower density (FLdensity)

Independent variables (platooning variables) considered are:

- Traffic flow in the direction of travel,
- Opposing traffic flow,
- Percent heavy vehicles,
- Standard deviation of free flow speeds, and
- Percent no-passing zones.
- Terrain

The general form of the regression model is:

$$Y = \beta_0 + \beta_1 \times X_1 + \beta_2 \times X_2 + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + \beta n \times X_n$$

Where

Y = Dependent variable

$X_1, X_2, \ldots\ldots X_n$ = Independent or Explanatory Variables

β_0 = Constant

β_1 , β_2 , β_3 = Model coefficients corresponds $X_1, X_2, \ldots\ldots X_n$

4.3 Model

Regression modeling and corresponding statistical analysis was performed using code written in the R statistical package. The statistically significant model is given in Table 1.

Table 1 Fitted models for various performance indicators

```
--------------------------------------------------------------------
model for  FL density   vs    VOL  ,  OPPVOL  ,  PTHV  , % No Passing,
and  Terrain
--------------------------------------------------------------------
Coefficients:
                     Estimate Std. Error t value Pr(>|t|)
(Intercept)        -0.4823332  0.0256702 -18.790  < 2e-16 ***
totaldata$VOL       0.0067640  0.0001519  44.531  < 2e-16 ***
totaldata$OPPVOL   -0.0006175  0.0001516  -4.074  4.9e-05 ***
totaldata$PTHV      0.0008791  0.0003850   2.283   0.0226 *
totaldata$PtNOPASS  0.0008097  0.0001768   4.580  5.1e-06 ***
totaldata$Terrain   0.2482458  0.0180203  13.776  < 2e-16 ***
---
Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

Residual standard error: 0.2074 on 1266 degrees of freedom
Multiple R-squared: 0.8456,   Adjusted R-squared: 0.845
F-statistic:  1387 on 5 and 1266 DF,  p-value: < 2.2e-16
```

Based upon the statistical analysis, follower density is chosen as the performance indicator. The model fitted between Follower Density (veh/mile/lane) as the independent variable, and Traffic Flow (vph) , Opposing Flow (vph) , Percent of Heavy Vehicles (%) , Percentage No Passing (%), and Terrain as the dependent variables, has the highest R^2 value and statistical significance.

```
Follower Density =
-0.4823332+0.0067640(Traffic Volume)-0.0006175(Opposing Volume)
+0.0008791 (%Heavy Vehicles) +0.0008097 (% No Passing)
+ 0.2482458 (Terrain)
R2 = 0.845
Terrain: Either Level or Rolling Type
1 for Level ; and 2- for Rolling
```

The next section deals with model validation and checking the model consistency for varying conditions.

4.4 Performance of Montana Study

The developed model is compared with the similar study done in the State of Montana by Ahmed Al-Kaisy and Sarah Karjala[11]. The Montana Models are shown below.

Performance Indicator	Linear Regression Model	R^2	Multiple R
Follower density (veh/mi)	Follower Density = 0.01041 (volume in vph) − 0.00022 (opposing volume in vph) − 0.03057 (% heavy vehicles) + 0.00500 (% no-passing zones) + 0.11670 (standard deviation of free-flow speed in mph)	0.96	0.98
% followers	% Followers = 0.03380 (volume in vph) + 0.00607 (opposing volume in vph) − 0.16062 (% heavy vehicles) + 0.10894 (% no-passing zones) + 2.12739 (standard deviation of free-flow speed in mph)	0.62	0.79

Figure 11 shows the performance of Montana Model when compared to observed follower density. A linear relation does not exist between them. The error between observed and Montana model exceeds 1 vehicle/mile/lane as given in Figure 12.

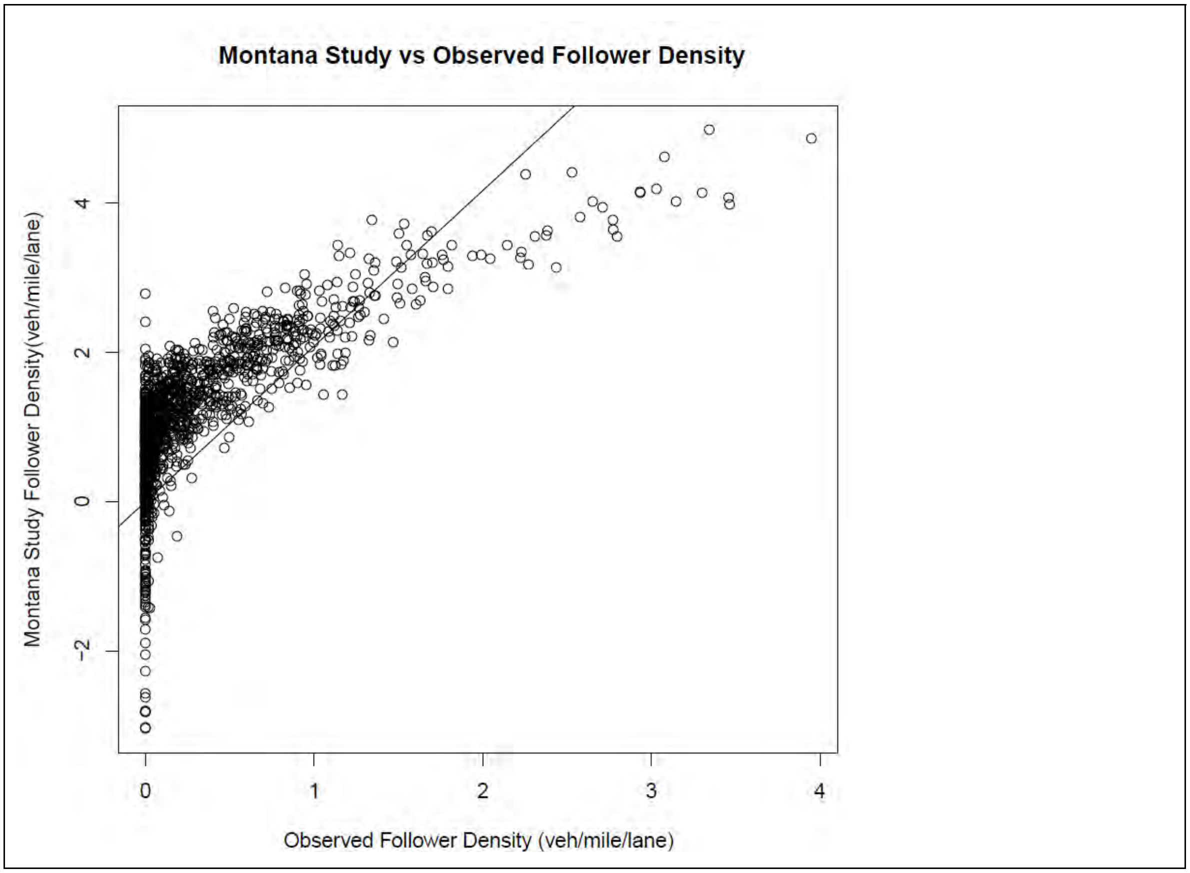


Figure 11 Montana Study and Observed Follower Density

[8]A. Al-Kaisy, and S. Karjala, Indicators of Performance on Two-Lane Rural Highways :Empirical Investigation, Transportation Research Record: Journal of the Transportation Research Board, No. 2071, Transportation Research Board of the National Academies, Washington, D.C., 2008, pp. 87–97.

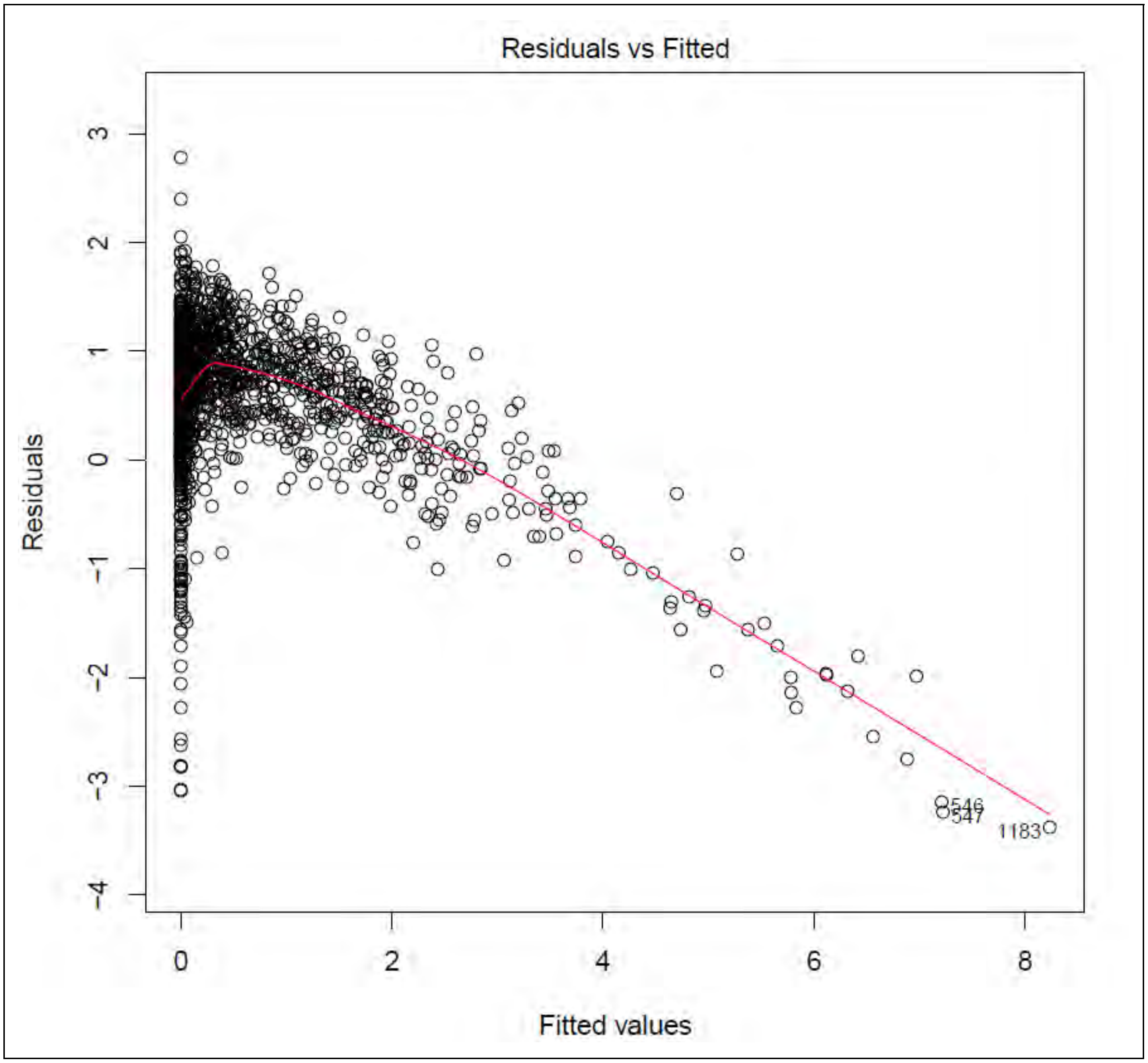


Figure 12 Error between Montana Study Follower Density and Observed Follower Density

5 Model Validation

5.1 Data Collection

The model is validated by using data sets from four sites. These sites are part of the original data collection efforts and are separated based on AADT, terrain, and geographic region to cover all possible conditions. The sites that are considered and their brief description are given in Appendix A, Table A2. Data analysis and preparation of data sets are done according to the procedure mentioned in the section 3.

5.2 Data Analysis

Hourly data from all sites are tested with the developed model and comparison is made between observed field densities and predicted field densities. The error between observed and predicted densities varies by ± 0.5 veh/mile/lane as shown in Figure 13.

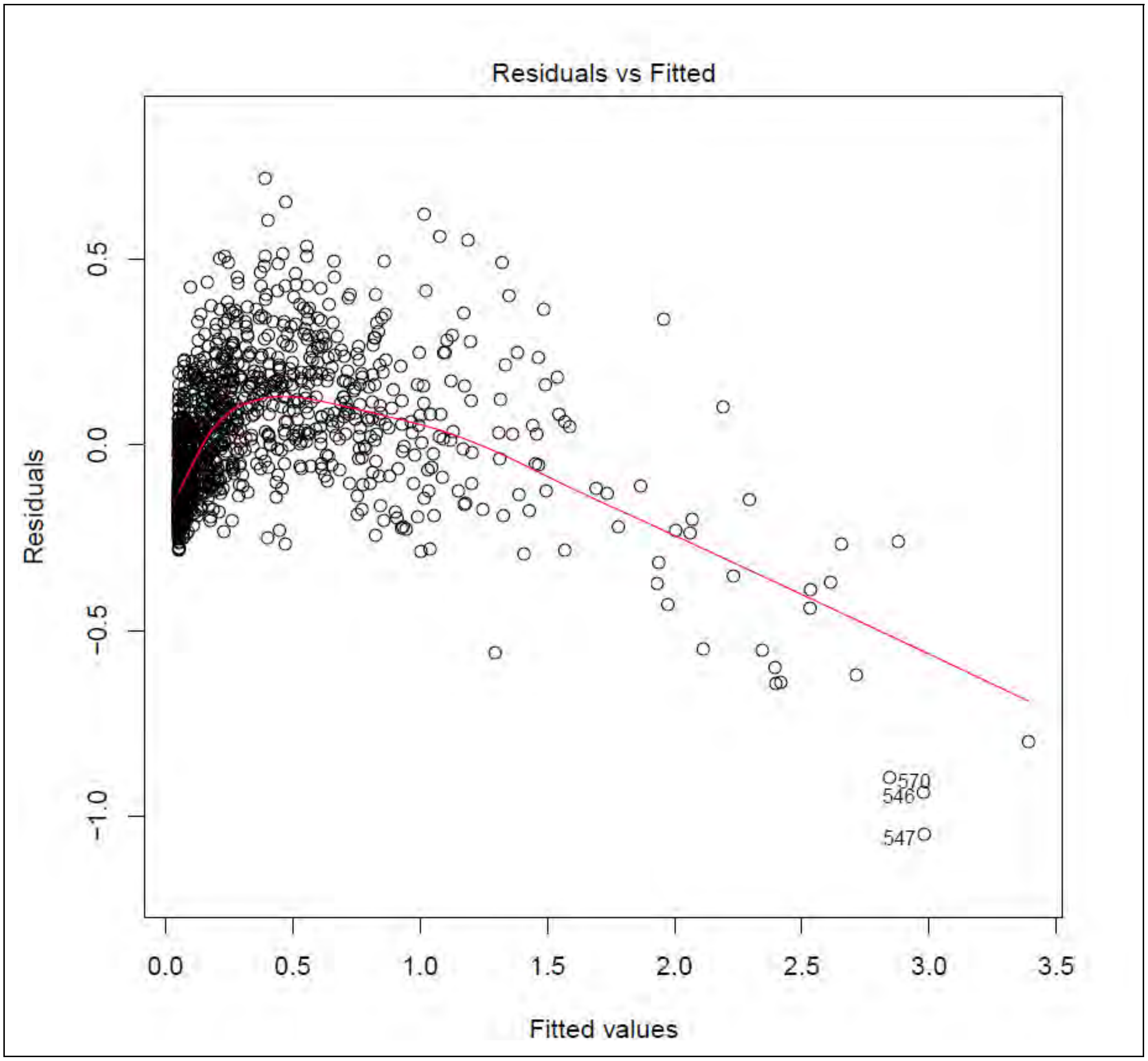


Figure 13 Residuals plot of predicted follower density and observed follower density

A regression model was fitted between the predicted follower density and observed follower densities. A summary of the regression analysis is listed in Table 2 and shown in Figure 14.

Table 2 Regression model between predicted follower density and observed follower density

```
Call:
lm(formula = totaldata$predictedFLdensity ~ totaldata$FLdensity,
    data = totaldata)
Coefficients:
                    Estimate Std. Error t value Pr(>|t|)
(Intercept)         0.051884   0.006334   8.191 6.26e-16 ***
totaldata$FLdensity 0.845617   0.010139  83.404  < 2e-16 ***

Signif. codes:  0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1
Residual standard error: 0.1904 on 1270 degrees of freedom
Multiple R-squared: 0.8456,   Adjusted R-squared: 0.8455
F-statistic:  6956 on 1 and 1270 DF,  p-value: < 2.2e-16
```

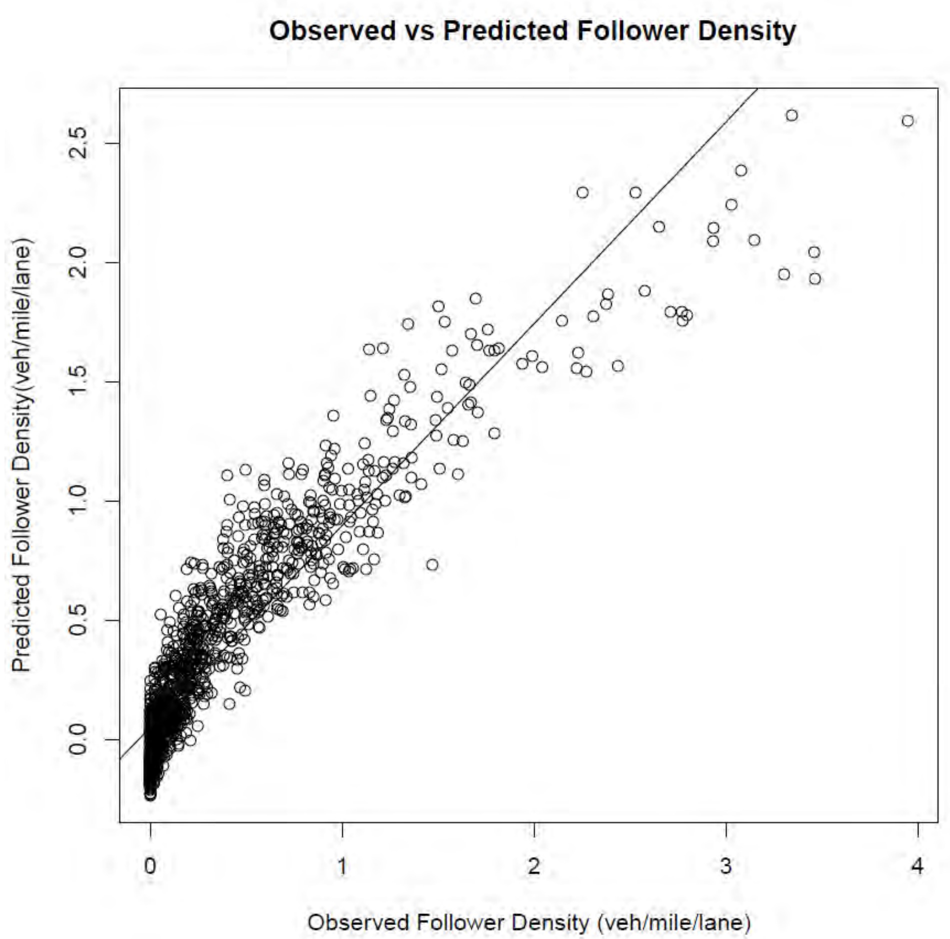


Figure 14 Observed follower density and predicted follower density

5.3 Model Comparison

Both the developed model and the Montana model are compared against the error in estimating follower densities. The error between observed follower densities and developed model estimated follower densities varies between ±0.5 veh/mile/lane. The error varies from + 1.3 veh/mile/lane to -2.7 veh/mile/lane for the Montana model.

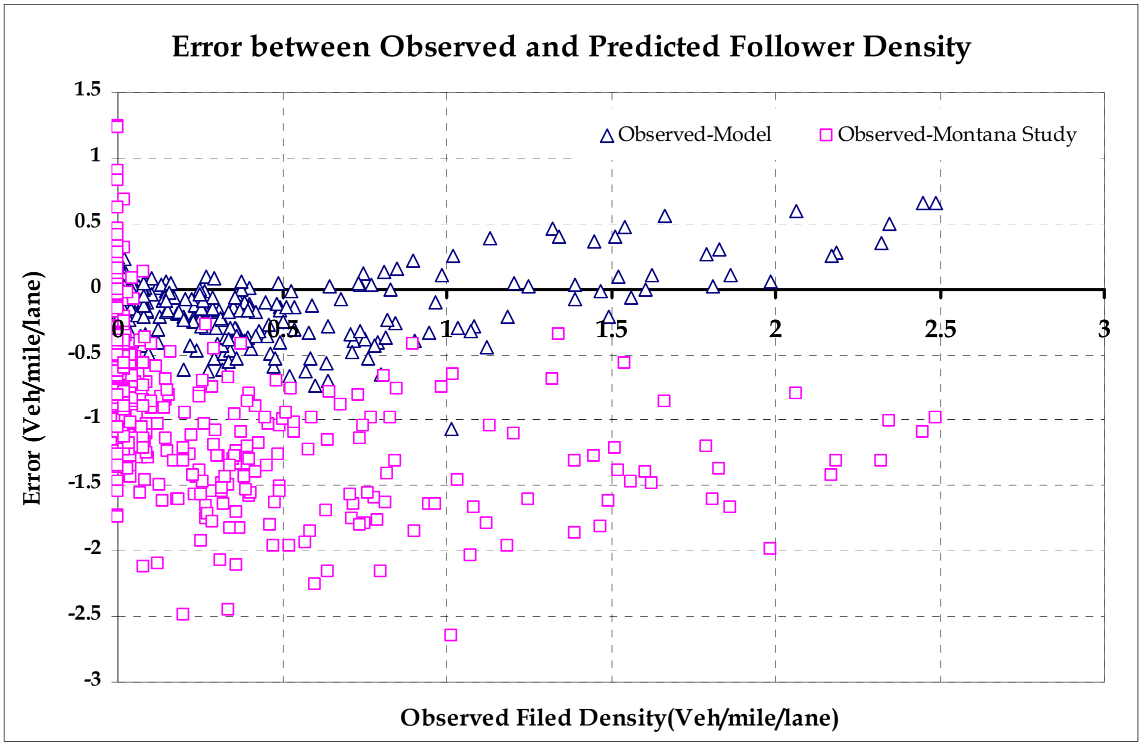


Figure 15 Error Variation among the Models

6 Summary

The difficulty in measuring Percentage Time Spent Following (PTSF) in the field led to the study of alternative performance measures to deal with operations on two-lane rural highways. This study is based on research work done in the state of Montana, expanded to state of Oregon rural two-lane highway conditions. Performance indicators Average travel speed (ATS), Average travel speed of passenger cars (ATSPC), ATS as a percent of free-flow speed (ATS/FFS), ATSPC as a percent of free-flow speed of passenger cars (ATSPC/FFSPC), Percent followers (PTfollowers), and Follower density (FLdensity) are tested on data collected through detectors at 13 sites. Regression models are developed by taking the above mentioned performance indicators as dependent variables, and the platooning variables, such as traffic flow in the direction of travel, opposing traffic flow, percent heavy vehicles, standard deviation of free flow speed, percent no-passing zones, and terrain as independent variables.

Out of various combinations, the model with follower density versus traffic flow, opposing volume, percent of heavy vehicles, percent no passing zones, and terrain yields better statistical significance. Later, data from 4 sites were used to validate the model. The error between the observed follower density and predicted follower density varies by ± 0.5 veh/mile/lane. The variation of observed follower density with average travel speed and percent followers has groups, but does not have clearly cut boundaries to mark level-of-service zones. Moreover, volume to capacity ratios on two-lane rural roads are small. Observed follower density varies from 0 to 4 veh/mile/lane. A wide spectrum of follower densities may designate more clear cut level of service categories.

```
Follower Density =
-0.4823332+0.0067640(Traffic Volume)-0.0006175(Opposing Volume)
+0.0008791 (%Heavy Vehicles) +0.0008097 (% No Passing)
+ 0.2482458 (Terrain)
R2 = 0.845
Terrain: Either Level or Rolling Type
1 for Level ; and 2- for Rolling
```

7 Scope for Future Work

The model can be refined by obtaining more site data covering a spectrum of geographic areas, higher volumes, and rolling and mountainous sites. Following other vehicles is critical during peak periods, so consideration should be given to further model development using variables calculated in those periods.

Appendix A -Description of Data Collection Sites

Table A1 Sites used for model development

No	TSM Site Id	Hwy_ No	BMP	MP (Count Loc)	EMP	Description	AADT	County	FC	% Trucks	Terrain	% No Passing
1	967	8	23.45	23.47	26.20	0.02 mile north of Blue Mt. Station Rd	6400	Umatilla	2	12	L	0%
2	1336	10	22.11	24.59	24.61	0.02 mile west of Good Rd	1800	Union	2	21	L	45%
3	1352	10	65.87	68.46	68.59	0.02 mile west of Crow Creek Rd	3300	Wallowa	2	21	L	90%
4	1610	21	6.46	6.61	9.18	Siskiyou ATR 15-007	1000	Jackson	6	7	L	85%
5	1654	22	45.31	51.37	54.87	0.10 mile east of Woodruff Bridge Rd	1900	Jackson	6	9	L	0%
6	1656	22	57.31	57.81	61.94	0.50 mile east of West Diamond Lake Hwy	850	Jackson	7	9	R	5%
7	3240	140	42.88	43.33	44.83	0.22 mile south of Bonney Rd	6000	Marion	6	5	L	25%
8	3437	160	20.76	22.15	22.96	Marquam ATR 03-013	3900	Marion	6	10	L	35%
9	3449	161	7.59	7.69	9.11	0.10 mile east of Canby-Marquam Rd	6000	Marion	6	9	L	20%
10	3558	171	30.92	39.13	39.23	0.10 mile north of Fish Creek Rd	1400	Clackamas	7	5	L	45%
11	3775	212	0.21	1.45	1.47	0.02 mile west of Bond Road	4400	Linn	6	16	L	55%
12	4074	272	8.64	9.76	11.83	At Johnson Creek, 0.07 mi east of Crystal Dr	2900	Jackson	6	5	L	100%
13	4118	281	3.61	4.16	5.09	0.02 mile south of Portland Dr	8100	Hood River	6	7	R	75%

Table A2 Sites used for model validation

No	TSM Site Id	Hwy_ No	BMP	MP (Count Loc)	EMP	Description	AADT	County	FC	% Trucks	Terrain	% No Passing
1	961	8	12.75	16.05	16.07	0.02 mile west of Pambrun Rd	4500	Umatilla	2	12	L	40%
2	2916	71	40.11	41.75	41.85	0.10 mile west of Dooley Mtn Hwy	1000	Baker	6	6	L	100%
3	3565	172	1.55	1.56	3.14	0.10 mile northeast of Judd Rd	5700	Clackamas	6	9	R	35%
4	4059	271	10.68	10.83	15.81	0.15 mile east of Table Rock Road	2500	Jackson	6	9	L	10%

Modeling Follower Density on Two-Lane Rural Highways

Transportation Development Division
Oregon Department of Transportation

February, 2014

Table of Contents

List of Figures

List of Tables

Introduction

Two-lane highway operations are characterized by passing maneuvers, formation of platoons within the traffic stream, and delay experienced by trailing vehicles while unable to pass lead vehicles. For increased passing demand, passing capacity decreases due to limited passing opportunities. Quality of service becomes unacceptable even for lower volume-to-capacity ratio. Hence, use of volume-to-capacity ratio may not be a good performance measure for two-lane highway analysis.

The HCM 2010 manual uses Percent-Time Spent Following (PTSF), Average Travel Speed (ATS), and Percent Free-flow Speed (PFFS) measures to assess two-lane highways operations. The PTSF measure is difficult to measure in the field. Lack of field validation and difficulty in obtaining PTSF measure in the field led to the development of alternative performance measures for two-lane highway operations. The Oregon Department of Transportation (ODOT) has conducted studies to develop alternative performance measures for two-lane highway analysis[1]. The studies were based on the framework adopted for empirical investigation of two-lane rural highway performance indicators in Montana (*Al-Kaisy and Karjala, 2008*). A preliminary study showed promising measures with limited data. However, the study did not provide any LOS thresholds based on alternative performance measure. Hence, an extension of the study was necessary using expanded datasets.

Objectives

The primary objectives of the study are to:

- Develop and select alternative performance measures for two-lane rural highway analysis
- Refine Level-of-Service(LOS) thresholds based on the selected measure(s)

Study Outline

The study identified performance indicators and platooning variables that influence operations of two-lane highways. Data collection and processing efforts provided required inputs to the model development. After model validation, follower density LOS thresholds were formulated for the identified two-lane highways classes.

Two Lane Highway Classes

The HCM 2010 two-lane highways methodology classified rural highways into three classes. The primary reason to establish the classification was to account for wide range of functionality and driver behavior. The present study also tries to develop models for different classes of rural highways. As per the HCM, arterials are considered to be Class I highways, and most collectors

[1] Modeling Performance Indicators on Two-Lane Rural Highways: The Oregon Experience (ODOT 2010 Study)

and local roads are considered to be Class II. Class III highways are a special case and may be any functional class. Definitions of the three classes are (HCM, 2010):

Class I two-lane highways are highways where motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks are generally assigned to Class I. These facilities serve mostly long-distance trips or provide the connections between facilities that serve long-distance trips. Rural Principal Arterials (Functional Class 02 highways) mostly act as Class I highways. Coos Bay-Roseburg Highway-OR 42 (No. 35) is an example of a Class I highway.

Class II two-lane highways are highways where motorists do not necessarily expect to travel at high speeds. Two-lane highways functioning as access routes to Class I facilities, serving as scenic or recreational routes (and not as primary arterials), or passing through rugged terrain (where high-speed operation would be impossible) are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning or ending portions of longer trips, or trips for which sightseeing plays a significant role. Rural Minor Arterials (Functional Class 06 highways) and Rural Major Collectors (Functional Class 07) mostly act as Class II highways. For instance, West Diamond Lake Hwy- OR 230 (No. 233) that connects Crater Lake Hwy (OR 62) and Diamond Lake Hwy (OR 138) primarily serves recreational trips and passes through undeveloped, rugged terrain.

Class III two-lane highways are special cases serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns, unincorporated communities, or developed recreational areas. On such segments, local traffic often mixes with through traffic, and the density of unsignalized roadside access points is noticeably higher than in a purely rural area. Class III highways may also be longer segments passing through more spread-out recreational areas, also with increased roadside densities. Such segments are often accompanied by reduced speed limits that reflect the higher activity level. Any signalized intersections in these areas convert the section to an urban street and this method no longer applies. Some example sections:

- Gearhart to Warrenton section on Oregon Coast Hwy-US 101 (No. 9)
- Detroit city section on N Santiam Hwy-OR 22 (No. 162)
- Richland city section on Baker – Copperfield Highway-OR 86 (No. 12)

The rural US 101 section from Gearhart to Warrenton is a spread-out recreational area with substantial development along the highway. The Detroit and Richland sections of the highways pass through small towns having speed restrictions, significant road side developments and unsignalized access points.

Adopted Performance Measures

The following performance measures were adopted for this study.

- Average travel speed (ATS)

- ATS as a percent of free-flow speed (PFFS),
- Percent followers (PTfollowers), and
- Follower density (FLdensity)

Percent followers represent the percentage of vehicles with short headways in the traffic stream. This performance indicator can easily be measured in the field by using a headway cutoff value of 3.0 seconds as recommended by the HCM 2010 manual. Follower density is the number of followers in a directional traffic stream over a unit length. Follower density measure considers the effect of both traffic level and speed on the performance. Generally, density is difficult to directly measure in the field. But, it can be estimated at point locations from volume and speed measurements using outputs from traffic detectors.

Data Collection & Analysis

Data collection sites were selected based on geographic setting, traffic volumes, and terrain. All sites were located in rural areas on roughly straight segments, and far from the influence of traffic signals and driveways. In total, data is collected at 168 sites by using automatic traffic recorders. Two data sets were collected at each study site, one in each direction of travel.

For each vehicle, data on vehicle class, speed (mph), headway (seconds), percentage of no passing zone, terrain (level, rolling, or mountainous), and functional classification (ODOT highway functional classification; 2= Rural Principal Arterial; 6= Rural Minor Arterial; and 7= Rural Major Collector) was collected. All sites operate as two-lane two-way traffic and traffic data was from year 2009 to 2013.

Data from automatic traffic recorders were processed to measure various performance indicators and platooning variables. For each direction of travel, vehicle counts were aggregated to hourly rates. The percentage of heavy vehicles was found from vehicle classification provided in the recorder output. Free-flow speed was calculated in this analysis by averaging the speed of all vehicles traveling with headways greater than 8.0 seconds (*Al-Kaisy and Karjala, 2008*) . Percent followers were calculated using headways less than 3.0 seconds (*HCM, 2010*). Follower density (veh/mile/lane) is the number of followers (vph) divided by their average travel speed (mph). Similar calculations were performed for each hour at all sites.

Study Methodology

The study segregated highway sections into Class I, II, and III highways based on HCM definition. Rural principal arterial mostly acts as Class I Highways, rural minor arterials and rural major collectors are Class II highways and some portions of Class I and Class II highways are Class III highways. Once the raw data was processed, regression models were developed between performance indicators and other explanatory variables for each class of rural highways. A part of data set was used to validate these models. After the model validation, LOS thresholds were developed.

Model Development

Model development is aimed at examining the level of association between performance indicators on two-lane highways and its major contributory factors. Scattered plot between the performance indicators and platooning variables reveal the trends and patterns existed in the data. As traffic flow and opposing volume increases, follower density increases as shown in Figure 1. Similarly, increase in follower density reduces average travel speed. Geographic distribution of follower density varies by region (Figure 2). Follower density increases as terrain changes from level to mountainous (Figure 3). Follower densities on Class II highways is more than Class I highways (Figure 4).

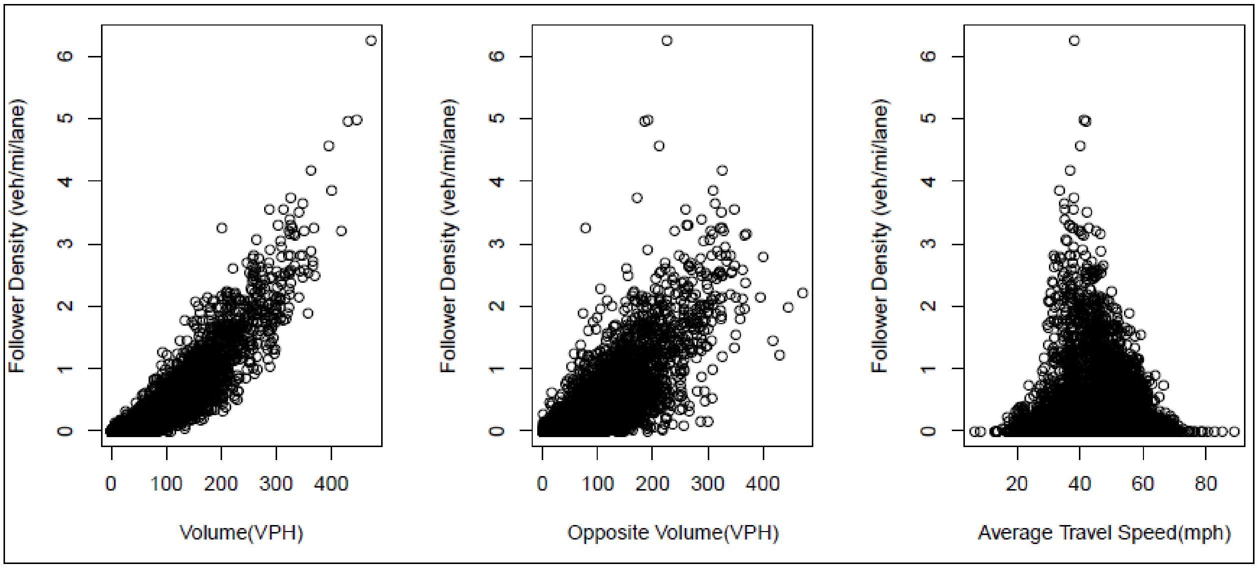


Figure 1. Follower Density versus Explanatory Variables

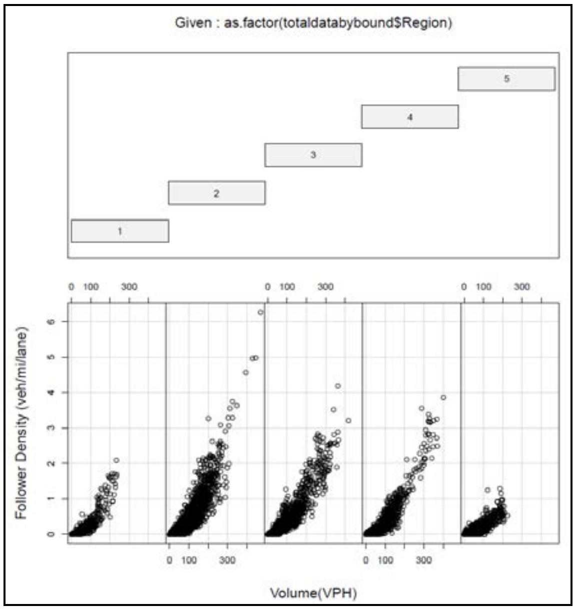


Figure 2. Follower Density by ODOT Region

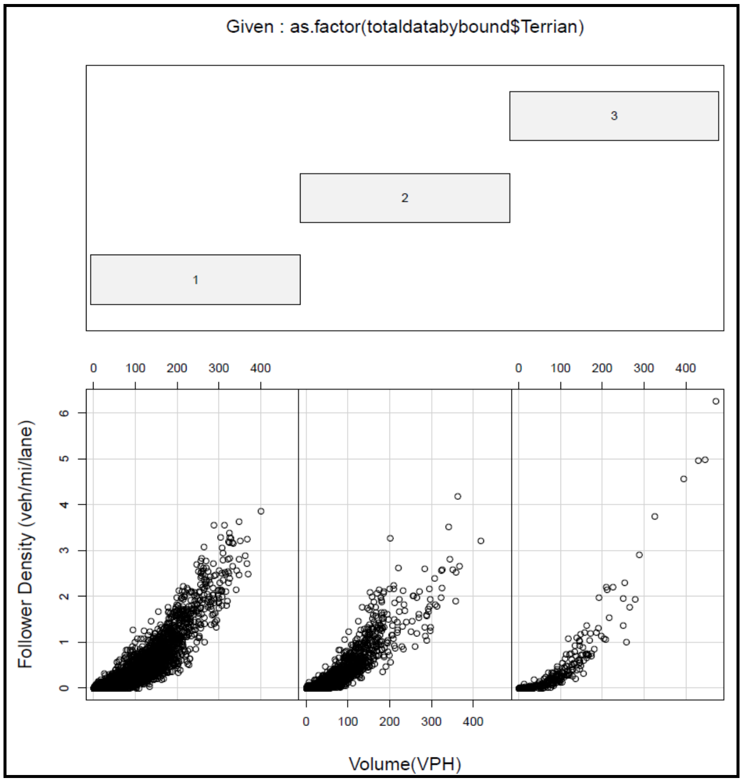


Figure 3. Variation of Follower Density by Terrain (1-level; 2-rolling; 3-mountainous)

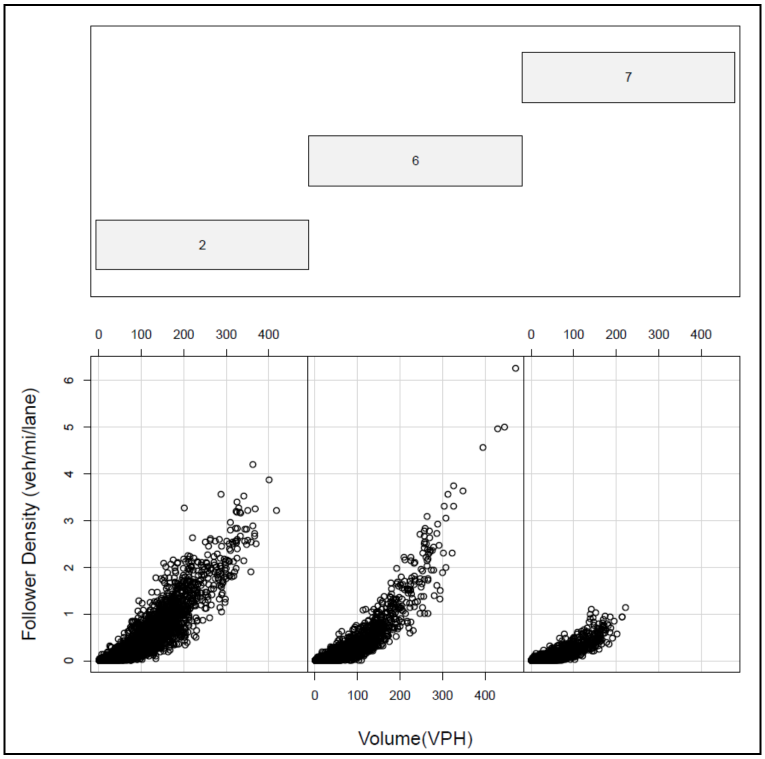


Figure 4. Follower Densities by Rural Highway Functional Classification (2-Rural Principal Arterial; 6-Rural Minor Arterial; 7-Rural Major Collector)

Model Form

The following dependent variables (performance indicators) are considered for the modeling:

- Average travel speed (ATS) in mph,
- ATS as a percent of free-flow speed (PFFS) in %,
- Percent followers (PTfollowers) in %, and
- Follower density (FLdensity) in veh/mile/lane.

Independent variables (explanatory or platooning variables) considered are:

- Traffic flow in the direction of travel (veh/h),
- Opposing traffic flow (veh/h),
- Percent heavy vehicles (%),
- Percent no-passing zones (%),
- RTerrain (dummy variable; 1 = Rolling Terrain, 0 = Otherwise) , and
- MTerrain (dummy variable; 1 = Mountainous Terrain, 0 = Otherwise).

The general form of the regression model is:

$$Y = \beta_0 + \beta_1 \times X_1 + \beta_2 \times X_2 + \ldots\ldots\ldots\ldots\ldots\ldots + \beta n \times X_n$$

Where

Y = Dependent variable

$X_1, X_2, \ldots\ldots X_n$ = Independent or Explanatory Variables

β_0 = Constant; β_1 , β_2 , β_3 = Model coefficients corresponds $X_1, X_2, \ldots\ldots X_n$

Models by Highway Functional Class

Regression modeling and corresponding statistical analysis was performed using the code written in the R programming language. Follower density and all other explanatory variables were calculated using R code. The code also facilitated model selection, development and validation. Based upon the statistical analysis, follower density is chosen as the performance indicator. The model fitted between follower density (veh/mile/lane) as the dependent variable, and traffic flow (vph), opposing flow (vph), percent of heavy vehicles (%), percentage no passing (%), RTerrain, and MTerrian as the independent variables, has the highest R^2 value and statistical significance. Table 6 lists follower density models by highway functional class.

Table 1. Follower Density Models by Rural Functional Classification

Class	Model Form	R^2
I	Follower Density = -0.1917 + 0.005953 (Traffic Volume) + 0.0005167 (Opposing Volume) + 0.0006739 (% Heavy Vehicles) + 0.0002392 (% No Passing) + 0.05248 (Rolling Terrain)	0.81
II	Follower Density = -0.1784 + 0.006189 (Traffic Volume) - 0.0001607 (Opposing Volume) + 0.0006163 (% Heavy Vehicles) + 0.0006055 (% No Passing) + 0.0168 (Rolling Terrain) + 0.03994 (Mountainous Terrain)	0.75
III	Follower Density = -0.04062 + 0.003244 (Traffic Volume) - 0.0003219 (Opposing Volume) + 0.0001127 (% Heavy Vehicles) + 0.0001877 (% No Passing) - 0.007543 (Rolling Terrain) - 0.01995 (Mountainous Terrain)	0.74

After model development, model validation was performed. Next section presents validation and model consistency checking efforts.

Model Validation

Developed model was validated by using data sets from 56 sites. These sites were a part of the original data collection efforts and were separated based on AADT, terrain, and geographic region to cover all possible conditions. Hourly data from all sites were tested with the developed model and comparison is made between observed follower densities and predicted follower densities. The developed models were also compared with the similar study done in the State of Montana (*Al-Kaisy and Karjala, 2008*). The Montana models were shown in Figure 5.

Performance Indicator	Linear Regression Model	R^2	Multiple R
Follower density (veh/mi)	Follower Density = 0.01041 (volume in vph) – 0.00022 (opposing volume in vph) – 0.03057 (% heavy vehicles) + 0.00500 (% no-passing zones) + 0.11670 (standard deviation of free-flow speed in mph)	0.96	0.98
% followers	% Followers = 0.03380 (volume in vph) + 0.00607 (opposing volume in vph) – 0.16062 (% heavy vehicles) + 0.10894 (% no-passing zones) + 2.12739 (standard deviation of free-flow speed in mph)	0.62	0.79

Source: *Al-Kaisy and Karjala, 2008*

Figure 5. Montana Study Models

Figure 6 shows the linear relationship between predicted follower density and observed follower density. However, the relationship between follower densities from Montana study and observed follower densities did not show a liner trend (Figure 7). Table 2 lists relationships between model and observed follower densities.

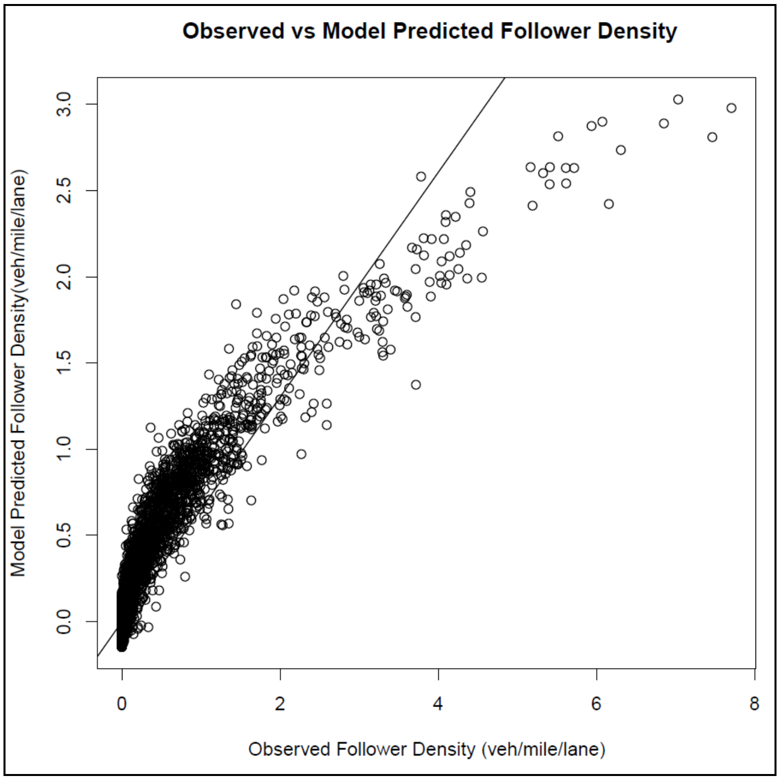


Figure 6. Observed Follower Density versus Predicted Follower Density

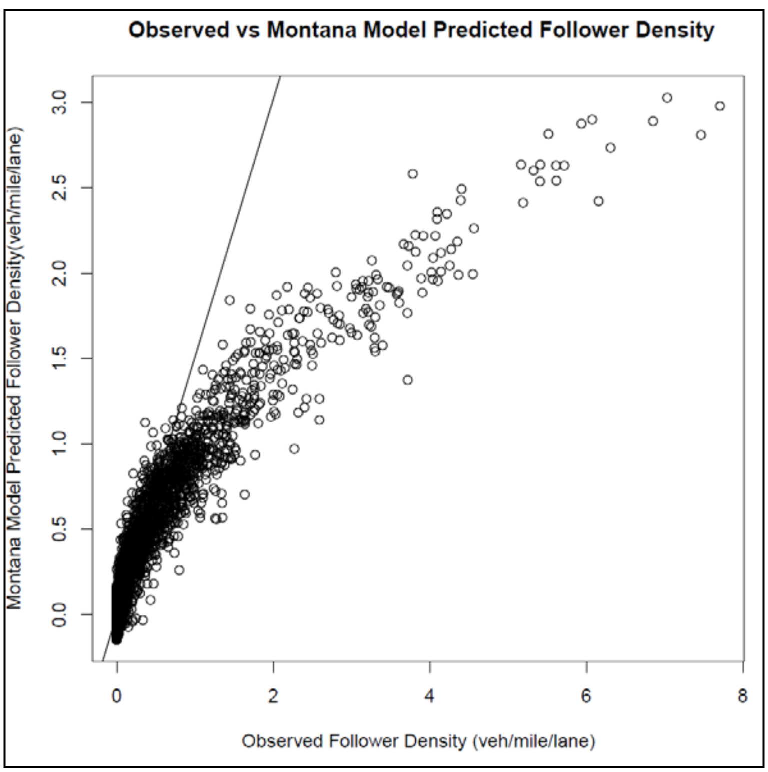


Figure 7. Observed Follower Density versus Follower Density from Montana Study

Table 2. Relationship between Model Follower Density and Observed Follower Density

Item	Model	R^2
ODOT Model vs Observed	Model Follower Density = 0.6514 (Observed Follower Density)	0.85
Montana Study vs Observed	Montana Study Follower Density = 1.5093 (Observed Follower Density)	0.57

Both the developed model and the Montana study model were compared against the error in estimating follower densities. Errors for developed model are less compared to Montana study models on class I highways (Figure 8). However, the Montana model error becomes smaller as the follower density increases. Similar trend was observed for Class II and Class III highways. This study used a follower density difference of ± 0.5 vehicle/mile/lane as the acceptable range; a difference greater than + 0.5 vehicle/mile/lane was labeled as over-estimated, and less than -0.5 vehicle/mile/lane was treated as under-estimated. Data on percent of acceptable, under-estimated and over-estimated follower densities were used for models comparison. Error distribution plots, Figures 9 to 11, show Montana study model over predicting the follower densities. On an average 90 percent of observations have acceptable range of error when using the developed models. However, only 30 percent of observations show acceptable range of error with Montana study model.

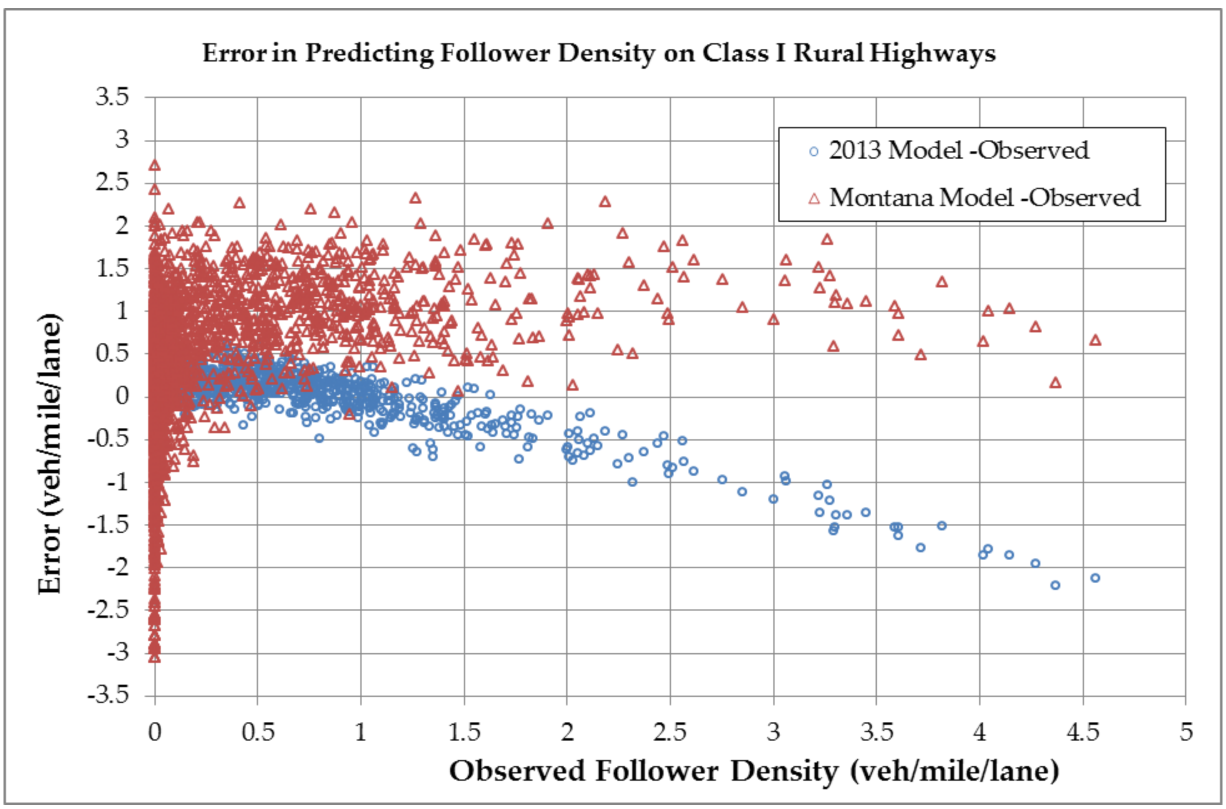


Figure 8. Error between Predicted and Observed Follower Densities for Class I Highways

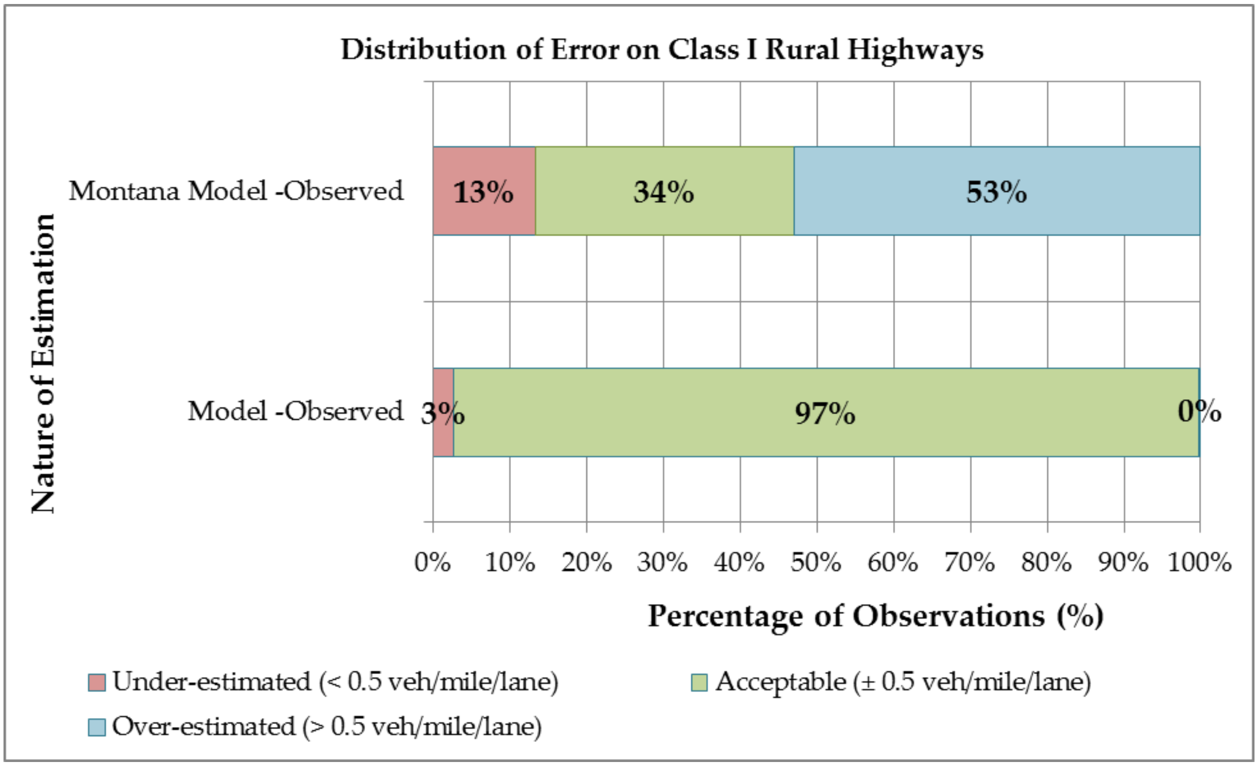


Figure 9. Comparison of Developed Model and Montana Model for Class I Highways

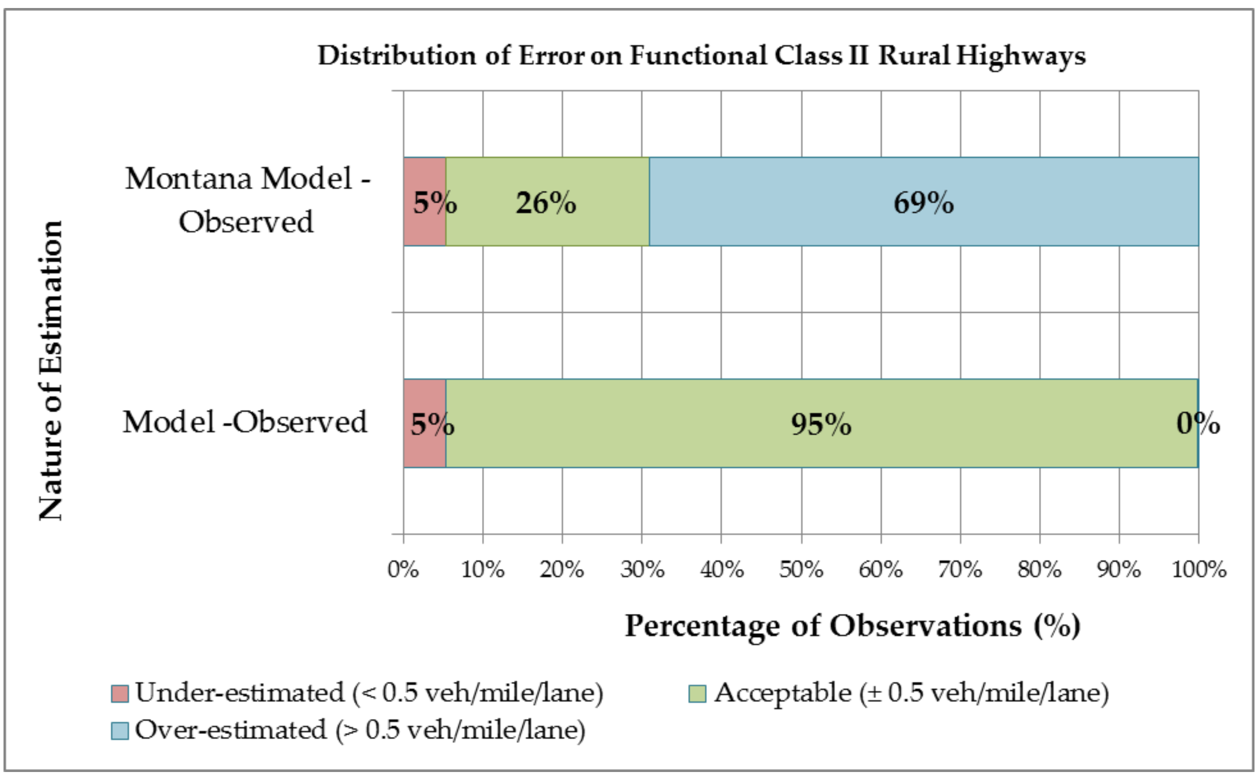


Figure 10. Comparison of Developed Model and Montana Model for Class II Highways

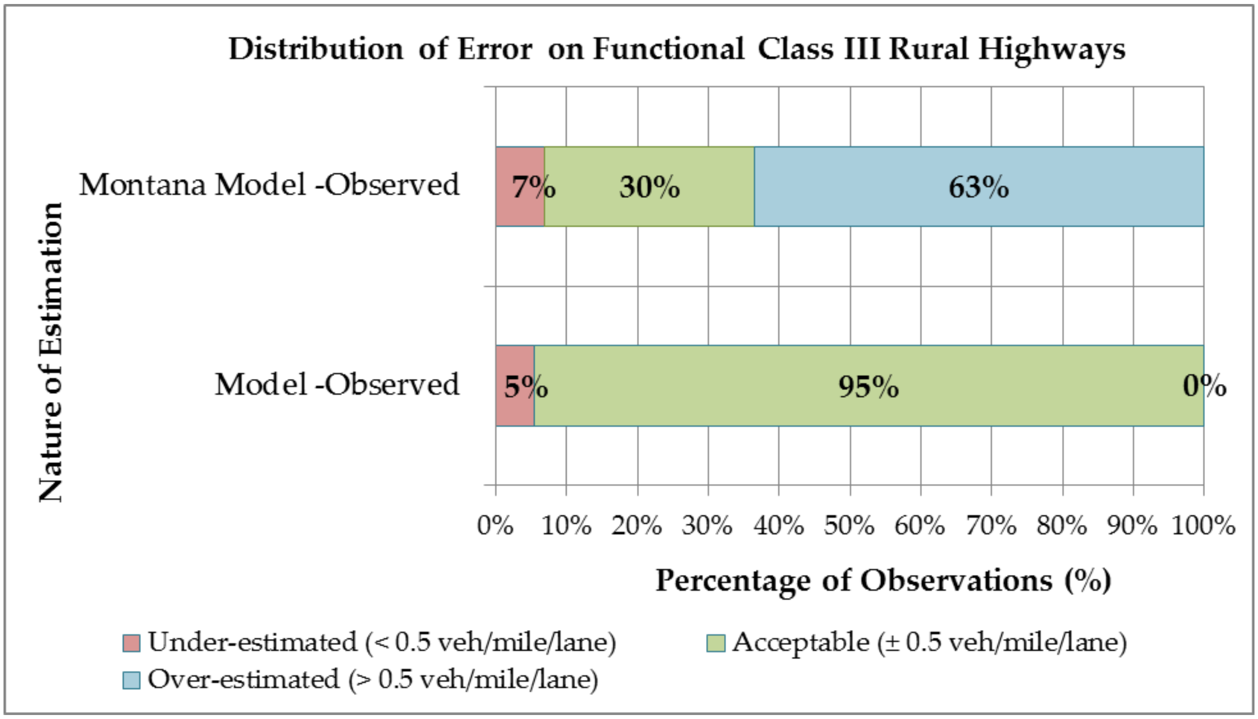


Figure 11. Comparison of Developed Model and Montana Model for Class III Highways

Validation clearly showed follower density models have the potential to be used as an alternative performance measure on two-lane rural highways.

Development of Follower Density Thresholds

Follower density acts as a surrogate measure to assess operations of rural two-lane highways. To leverage the potential of follower density measure, development of follower density thresholds corresponding to each LOS category was necessary. The follower density models helped to develop thresholds at each LOS category by two-lane highway class. According to the HCM 2010 manual, LOS on Class I two-lane highways considers both ATS and PTSF measures. On Class II highways, PTSF governs LOS. On Class III highways Percent of Free-flow Speed (PFFS) is used to define LOS. The HCM 2010 LOS criteria for two-lane highways are shown in Table 3.

Table 3. LOS Criteria for Two-Lane Highways

LOS	Class I Highways		Class II Highways	Class III Highways
	ATS (mi/h)	PTSF (%)	PTSF (%)	PFFS (%)
A	>55	≤35	≤40	>91.7
B	>50–55	>35–50	>40–55	>83.3–91.7
C	>45–50	>50–65	>55–70	>75.0–83.3
D	>40–45	>65–80	>70–85	>66.7–75.0
E	≤40	>80	>85	≤66.7

Source: HCM, 2010, Exhibit 15-3

Both ATS and PFFS measures are obtainable from field data. However, PTSF is difficult to obtain from field. This study uses PTSF LOS boundaries to define follower density thresholds. The procedure to determine the thresholds as follows:

Step 1. Use the relationship between PTSF and volume given in the HCM manual (HCM 2010, Equation 15-10) to develop corresponding volumes at each LOS boundary

Step 2. Develop relationship between volume and follower density from data used for model development

Step 3. With the help of the boundary volumes found in the step 1, designate the follower density thresholds using the relationship obtained from the step 2.

LOS Criteria for Class I Rural Two-Lane Highways

Step 1

According to the HCM (2010), base PTSF is calculated as: $BPTSF_d = 100\left[1-\exp\left(av_d^b\right)\right]$

Where

$BPTSF_d$ is base percent time-spent-following in the analysis direction,

V_d is the demand flow rate in the analysis direction, and

a and *b* are constants.

PTSF ranges as per the HCM (2010) LOS criteria were taken from Table 3. For a given PTSF value at each LOS category, possible volume ranges were calculated (see Table 4).

Table 4. Volume Range at each LOS Category for Class I Rural Two-Lane Highways

Opposing Volume (veh/hr)	a	b	LOS A		LOS B		LOS C		LOS D	
			PTSF (%)	VOL (veh/hr)	PTSF (%)	VOL (veh/hr)	PTSF (%)	VOL (veh/hr)	PTSF (%)	VOL (veh/hr)
200	-0.0014	0.973	35	361	50	589	65	902	80	1398
400	-0.0022	0.923	35	305	50	510	65	799	80	1269
600	-0.0033	0.87	35	271	50	468	65	753	80	1230
800	-0.0045	0.833	35	239	50	423	65	697	80	1163
1000	-0.0049	0.829	35	222	50	393	65	649	80	1086
1200	-0.0054	0.825	35	202	50	360	65	595	80	998
1400	-0.0058	0.821	35	190	50	340	65	563	80	947
1600	-0.0062	0.817	35	180	50	322	65	535	80	902

Step 2

For the collected data, scattered diagram between volume and follower density showed an increasing trend of follower density with the volume (Figure 14). Linear relationship, *follower density = 0.0064 (volume) - 0.138* ($R^2 = 0.81$), was found to be statistically significant.

Step 3

Substituting volume ranges from Table 4 in the above mentioned model (in step 2), follower density ranges were obtained (Table 5).

Table 5. Follower Density Ranges for a given LOS Category for Class I Two-Lane Highways

Opposing Volume (veh/hr)	LOS A		LOS B		LOS C		LOS D	
	VOL	FD	VOL	FD	VOL	FD	VOL	FD
200	361	2.2	589	3.6	902	5.6	1398	8.8
400	305	1.8	510	3.1	799	5.0	1269	8.0
600	271	1.6	468	2.9	753	4.7	1230	7.7
800	239	1.4	423	2.6	697	4.3	1163	7.3
1000	222	1.3	393	2.4	649	4.0	1086	6.8
1200	202	1.2	360	2.2	595	3.7	998	6.2
1400	190	1.1	340	2.0	563	3.5	947	5.9
1600	180	1.0	322	1.9	535	3.3	902	5.6

Note: VOL stands for demand flow rate in veh/hr; FD stands for follower density in veh/mile/lane

The follower densities (in Table 5) are consolidated to get final follower density thresholds as shown in Table 6.

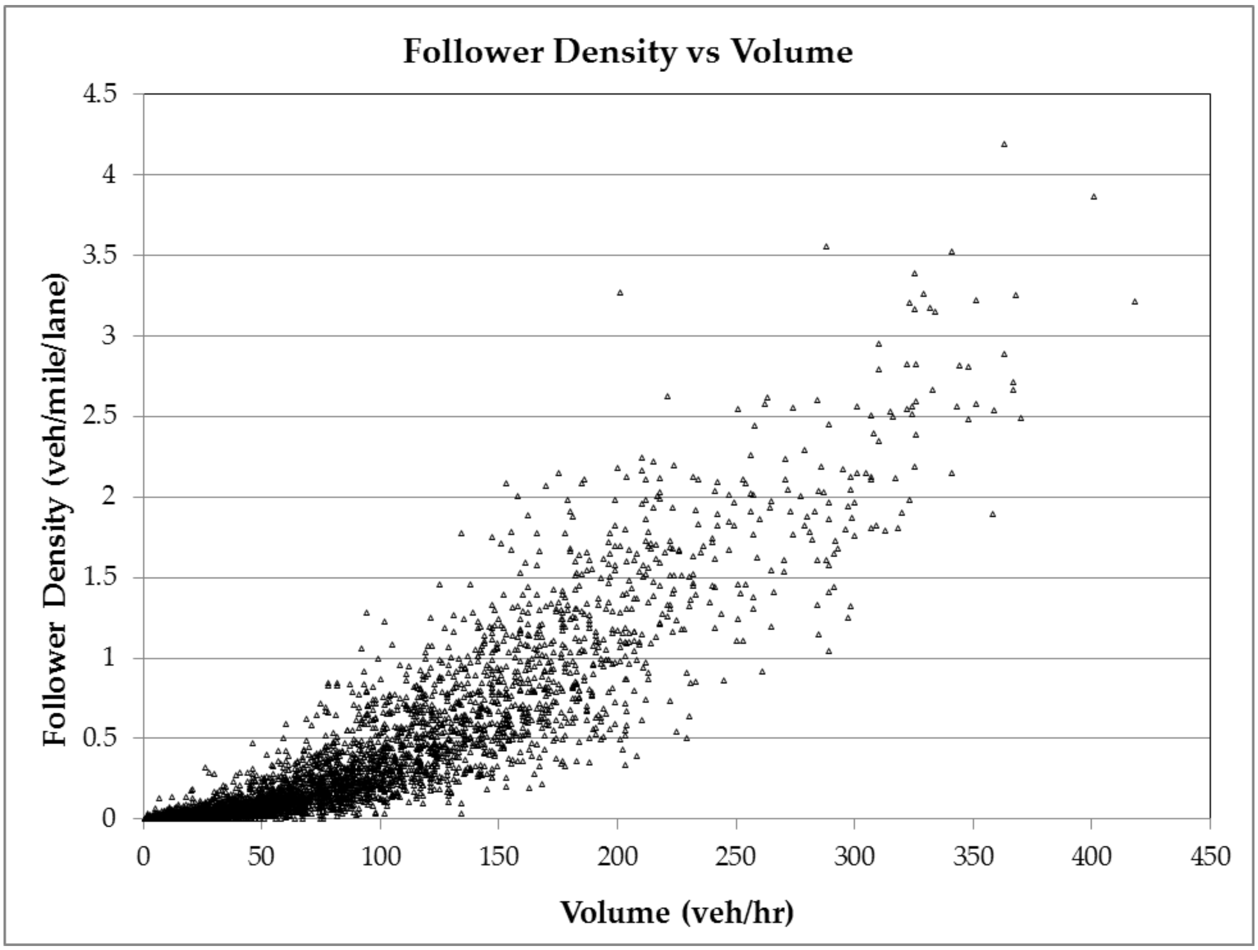


Figure 12. Scattered Plot between Follower Density and Volume on Class I Highways

Table 6. LOS Criteria for Class I Highways (Rural Principal Arterials)

HCM 2010, PTSF Range	HCM 2010, LOS	Suggested Follower Density (veh/mile/lane)
<= 35	A	<= 2.0
> 35 - 50	B	> 2.0 - 3.5
> 50 - 65	C	> 3.5 - 6.0
> 65 - 80	D	> 6.0 - 9.0
> 80	E	> 9.0

LOS Criteria for Class II Rural Two-Lane Highways

Step 1

PTSF values from Table 3 were used for Class II two-lane highways to get the LOS thresholds. For a given PTSF value to each LOS category, corresponding range of possible volumes was calculated (Table 7).

Step 2

Volume and follower density scatter diagram showed that follower density increases with the volume (Figure 15). Linear relationship, *follower density = 0.006 (volume) - 0.124* (R^2 = 0.74), was found to be statistically significant.

Step 3

Using the above relationship, follower density ranges were developed (see Table 8). Follower density ranges were given in the Table 9.

Table 7. Volume Range at each LOS Category for Class II Rural Two-Lane Highways

Opposing Volume (veh/hr)	a	b	LOS A		LOS B		LOS C		LOS D	
			PTSF (%)	VOL (veh/hr)	PTSF (%)	VOL (veh/hr)	PTSF (%)	VOL (veh/hr)	PTSF (%)	VOL (veh/hr)
200	-0.0014	0.973	40	430	55	681	70	1038	85	1656
400	-0.0022	0.923	40	366	55	594	70	927	85	1516
600	-0.0033	0.870	40	329	55	550	70	881	85	1486
800	-0.0045	0.833	40	294	55	502	70	821	85	1417
1000	-0.0049	0.829	40	272	55	466	70	765	85	1324
1200	-0.0054	0.825	40	249	55	427	70	702	85	1219
1400	-0.0058	0.821	40	234	55	403	70	665	85	1156
1600	-0.0062	0.817	40	222	55	383	70	633	85	1103

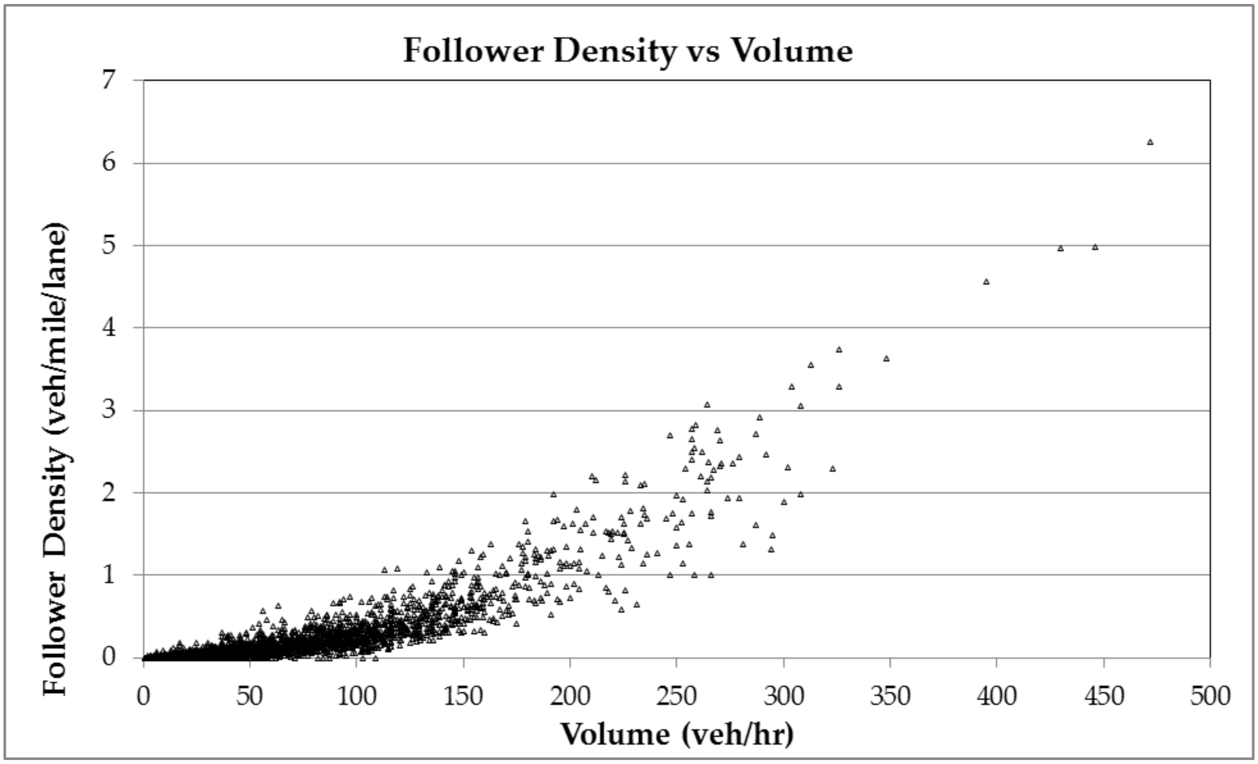


Figure 13. Scattered Plot between Follower Density and Volume on Class II Highways

Table 8. Follower Density Ranges for a given LOS Category for Class II Two-Lane Highways

Opposing Volume (veh/hr)	a	b	LOS A		LOS B		LOS C		LOS D	
			VOL (veh/hr)	FD	VOL (veh/hr)	FD	VOL (veh/hr)	FD	VOL (veh/hr)	FD
200	-0.0014	0.973	430	2.5	681	4.0	1038	6.1	1656	9.8
400	-0.0022	0.923	366	2.1	594	3.4	927	5.4	1516	9.0
600	-0.0033	0.870	329	1.9	550	3.2	881	5.2	1486	8.8
800	-0.0045	0.833	294	1.6	502	2.9	821	4.8	1417	8.4
1000	-0.0049	0.829	272	1.5	466	2.7	765	4.5	1324	7.8
1200	-0.0054	0.825	249	1.4	427	2.4	702	4.1	1219	7.2
1400	-0.0058	0.821	234	1.3	403	2.3	665	3.9	1156	6.8
1600	-0.0062	0.817	222	1.2	383	2.2	633	3.7	1103	6.5

Note: VOL stands for demand flow rate in veh/hr; FD stands for follower density in veh/mile/lane

Table 9. LOS Criteria for Class II Highways (Rural Minor Arterials)

HCM 2010, PTSF Range	HCM 2010, LOS	Follower Density (veh/mile/lane)
<= 40	A	< = 2.5
> 40 - 55	B	> 2.5 - 4.0
> 55 - 70	C	> 4.0 - 6.5
> 70 - 85	D	> 6.5 - 10.0
> 85	E	> 10.0

Sample size from Class III highways was very limited. In addition, Class III highways use PFFS as a LOS measure that can be obtained from field data. Hence, follower density thresholds were not developed for Class III highways. Until refinement, users are advised to use the HCM 2010 methodology and LOS criteria for rural major collector highways.

Summary

The study adopted alternative performance measures to analyze operations on two-lane rural highways. Performance indicators like average travel speed, percent followers, and follower density were tested on data collected from 168 rural highway sites. Datasets covers rural principal arterial, rural minor arterials, and rural major collector highways class. These highway classes are re-designated as per the HCM (2010) definition of class I, class II, and class III two-lane highways.

For each class of two-lane highways, regression models were developed between performance indicators as dependent variables, and the platooning variables, such as traffic flow in the direction of travel, opposing traffic flow, percent heavy vehicles, percent no-passing zones, and terrain as independent variables. Out of various combinations, the model with follower density versus traffic flow, opposing volume, percent of heavy vehicles, percent no passing zones and terrain yields better statistical significance. Model forms by two-lane highway class are shown in Table 10.

Later, data from 58 sites were used to validate the model. A follower density difference of ± 0.5 vehicle/mile/lane was used as the acceptable range of error between model and observations. Data on percent of acceptable, under-estimated and over-estimated observations facilitated models comparison. On an average, 95 percent of observations had an acceptable range of error with the developed models. Model from Montana study is over predicting the follower densities with only 30 percent of observations showing acceptable range of error.

The study also outlined a procedure to develop follower density thresholds. The HCM 2010 manual PTSF boundaries related to Class I and II two-lane highways were used to designate follower density thresholds.

Table 10. Follower Density Models by Rural Two-Lane Highway Functional Class

Functional Class	Model Form	R^2
Class I Highways	Follower Density = -0.1917 + 0.005953 (Traffic Volume) + 0.0005167 (Opposing Volume) + 0.0006739 (% Heavy Vehicles) + 0.0002392 (% No Passing) + 0.05248 (Rolling Terrain)	0.81
Class II Highways	Follower Density = -0.1784 + 0.006189 (Traffic Volume) - 0.0001607 (Opposing Volume) + 0.0006163 (%Heavy Vehicles) + 0.0006055 (% No Passing) + 0.0168 (Rolling Terrain) + 0.03994 (Mountainous Terrain)	0.75
Class III Highways	Follower Density = -0.04062 + 0.003244 (Traffic Volume) - 0.0003219 (Opposing Volume) + 0.0001127 (%Heavy Vehicles) + 0.0001877 (% No Passing) - 0.007543 (Rolling Terrain) - 0.01995 (Mountainous Terrain)	0.74

The study did not set up follower density boundaries for class III highways due to limited sample size. Until further refinement, use of HCM 2010 methodology for class III highways is recommended. New LOS criteria for two-lane rural highways are (except for class III two-lane highways) shown in Table 11.

Table 11. LOS Criteria by Rural Two-Lane Highway Functional Class

LOS	Class I Highways	Class II Highways
	Follower Density (veh/mile/lane)	Follower Density (veh/mile/lane)
A	<= 2.0	<= 2.5
B	> 2.0 - 3.5	> 2.5 - 4.0
C	> 3.5 - 6.0	> 4.0 - 6.5
D	> 6.0 - 9.0	> 6.5 - 10.0
E	> 9.0	> 10.0

Scope for Future Work

Follower density models can be used as a two-lane highways network analysis tool. Using the models and readily available data from HERS (Highway Economic Requirement Systems), follower densities (thereby LOS) can be mapped on each highway network section. Percent miles by LOS category will play a key role in strategic investment, operations and maintenance decisions. However Class III highway models need refinement by obtaining more data. In addition, data expansion to sites with higher directional and opposite traffic flows, and sites located in the mountainous terrain may enhance modelling outcomes.

References

Al-Kaisy, A., and Karjala, S. (2008). Indicators of Performance on Two-Lane Rural Highways: Empirical Investigation, Transportation Research Record, No. 2071, pp. 87–97.

Bessa Jr., J.E., and Settia, J. R. (2011). Derivation of ATS and PTSF Functions for Two-lane Rural Highways in Brazil, Procedia Social and Behavioral Sciences 16, pp.282–292.

Brilon, W., and Weiser, F. (1998). Capacity and Speed-Flow Relationships On Rural Two-Lane Highways in Germany, Transportation Research Circular: Third International Symposium on Highway Capacity, p. 199-218.

Brilon, W., and Weiser, F. (2006). Two-lane Rural Highways: the German Experience, Transportation Research Record: Journal of the Transportation Research Board, v. 1988, p. 38-47.

Cohen, M., and Polus, A. (2009). Theoretical and Empirical Relationships for the Quality of Flow and for a New Level of Service on Two-Lane Highways, Journal of Transportation Engineering, Vol. 135, No. 6.

Cohen, M., and Polus, A. (2011). Estimating Percent-Time-Spent-Following On Two-Lane Rural Highways. Transportation Research Part C, vol. 19, pp. 1319–1325.

Dixon, M. P., Sarepali, S. S. K., and Young, K. A. (2002). Field Evaluation of Highway Capacity Manual 2000 Analysis Procedures for Two-Lane Highways. Journal of the Transportation Research Board, No. 1802, pp. 125–132.

Farah, H., and Toledo, T. (2010). Passing Behavior on Two-Lane Highways, Transportation Research Part F, vol.13, pp. 355–364.

Federal Highway Administration (FHWA). (2013). Highway Functional Classification: Concepts, Criteria and Procedures: 2013 Edition. Accessed on January 28, 2014 from https://www.fhwa.dot.gov/planning/processes/statewide/related/highway_functional_classifications/fcauab.pdf

Hashim, I. H., and Abdel-Wahed, T.A. (2011). Evaluation of Performance Measures For Rural Two-Lane Roads in Egypt, Alexandria Engineering Journal, Vol.50, pp.245–255.

Highway Capacity Manual. (2010). Transportation Research Board, National Research Council, Washington, D.C.

Ibrahima., M. N., Puana, O. C., and Mustaffara, M. (2013). Review of Percent Time Spent Following (PTSF) as Performance Measure for Two-lane Highways, pp.9–18. Accessed on January 29, 2014 from www.jurnalteknologi.utm.my

Laval, J. A. (2006). A macroscopic theory of two-lane rural roads. Transportation Research Part B, vol.40, pp. 937–944.

Li, J., and Washburn, S.S. (2011). Implementing Two-lane Highway Simulation Modeling into CORSIM1, Procedia Social and Behavioral Sciences 16, pp. 293–305.

Luttinen, R. T. (2001). Percent Time-Spent-Following as Performance Measure for Two-Lane Highways. Journal of the Transportation Research Board, No. 1776, pp. 52–59.

Morrall, J. F., and Werner, A. (1990). Measuring Level of Service of Two-Lane Highways by Overtakings. Transportation Research Record, No. 1287, pp. 62–69.

Newell, G. F. (1998). A Moving Bottleneck. Transportation Research Part B, Vol. 32, No. 8, pp. 531-537.

Romana, M. G., and Pérez, I. (2006). Measures of Effectiveness for Level-of- Service Assessment of Two-Lane Roads: An Alternative Proposal Using a Threshold Speed. Journal of the Transportation Research Board, No. 1988, pp. 56–62.

Van As, C. (2003).The Development of an Analysis Method for the Determination of Level of Service on Two-Lane Undivided Highways in South Africa. South African National Roads Agency, Pretoria.

Vlahogianni, E. I. (2013). Modeling Duration of Overtaking in Two Lane Highways, Transportation Research Part F, vol.20, pp.135–146.

Appendix A: VMT and Mileage Guidelines by Functional Class

Table 3-5: VMT and Mileage Guidelines by Functional Classifications - Arterials

	Arterials			
	Interstate	Other Freeways & Expressway	Other Principal Arterial	Minor Arterial
Typical Characteristics				
Lane Width	12 feet	11 - 12 feet	11 - 12 feet	10 feet - 12 feet
Inside Shoulder Width	4 feet - 12 feet	0 feet - 6 feet	0 feet	0 feet
Outside Shoulder Width	10 feet - 12 feet	8 feet - 12 feet	8 feet - 12 feet	4 feet - 8 feet
AADT[1] (Rural)	12,000 - 34,000	4,000 - 18,500[2]	2,000 - 8,500[2]	1,500 - 6,000
AADT[1] (Urban)	35,000 - 129,000	13,000 - 55,000[2]	7,000 – 27,000[2]	3,000 - 14,000
Divided/Undivided	Divided	Undivided/Divided	Undivided/Divided	Undivided
Access	Fully Controlled	Partially/Fully Controlled	Partially/Uncontrolled	Uncontrolled
Mileage/VMT Extent (Percentage Ranges)[1]				
Rural System				
Mileage Extent for Rural States[2]	1% - 3%	0% - 2%	2% - 6%	2% - 6%
Mileage Extent for Urban States	1% - 2%	0% - 2%	2% - 5%	3% - 7%
Mileage Extent for All States	1% - 2%	0% - 2%	2% - 6%	3% - 7%
VMT Extent for Rural States[2]	18% - 38%	0% - 7%	15% - 31%	9% - 20%
VMT Extent for Urban States	18% - 34%	0% - 8%	12% - 29%	12% - 19%
VMT Extent for All States	20% - 38%	0% - 8%	14% - 30%	11% - 20%
Urban System				
Mileage Extent for Rural States[2]	1% - 3%	0% - 2%	4% - 9%	7% - 14%
Mileage Extent for Urban States	1% - 2%	0% - 2%	4% - 5%	7% - 12%
Mileage Extent for All States	1% - 3%	0% - 2%	4% - 5%	7% - 114%
VMT Extent for Rural States[2]	17% - 31%	0% - 12%	16% - 33%	14% - 27%
VMT Extent for Urban States	17% - 30%	3% - 18%	17% - 29%	15% - 22%
VMT Extent for All States	17% - 31%	0% - 17%	16% - 31%	14% - 25%
Qualitative Description (Urban)	• Serve major activity centers, highest traffic volume corridors, and longest trip demands • Carry high proportion of total urban travel on minimum of mileage • Interconnect and provide continuity for major rural corridors to accommodate trips entering and leaving urban area and movements through the urban area • Serve demand for intra-area travel between the central business district and outlying residential areas			• Interconnect with and augment the principal arterials • Serve trips of moderate length at a somewhat lower level of travel mobility than principal arterials • Distribute traffic to smaller geographic areas than those served by principal arterials • Provide more land access than principal arterials without penetrating identifiable neighborhoods • Provide urban connections for rural collectors
Qualitative Description (Rural)	• Serve corridor movements having trip length and travel density characteristics indicative of substantial statewide or interstate travel • Serve all or nearly all urbanized areas and a large majority of urban clusters areas with 25,000 and over population • Provide an integrated network of continuous routes without stub connections (dead ends)			• Link cities and larger towns (and other major destinations such as resorts capable of attracting travel over long distances) and form an integrated network providing interstate and inter-county service • Spaced at intervals, consistent with population density, so that all developed areas within the State are within a reasonable distance of an arterial roadway Provide service to corridors with trip lengths and travel density greater than those served by rural collectors and local roads and with relatively high travel speeds and minimum interference to through movement

1- *Ranges in this table are derived from 2011 HPMS data.*
2- *For this table, Rural States are defined as those with a maximum of 75 percent of their population in urban centers.*

Source: FHWA, 2013

Figure A1. VMT and Mileage Guidelines for Arterial Highways

Table 3-6: VMT and Mileage Guidelines by Functional Classifications – Collectors and Locals

	Collectors		Local
	Major Collector[2]	Minor Collector[2]	
Typical Characteristics			
Lane Width	10 feet - 12 feet	10 - 11 feet	8 feet - 10 feet
Inside Shoulder Width	0 feet	0 feet	0 feet
Outside Shoulder Width	1 feet - 6 feet	1 feet - 4 feet	0 feet - 2 feet
AADT[1] (Rural)	300 - 2,600	150 - 1,110	15 - 400
AADT[1] (Urban)	1,100 - 6,300[2]		80 - 700
Divided/Undivided	Undivided	Undivided	Undivided
Access	Uncontrolled	Uncontrolled	Uncontrolled
Mileage/VMT Extent (Percentage Ranges)[1]			
Rural System			
Mileage Extent for Rural States[3]	8% - 19%	3% - 15%	62% - 74%
Mileage Extent for Urban States	10% - 17%	5% - 13%	66% - 74%
Mileage Extent for All States	9% - 19%	4% - 15%	64% - 75%
VMT Extent for Rural States[3]	10% - 23%	1% - 8%	8% - 23%
VMT Extent for Urban States	12% - 24%	3% - 10%	7% - 20%
VMT Extent for All States	12% - 23%	2% - 9%	8% - 23%
Urban System			
Mileage Extent for Rural States[3]	3% - 16%	3% - 16%[2]	62% - 74%
Mileage Extent for Urban States	7% - 13%	7% - 13%[2]	67% - 76%
Mileage Extent for All States	7% - 15%	7% - 15%[2]	63% - 75%
VMT Extent for Rural States[3]	2% - 13%	2% - 12%[2]	9% - 25%
VMT Extent for Urban States	7% - 13%	7% - 13%[2]	6% - 24%
VMT Extent for All States	5% - 13%	5% - 13%[2]	6% - 25%
Qualitative Description (Urban)	• Serve both land access and traffic circulation in higher density residential, and commercial/industrial areas • Penetrate residential neighborhoods, often for significant distances • Distribute and channel trips between local streets and arterials, usually over a distance of greater than three-quarters of a mile	• Serve both land access and traffic circulation in lower density residential, and commercial/industrial areas • Penetrate residential neighborhoods, often only for a short distance • Distribute and channel trips between local streets and arterials, usually over a distance of less than three-quarters of a mile	• Provide direct access to adjacent land • Provide access to higher systems • Carry no through traffic movement
Qualitative Description (Rural)	• Provide service to any county seat not on an arterial route, to the larger towns not directly served by the higher systems, and to other traffic generators of equivalent intra-county importance such as consolidated schools, shipping points, county parks, important mining and agricultural areas • Link these places with nearby larger towns and cities or with arterial routes • Serve the most important intra-county travel corridors	• Be spaced at intervals, consistent with population density, to collect traffic from local roads and bring all developed areas within reasonable distance of a minor collector • Provide service to smaller communities not served by a higher class facility • Link locally important traffic generators with their rural hinterlands	• Serve primarily to provide access to adjacent land • Provide service to travel over short distances as compared to higher classification categories • Constitute the mileage not classified as part of the arterial and collectors systems

1- *Ranges in this table are derived from 2011 HPMS data.*
2- *Information for Urban Major and Minor Collectors is approximate, based on a small number of States reporting.*
3- *For this table, Rural States are defined as those with a maximum of 75 percent of their population in urban centers.*

Source: FHWA, 2013

Figure A2. VMT and Mileage Guidelines for Collectors and Locals Highways

Development of Queue Length Models at Two-way STOP Controlled Intersection:

A Surrogate Method

Facility Analysis and Simulation Team
Transportation Development Division
Oregon Department of Transportation

October 2010

Table of Contents

Table of Figures

Table of Tables

Executive Summary

This study aims at developing queue length models at two-way STOP controlled intersections. A significant amount of research on the estimation of capacity, delay, and queue lengths at unsignalized intersection has resulted in a variety of models ranging from empirical to simulation models. Most agencies are following methods like the Two-Minute Rule, Highway Capacity Manual Method, and the Harmelink Curves to estimate queue lengths. But, these methods are yielding inconsistence estimates and questions often arise as to the correct method to use. This study documents the inconsistency among these methods and takes further steps to improve queue length estimates by developing surrogate models.

Data at 15 two-way STOP controlled intersections covering various functional classifications of highways, geometric configurations, and geographic regions were collected by using video tapes. Data was processed to meet the requirements of the methods. Queue length estimations from each method were noted. Later, models were compared for their performance in estimating 95th percentile queue lengths. It was shown that the Highway Capacity Manual method consistently underestimates queues. The two-minute rule estimated fairly closer queue lengths except for major left turn movements, due to not considering opposing volumes. The Harmelink curves are applicable only for major left turns. Queue length estimation equations developed by John T. Gard showed better results, but the variation among observed and estimated queue lengths was still high.

Data processed for comparison was used to estimate the models. First, looking at the data clearly indicated that random phenomena prevail among queue lengths and the associated explanatory variables. Exhaustive statistical analysis was conducted to understand queue behavior on both major and minor approach lane groups. Poisson regression models were fitted to explain the random process. A model comparison showed significantly improved performance of the new models in predicting maximum queue lengths.

Further, field data at 25 intersections was collected covering wide array of condition to test the developed model consistency. More than 70 % of the predicted queue lengths were close to observed queue length estimates. In addition, model sensitivity analysis was performed to assess the stability of the model. When the major and minor approach volumes are within limits of MUTCD signal warrant volumes, acceptable ranges of queue lengths are predicted. Beyond the MUTCD suggested volume ranges, marginal increase in input variable substantiates queue lengths.

This report is organized in seven chapters. Various methods are reviewed in Chapter 1. Problems identified in each method, objectives, scope and methodology are discussed. Chapter 2 is dedicated to data collection and analysis efforts. Chapter 3 compares the methods and highlights their differences. Chapter 4 explains the basic philosophy of developing Poisson regression models. Detailed statistical analysis, including data description, model selection, variable selection and model statistics are included. Data validation steps are presented in chapter 5. Chapter 6 deals with the stability analysis of the model through sensitivity tests. Chapter 7 gives the summary, conclusions, and scope for future study.

1 Introduction

A significant amount of research on the estimation of capacity, delay, and queue lengths at unsignalized intersection has resulted in the development of a variety of models ranging from empirical to simulation. A literature review of all available models is beyond the scope of this project, but unsignalized intersection theory chapter in the Traffic Flow Theory1 gives a good start. In particular, the philosophies behind the methods like, Two-Minute Rule, Highway Capacity Manual Method, the Harmelink Curves, and equations given by John T. Gard are discussed.

This chapter introduces the above mentioned methods briefly. Next, problems associated with each method are explained. This leads to the definition of the problem. Finally, the step-by-step process used for the study is presented.

1.1 Methodologies

1.1.1 Two-Minute Rule[2]

The Two-Minute Rule is a rule of thumb methodology that estimates queue lengths for major street left turns and minor street movements by using the queue that would result from a two-minute stoppage of the turning demand volume. This method does not consider the magnitudes and impacts of the conflicting flows on the size of the queue. The calculation of the 95th percentile queue using the two-minute rule methodology shall use the following equation:

S = (v) (t) (L)

Where:

S = the 95th percentile queue storage length (feet)

v = the average left-turn volume arriving in a 2-minute interval

t = a variable representing the ability to store all vehicles; usually 1.75 to 2.0 (Use Table 1-1)

L = average length of the vehicles being stored and the gap between vehicles; 25 ft. for cars. This value can be increased where a significant number of trucks are present

[1] R.J.Troutbeck., and Brilon, W. "*Unsignalised Intersection Theory*", Traffic Flow Theory, TRB Special Report 165, Washington D.C.

[2] Chapter 7: Intersection Analysis, "*Analysis Procedure Manual* ", Updated: May 2010, Pg:237

Table 1-1 Selection of "t" values (*source: APM*)

Exhibit 7-29 Selection of "t" Values

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

It should also be noted that the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased according to Table 1-2. This adjustment is only for the manual methods; software packages may require a different adjustment.

Table 1- 2 Storage Length Adjustments for Trucks (*source: APM)*

Exhibit 7-30 Storage Length Adjustments for Trucks

Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

While both the nomograph given in the Analysis Procedure Manual and the rule of thumb equation are intended for use in estimating vehicle queue lengths for single-lane left turn movements, the vehicle queue lengths for double left turn lanes can be estimated by dividing the results of these methods by 1.8. This value represents the assumption that queued vehicles will not be evenly distributed between the turn lanes.

1.1.2 Harmelink Curves[3]

M.D. Harmelink , in a paper that was published in 1967, provided the foundation for many current left-turn guidelines. Harmelink based his work on a queuing model in which arrival and service rates are assumed to follow negative exponential distributions. He stated that the probability of a through vehicle arriving behind a stopped, left-turning vehicle should not exceed 0.02 for 40 mph (64 km/h), 0.015 for 50 mph (80 km/h), and 0.01 for 60 mph (96 km/h). He presented his criteria in the form of graphs, 18 in all. To use his graphs, the advancing volume, opposing volume, operating speed, and left-turn percentage

[3] M.D.Harmelink, "*Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections*", Highway Research Record 211, 1967

need to be known. Graphs for speeds of 40, 50, and 60 mph (64, 80, and 96 km/h) are given, as well as 5, 10, 15, 20, 30, and 40 percent left-turn volumes.

1.1.3 Highway Capacity Manual[4]

HCM 2000 relies on refined models developed in Germany based on both gap acceptance and empirical models which describe the interaction of the minor or stop controlled approach with drivers on the major street. The following figure shows the computational steps to calculate the queue lengths for two-way stop controlled approaches.

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\,\right]\left(\frac{c_{m,x}}{3600}\right) \qquad (17\text{-}37)$$

where

Q_{95} = 95th-percentile queue (veh),
v_x = flow rate for movement x (veh/h),
$c_{m,x}$ = capacity of movement x (veh/h), and
T = analysis time period (h) (T = 0.25 for a 15-min period).

95th percentile queue lengths are calculated by the above equation 17-37 of the HCM. For varying volume-to-capacity ratios, expected maximum number of vehicles in queue are obtained from Figure 1-1.

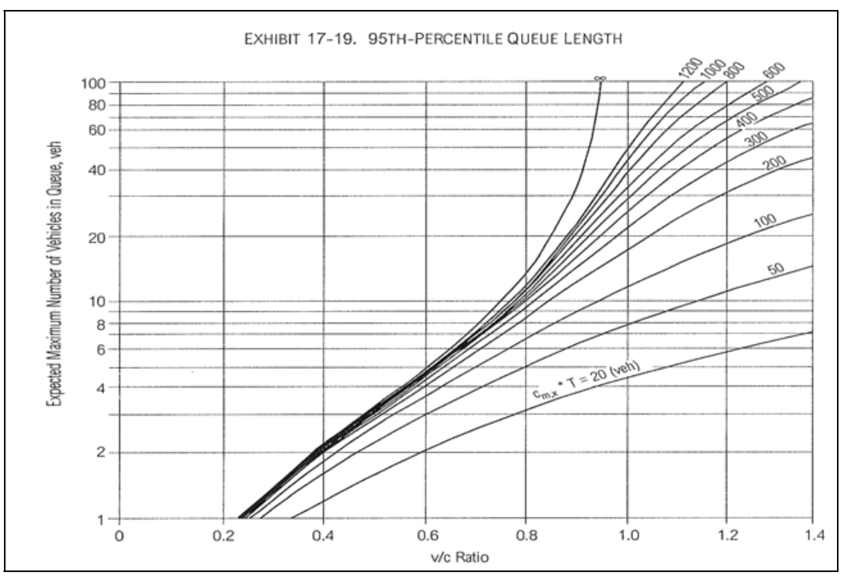


Figure 1-1 Expected maximum number of vehicles in Queue by HCM

---

[4] "Chapter 17- Unsignalized intersections", *2000 Highway Capacity Manual*, Transportation Research Board, Washington, D.C.

1.1.4 Gard's Equation[5]

John T. Gard developed regression equations for the prediction of queue lengths for major-street left turn, minor street left turn, minor street right turn, and minor street shared left/through/right turn configurations through a study of 15 unsignalized intersections in Sacramento, CA. Queue length represents the maximum number of vehicles in the queue. Table 1-3 describes the study intersections and Table 1-4 gives a summary of regression equations. R^2 values of the equations vary from 0.65 to 0.80.

Table 1-3 Description of Gard's Study Intersections[5]

Table 1. Description of study intersections.

	Major street				Minor street	
Intersection	Roadway type	Posted speed limit	Traffic signal within one-quarter mile?	Two-way left-turn lane?	Roadway type	Lane configuration
1	2-lane collector	30 mph	Yes	No	2-lane collector	Exclusive left & right lanes
2	2-lane arterial	45 mph	Yes	Yes	2-lane collector	Shared left/right lane
3	2-lane arterial	55 mph	No	No	2-lane arterial	Shared left/right lane
4	2-lane arterial	50 mph	No	No	2-lane arterial	Exclusive left & right lanes
5	2-lane arterial	45 mph	No	No	2-lane collector	Shared left/right lane
6	2-lane highway	55 mph	No	No	2-lane collector	Shared left/right lane
7	2-lane highway	50 mph	No	No	2-lane arterial	Exclusive left & right lanes
8	4-lane arterial	45 mph	Yes	No	Hospital driveway	Exclusive left/through & right lanes
9	4-lane arterial	45 mph	Yes	No	Shopping center driveway	Shared left/right lane
10	4-lane arterial	40 mph	Yes	Yes	Office complex driveway	Exclusive left/through & right lanes
11	4-lane arterial	45 mph	No	Yes	2-lane collector	Exclusive left & right lanes
12	4-lane arterial	45 mph	Yes	Yes	Shopping center driveway	Exclusive left & right lanes
13	4-lane arterial	45 mph	Yes	Yes	2-lane collector	Shared left/through & exclusive right lanes
14	6-lane arterial	35 mph	Yes	No	Shopping center driveway	Exclusive right lane
15	6-lane arterial	45 mph	Yes	No	Shopping center driveway	Exclusive right lane

Note: All turning movements are permitted at intersections 1–13. Left-turn ingress and right-turn movements are permitted at intersections 14 and 15.

Table 1-4 Gard's Regression Equations[5]

Table 4. Regression equations.

Movement	Condition	Equation
Major-street left turn	Approach volume ≤ 100 VPH/PHF	Max. Queue = –2.042 + 1.167 ln(AppVol) + 0.975*TS
	Approach volume > 100 VPH/PHF	Max. Queue = +4.252 – 1.23*Lanes + 0.07996*Speed + 1.412*TS – 374.028/AppVol + 0.00001144*AppVol *ConflVol
Minor-street left turn	Approach volume ≤ 60 VPH/PHF	Max. Queue = +0.958 + 0.00111*(AppVol)^2 + 0.000333* (ConflVol)
	Approach volume > 60 VPH/PHF	Max. Queue = +6.174 – 2.313*TS + 0.03307*Speed – 1201.644/ConflVol + 0.00006549 (AppVol)^2
Minor-street right turn	Approach volume ≤ 100 VPH/PHF	Max. Queue = –19.822 + 0.688ln(AppVol) + 1.886*TS + 0.369*(Lanes)^2 + 0.000000288* (ConflVol) ^2 + 0.401*Speed
	Approach volume > 100 VPH/PHF	Max. Queue = –26.23 + 0.132*Speed + 0.000000603* (ConflVol) ^2 + 4.909ln(AppVol)
Minor-street shared left/through/right	All conditions	Max. Queue = –12.916 + 3.225ln(AppVol) +0.00569*(ConflVol for LTs & THs) – 0.000177*(ConflVol for RTs) – 2.109*(RT %) – 3.157*TS

Where

---

[5] John T. Gard. "*Estimation of Maximum Queue Lengths at Unsignalized Intersection*", ITE Journal, November 2001, Pg: 26-34

AppVol = hourly traffic volume divided by peak-hour factor (PHF) for subject movement

ConflVol = hourly traffic volume divided by PHF that conflicts with subject movement (refer to the Highway Capacity Manual to identify movements that conflict with subject approach)

TS = a dummy variable with a value of 1 if a traffic signal is located on the major street within one-quarter mile of the subject intersection and 0 otherwise

Lanes = number of through lanes occupied by conflicting traffic

Speed = posted speed limit on major street (in miles per hour)

RT % = Percentage of vehicles on shared left/through/right minor street approach that turn right

In his comparison, Gard found that the 2000 HCM method showed a tendency to underestimate the queues. The Two-Minute Rule was successfully predicting 8 out 10 cases, with in one vehicle variation. According to the author, in 49 out of 51 comparisons, the regression equation provided maximum queue-length estimates that were as accurate as or more accurate than other methods used in this study.

1.2 Problem

The problem for this work is defined as reasonably estimating the 95^{th} percentile queue length, the length of the queue that has a probability of 5 percent or less of being exceeded during the peak hour, which is critical to the operational success and safety of the intersection.

1.3 Purpose

This study describes the maximum queue length model development and validation for two-way stop control. It further checks the consistency of queue length predictions among the widely used methods.

1.4 Study Methodology

The first step in the development of a model is data collection. Data collection requires prior effort in the form of: identifying the parameters influencing queue behavior, checking the sample size, location, season and duration of data collection. After collection, data is processed to get the required inputs. Then, model comparison is performed to identify the deficiencies among the existing models. This leads to model formulation and statistical analysis of data to predicted the maximum number of vehicles in the stopped queue. Next, model validation is conducted to check the model accuracy. Finally, sensitivity analysis will identify the limits to the value of input variables. The step-by-step procedure is shown in Figure 1-2.

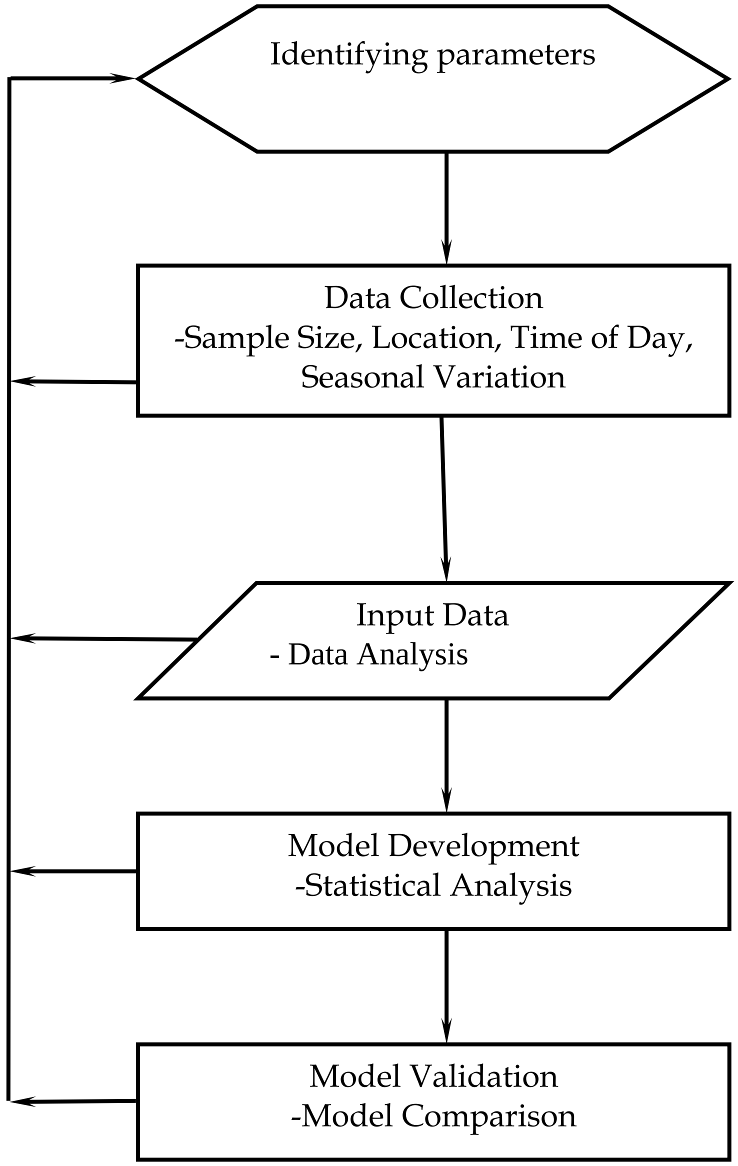


Figure 1-2 Study Methodology

1.5 Summary

This chapter briefly explains the current adopted methods to estimate the queue lengths at two-way STOP controlled intersections. Not every model is representing a true condition, so this study was directed to get a closer solution to the problem. The methodology explained above outlines the study process.

2 Data Collection & Analysis

Data plays a key role in model development, validation and comparing competitive models for consistency. This chapter focuses on the data collection procedure, data synthesis, and preparation of data sets for various lane groups. Data collection technologies are not discussed here.

2.1 *Data collection*

Intersections were chosen to cover a range of lane configurations, geographic regions, functional classifications, and traffic conditions. In total, 15 intersections shown in Appendix B, Table B1 were used for data collection.

Out of 15 intersections: 7 of are from region 2, 5 are from region 3, one from region 4, and the remaining 2 from region 5. 10 (67%) are within the urban growth boundary, and the remainder 5 (33%) are rural. 13 intersections have either OR or US route as the major approach. Three of the intersections have an upstream signal within 1000 ft. Six intersections have either an exclusive or two-way left turn lane. More than half of the intersections, totaling 9, have skewed approaches. Three intersection approaches are off-set from the major approach. Nearly half of the intersections are 3 legged (7 or 47%). Finally, three intersections major approaches have flaredness.

Appendix B, Table B2 represents the time frame of the data collection. All the data collected belong to either year 2010 or year 2009. Where the data is available, both AM and PM peak periods are covered.

2.2 *Data Analysis*

For each approach, information regarding the geometry, lane groups and associated movements, turn lane information, and traffic volume by movement is noted.

Queue data is collected through video logs provided by the Transportation Data Section. Maximum number of vehicles in the stopped queue is noted for every 15 minute interval for both peak periods by taking the maximum value of the observed queue on each lane group. Traffic volume for the same period is obtained from Traffic Count Management (TCM) program used by Transportation Data Section.

Hourly traffic volume is computed by summing the corresponding four 15 min intervals. A peak hour factor is calculated and applied to obtain hourly flow rates. Time periods having only hourly traffic volume are not considered in the analysis period. In the absence of a calculated peak hour factor, default peak hour factors can be taken to prevent data loss. This was not done due to the importance of the study for model comparison and development in order to emulate actual traffic conditions.

The next step is to calculate the conflicting traffic flow rate according the procedure documented in Highway Capacity Manual. An excerpt from the HCM showing the two-way STOP controlled configurations are given in Appendix A, Figure A1 and the calculation of the associated conflicting flow rates for different lane movements are shown in Appendix A, Figure A2. Conflicting flow rates from individual movements in a lane group are added algebraically to obtain the lane group conflicting movements. Similarly, lane group flow rates are obtained by adding individual lane group flow rates.

This methodology adopted disaggregates approaches where individual intersection approaches and individual lane groups within approaches are treated separately. Both geometry of the intersection and the distribution of traffic movements play a key role in segmenting the intersection into lane groups. The following are the excerpts from HCM[6] related to the definitions of various lane groups:

An exclusive left-turn lane or lanes should normally be designated as a separate lane group unless there is also a shared left-through lane present, in which case the proper lane grouping will depend upon the distribution of traffic volume between the movements. The same is true of an exclusive right-turn lane.

On approaches with exclusive left-turn or right-turn lanes, or both, all other lanes on the approach would generally be included in a single lane group. Some example lane groups according to HCM are given in Appendix A, Figure A3.

The following lane groups are considered:

MNLTR is for a minor street on a four legged intersection having a single lane for left, through and right turn movements

MNLR is for a minor street on a three legged intersection having a single lane for left and right turn movements

MJL is for a major left turn movement irrespective of exclusive/median/TWTL configuration

MNL is for an exclusive left turn lane on a minor approach of either a four or three legged intersection

MNR is for an exclusive right turn lane on a four legged intersection minor approach

---

[6] "Chapter 16- signalized intersections", *2000 Highway Capacity Manual*, Transportation Research Board, Washington, D.C., pg:16-6

The next step is to calculate the hourly observed queue length by taking the maximum of four 15 min interval queue lengths. Estimate queue lengths by the 2 minute rule, HCM methodology, and Gard's equation. This step completes the data set for one intersection on one approach. Repeat the process for all approaches and intersections.

2.3 *Summary*

The data collection methodology is explained in this chapter. Then the step by step process for data analysis and the preparation of input data are given. This step leads to the comparison of existing methodologies to check the consistency among the models.

3 Comparison of Existing Methodologies

The first part of this study is to check the performance of the 2 minute rule, HCM method and Gard's equation for estimating queue lengths. The following sub sections analyze performance of these models for each lane configuration.

3.1 Major Left (MJL)

Summary statistics are given in Table 3-1. Observed queue length varies from a minimum of 1 vehicle to a maximum of 8 vehicles. At lower volumes, for instance a minimum volume of 1 vehicle, all methods tend to give zero queue lengths corresponding to the observed queue length of 1. If the volume reaches the maximum observed volume of 134 vph, the HCM method seems to be insensitive, only yielding a single vehicle in the stopped queue.

Table 3-1 Summary Statistics for MJL

	Observed	Two_min_Rule	HCM_Method	Gards	VOL
Count	219	219	219	219	219
Average	2.58904	0.931507	0.0182648	1.73973	29.9361
Variance	2.61933	0.843911	0.0180135	1.48699	618.436
Standard deviation	1.61843	0.918646	0.134214	1.21942	24.8684
Minimum	1.0	0.0	0.0	0.0	1.0
Maximum	8.0	5.0	1.0	4.0	134.0
Range	7.0	5.0	1.0	4.0	133.0
Stnd. skewness	6.76384	5.8038	43.7694	-0.987539	6.45329
Stnd. kurtosis	3.52603	4.16918	153.913	-3.80584	3.8392

	CONVOL
Count	219
Average	506.95
Variance	60516.9
Standard deviation	246.002
Minimum	2.0
Maximum	972.0
Range	970.0
Stnd. skewness	-0.729985
Stnd. kurtosis	-1.97501

A scatter diagram of both observed and estimated queue lengths of the 219 data points is shown in Figure 3-1. The horizontal axis shows the observation number and the vertical axis represents the model predicted maximum number of vehicles in the stopped queue. Table 3-2 compares the relative performance of each model. The queue length given by Gard's equations for the MJL lane configuration matches 32% of observations, which is highest among these methods. Only 8% are matched by the Two-Minute rule. If the queue length matching criterion is relaxed to predict ± 1 vehicle, 53% are matched by the Two-Minute rule and 76% by Gard's equation. The Two-Minute rule is out-performed here due to the fact that it does not consider the opposing traffic volume.

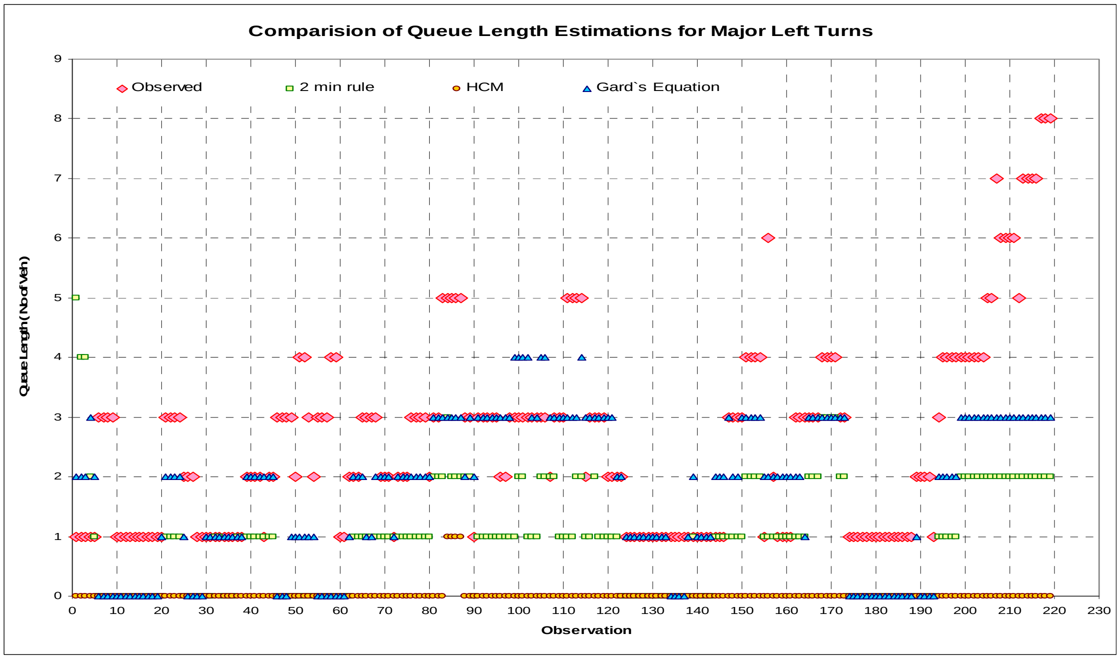


Figure 3-1 Scatter plot of all methods for MJL

Table 3-2 Difference between Observed and Model Outputs for MJL

Type of Estimation	Difference (against Observed)	Observed - 2 min rule		Observed - HCM method		Observed - Gard's Equation	
		% of Obs	Cum. % of Obs	% of Obs	Cum. % of Obs	% of Obs	Cum. % of Obs
Over Estimated	-5	0%	1%	0%	0%	0%	0%
	-4	0%		0%		0%	
	-3	1%		0%		0%	
	-2	0%		0%		0%	
Acceptable	-1	0%	53%	0%	35%	12%	76%
	0	8%		0%		32%	
	1	44%		35%		32%	
Under Estimated	2	26%	46%	16%	65%	11%	24%
	3	11%		28%		7%	
	4	5%		12%		4%	
	5	3%		4%		1%	
	6	1%		2%		0%	
	7	0%		2%		0%	
	8	0%		1%		0%	
	9	0%		0%		0%	
	10	0%		0%		0%	

3.2 *Minor, Share LTR (MNLTR)*

The scatter plot matrix in Figure 3-2 clearly shows that an increase in both volume and conflicting volume has positive impact on the number of vehicles in queue. Table 3-3 shows the summary statistics for each of the selected data variables. It includes measures of central tendency, measures of variability, and measures of shape. As Volume increase, a nearly linear relation is observed between queue length and volume. A similar relation is observed for Gard's equation for increasing conflicting volume. The HCM method yields almost constant queue lengths up to a point, beyond which even a small increment in conflicting volume triggers an exponential increase in queue length.

The scatter plot of queue lengths for different models is shown in Figure 3-2 and Figure 3-3. It seems the Two-Minute Rule relatively closely follows the observed trend. To explore further, the differences between observed queue lengths and predicted queue lengths are tabulated in Table 3-4. The Two-Minute Rule shows 22% of the predictions exactly matched observations. The Two-Minute Rule attains 65% predictability within ± 1 vehicle. For the same instance, 36% are matched by the HCM method.

Table 3-3 Summary Statistics for MJL

	VOL	CONVOL	OBSERVED	Two min Rule	HCM
Count	143	143	143	143	143
Average	80.1119	2159.78	2.55944	2.67832	2.01399
Variance	9690.61	936297.0	3.19186	11.1775	25.2252
Standard deviation	98.4409	967.625	1.78658	3.34327	5.02247
Minimum	3.0	897.0	1.0	0.0	0.0
Maximum	396.0	4016.0	8.0	13.0	21.0
Range	393.0	3119.0	7.0	13.0	21.0
Stnd. skewness	8.73745	3.07341	6.83757	8.43334	12.4561
Stnd. kurtosis	5.4864	-2.00372	3.77464	5.02124	12.931

	Gards
Count	143
Average	8.8951
Variance	57.672
Standard deviation	7.59421
Minimum	0.0
Maximum	24.0
Range	24.0
Stnd. skewness	4.16384
Stnd. kurtosis	-1.2633

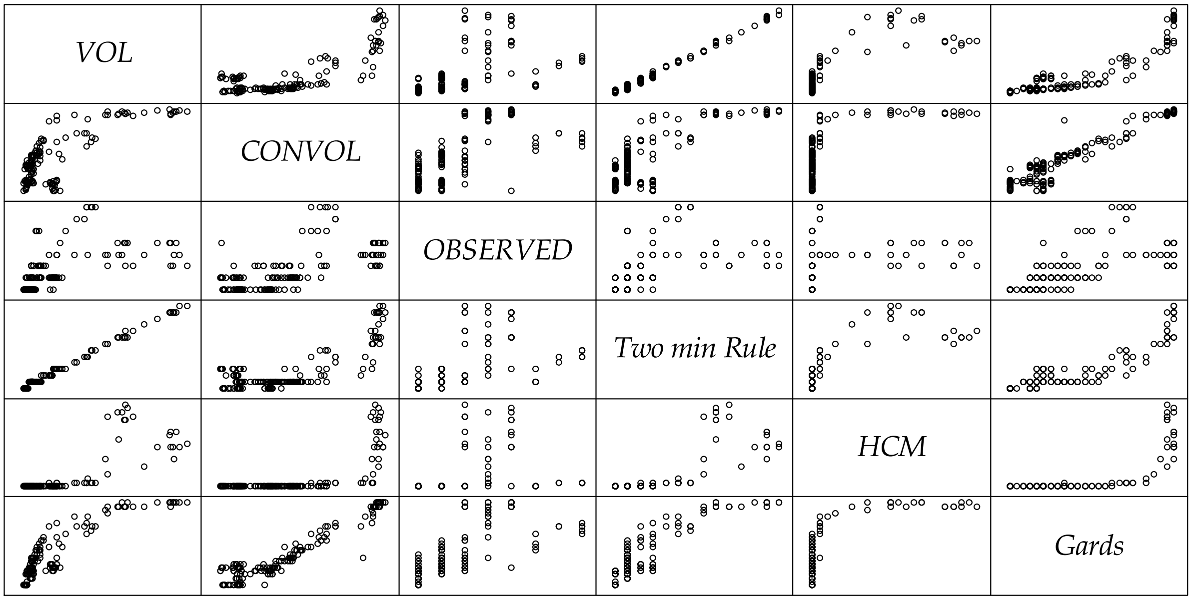


Figure 3-2 Scattered plot of VOL, CONVOL, & Queue Lengths

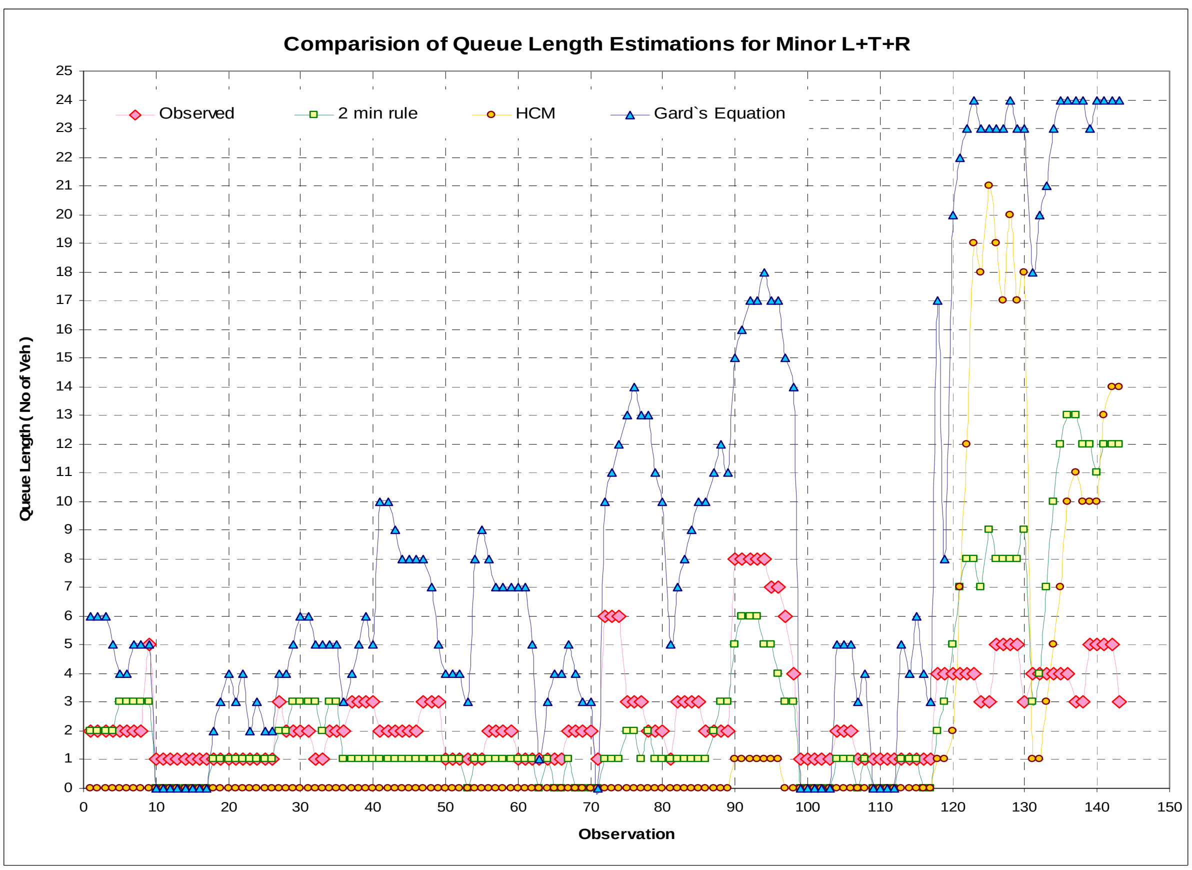


Figure 3-3 Scatter plot of all methods for MNLTR

Table 3-4 Difference between Observed and Model Outputs for MNLTR

Type of Estimation	Difference (against Observed)	Observed - 2 min rule		Observed - HCM method		Observed - Gard's Equation	
		% of Obs	Cum. % of Obs	% of Obs	Cum. % of Obs	% of Obs	Cum. % of Obs
Over Estimated	< -10	0%	15%	6%	13%	19%	80%
	-10	1%		0%		5%	
	-9	2%		1%		4%	
	-8	1%		2%		4%	
	-7	2%		1%		3%	
	-6	3%		1%		6%	
	-5	0%		1%		5%	
	-4	2%		0%		10%	
	-3	4%		1%		15%	
	-2	1%		0%		10%	
Acceptable	-1	9%	65%	1%	36%	6%	20%
	0	22%		0%		1%	
	1	34%		36%		13%	
Under Estimated	2	15%	20%	28%	50%	0%	0%
	3	3%		13%		0%	
	4	0%		1%		0%	
	5	2%		1%		0%	
	6	0%		4%		0%	
	7	0%		3%		0%	
	8	0%		0%		0%	
	9	0%		0%		0%	
	10	0%		0%		0%	

3.3 MNLR

A minimum queue length of a single vehicle to a maximum of 7 vehicles in the queue is observed for MNLR lane configuration. The Two-Minute Rule estimated a maximum of 7 vehicles, while the HCM predicts a maximum of 3 vehicles in queue for the similar prevailing conditions. Gard's equation tends to overestimate the queues as shown in Table 3-6. The data description is given in Table 3-5 and shown in Figure 3-4 and Figure 3-5.

Around 33% are exactly matched with observed values by the Two-Minute Rule which performs far better than other models. If the difference between the observed and estimated queue lengths is relaxed to ±1, 84% are matched for the Two-Minute Rule, while 46% is matched for Gard's equation.

Table 3-5 Summary Statistics for MNLR

	VOL	CONVOL	Observed	Two_min_Rule	HCM
Count	81	81	81	81	81
Average	66.7654	1490.32	2.39506	2.23457	0.259259
Variance	2683.03	282928.0	1.96698	2.73179	0.519444
Standard deviation	51.798	531.909	1.40249	1.65281	0.720725
Minimum	12.0	802.0	1.0	0.0	0.0
Maximum	224.0	2489.0	7.0	7.0	3.0
Range	212.0	1687.0	6.0	7.0	3.0
Stnd. skewness	5.40679	1.60496	5.77836	5.02874	11.2284
Stnd. kurtosis	3.87235	-1.99022	5.03514	2.94678	16.0452

	Gards
Count	81
Average	4.23457
Variance	7.68179
Standard deviation	2.7716
Minimum	0.0
Maximum	9.0
Range	9.0
Stnd. skewness	0.768588
Stnd. kurtosis	-1.66468

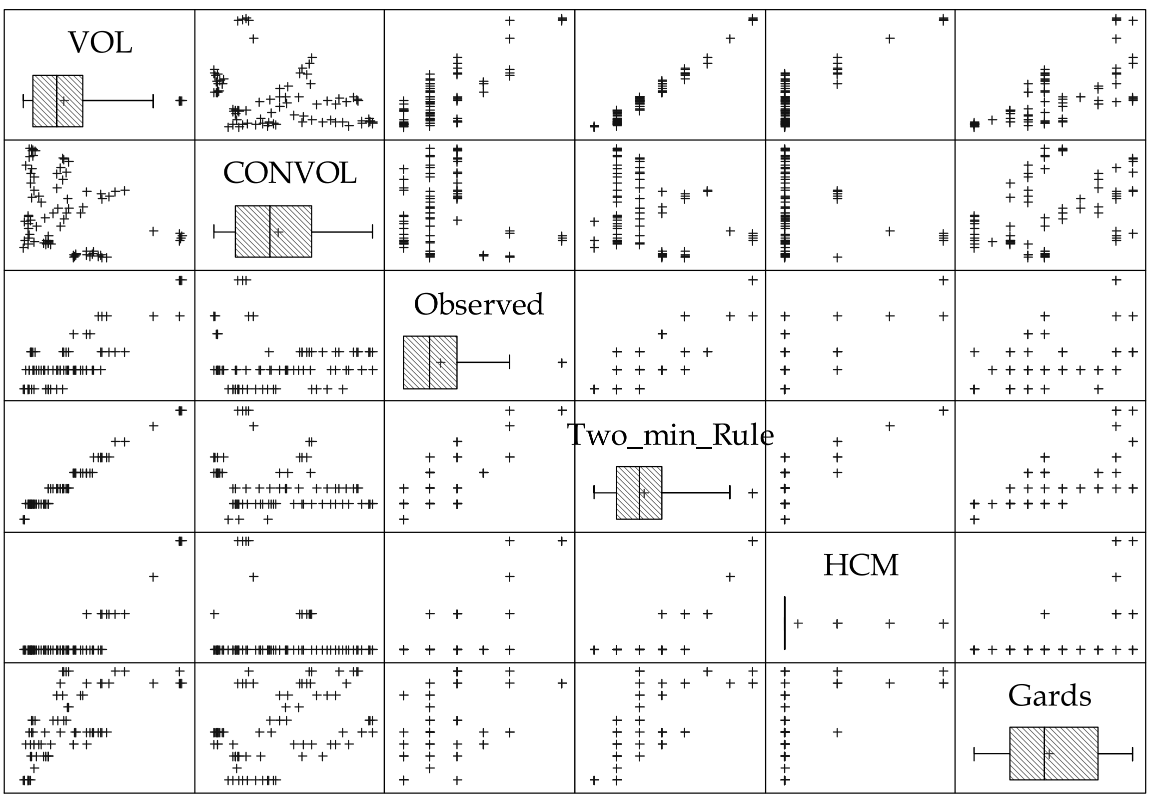


Figure 3-4 Scattered plot of VOL, CONVOL, & Queue Lengths for MNLR

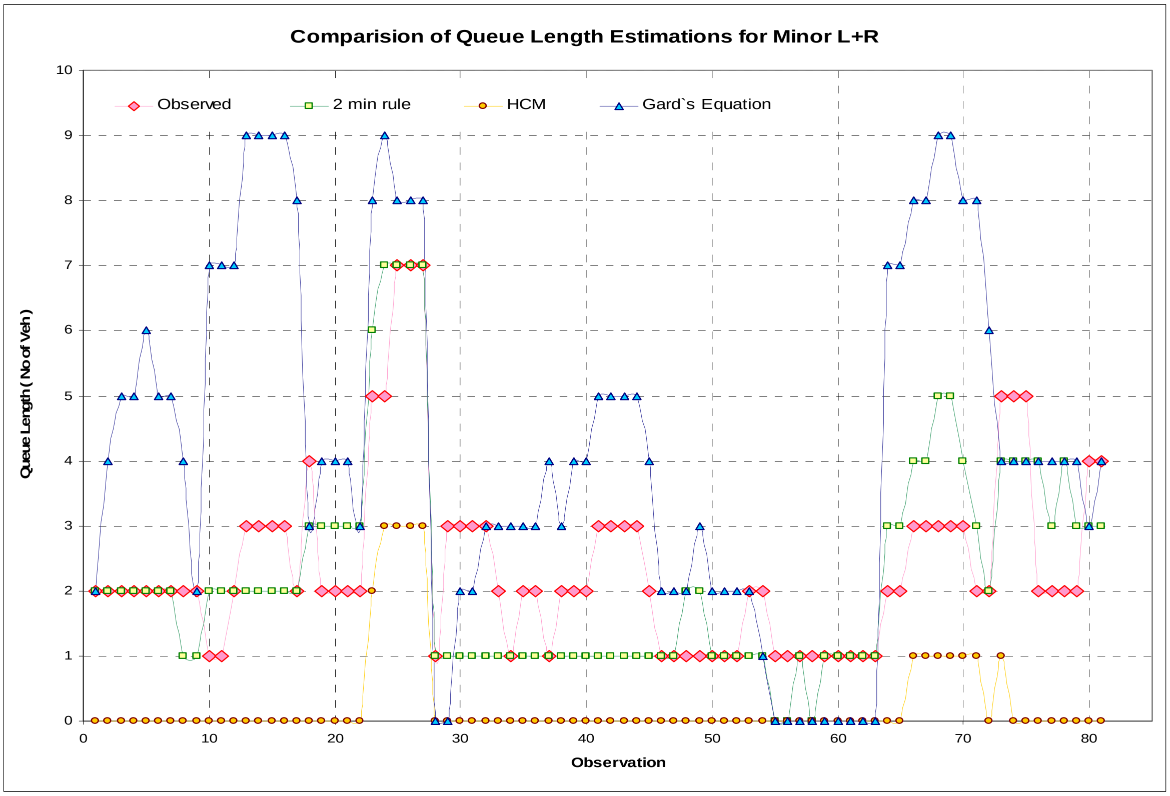


Figure 3-5 Scatter plots of all methods for MNLR

Table 3-6 Difference between Observed and Model Outputs for MNLR

Type of Estimation	Difference (against Observed)	Observed - 2 min rule % Obs	Observed - 2 min rule Cum % of Obs	Observed - HCM method % Obs	Observed - HCM method Cum % of Obs	Observed - Gard's Equation % Obs	Observed - Gard's Equation Cum % of Obs
Over Estimated	-6	0%	6%	0%	0%	12%	53%
	-5	0%		0%		7%	
	-4	0%		0%		4%	
	-3	0%		0%		7%	
	-2	6%		0%		22%	
Acceptable	-1	21%	84%	0%	27%	17%	46%
	0	33%		0%		6%	
	1	30%		27%		22%	
Under Estimated	2	10%	10%	46%	73%	0%	1%
	3	0%		16%		1%	
	4	0%		9%		0%	
	5	0%		2%		0%	
	6	0%		0%		0%	
	7	0%		0%		0%	
	8	0%		0%		0%	
	9	0%		0%		0%	

3.4 MNL

All models consistently underestimate the MNL queue length. Only the Two-Minute Rule predictions show a consistent trend with increasing volumes. This group has the lowest number of observations, 38. A maximum queue length of 11 vehicles is observed in queue. The description of the data is shown in Table 3-7. Scatter plots of the observed data and the distribution of the queue length predictions are shown in Figure 3-6 and 3-7 respectively.

The Two-Minute Rule predicts 18% of the observed values. Predictions improved to nearly 40% with a tolerance of a single vehicle. Gard's equation predicts 32% as shown in Table 3-8.

Table 3-7 Summary Statistics for MNL

	VOL	CONVOL	Observed	Two_min_Rule	HCM
Count	38	38	38	38	38
Average	70.0	928.132	4.44737	2.26316	0.526316
Variance	1469.95	62556.9	7.33499	1.60455	0.472262
Standard deviation	38.3399	250.114	2.70832	1.26671	0.687213
Minimum	7.0	457.0	1.0	0.0	0.0
Maximum	132.0	1286.0	11.0	4.0	2.0
Range	125.0	829.0	10.0	4.0	2.0
Stnd. skewness	-0.189789	-0.58707	3.24311	-0.691012	2.4046
Stnd. kurtosis	-1.37725	-1.66867	1.94871	-1.15355	-0.306799

	Gards
Count	38
Average	4.92105
Variance	5.58819
Standard deviation	2.36394
Minimum	0.0
Maximum	7.0
Range	7.0
Stnd. skewness	-2.88054
Stnd. kurtosis	0.120273

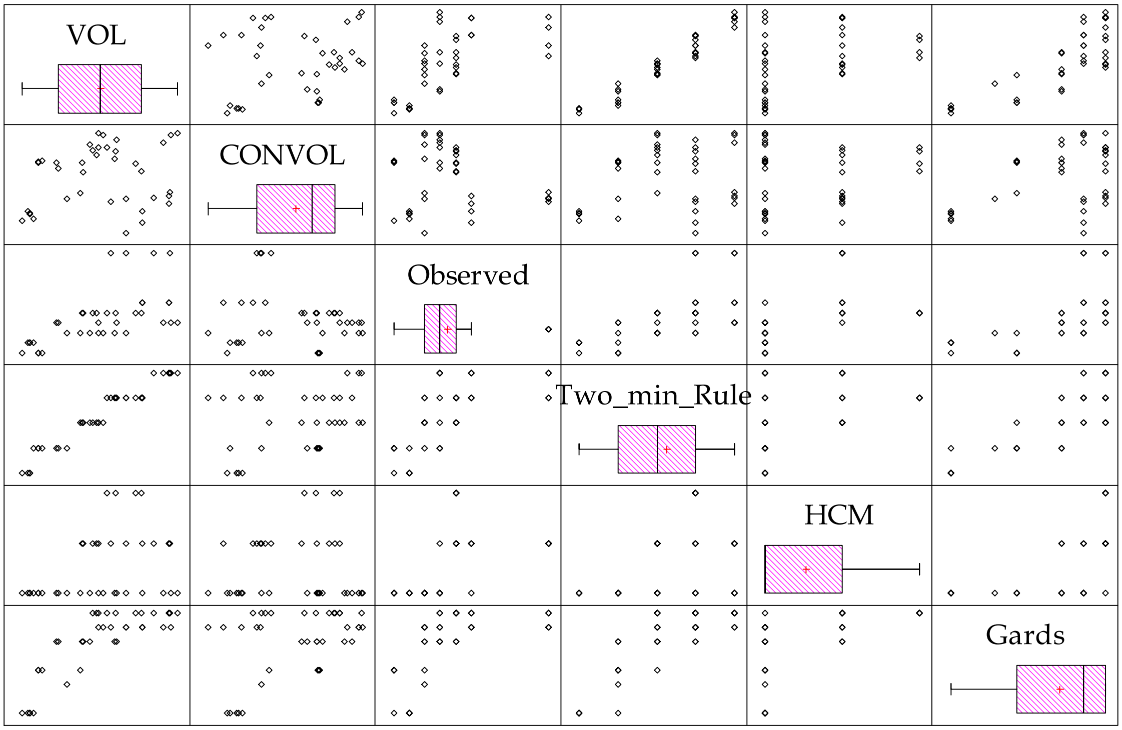


Figure 3-6 Scattered plot of VOL, CONVOL, & Queue Lengths for MNL

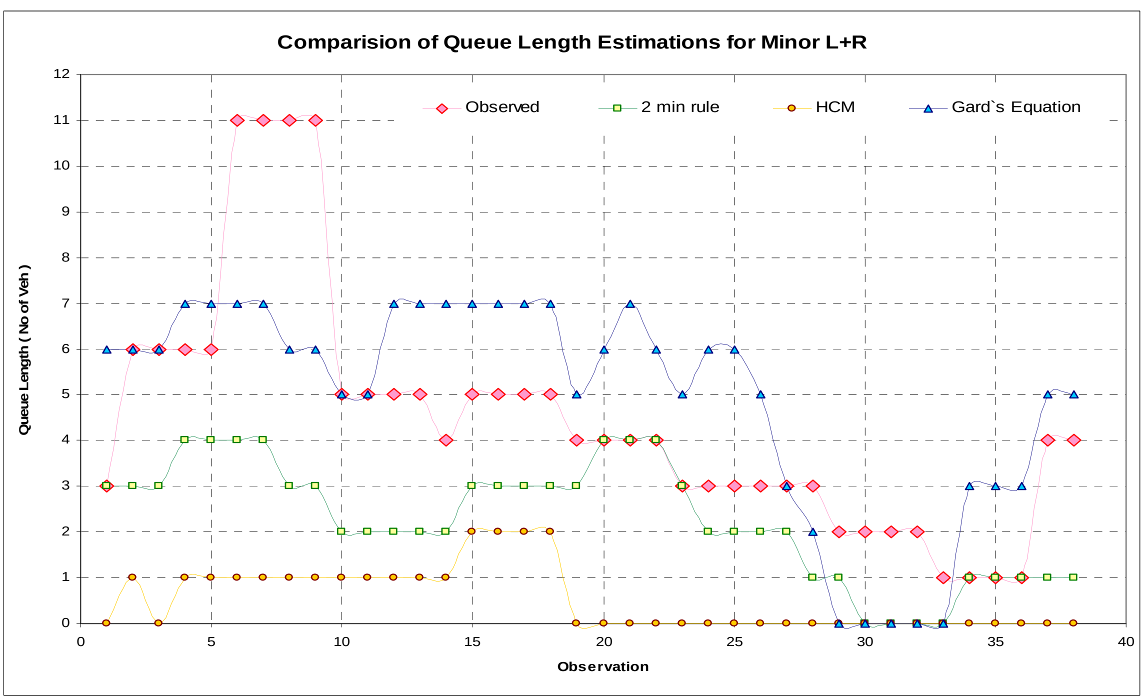


Figure 3-7 Scatter plot of all methods for MNL

Table 3-8 Difference between Observed and Model Outputs for MNL

Type of Estimation	Difference (against Observed)	Observed - 2 min rule		Observed - HCM method		Observed - Gard's Equation	
		% Obs	Cum. % Obs	% Obs	Cum. % Obs	% Obs	Cum. % Obs
Over Estimated	-5	0%	0%	0%	0%	0%	47%
	-4	0%		0%		0%	
	-3	0%		0%		13%	
	-2	0%		0%		34%	
Acceptable	-1	0%	39%	0%	11%	13%	32%
	0	21%		0%		13%	
	1	18%		11%		5%	
Under Estimated	2	29%	61%	11%	89%	11%	21%
	3	21%		32%		0%	
	4	0%		26%		5%	
	5	0%		8%		5%	
	6	0%		3%		0%	
	7	5%		0%		0%	
	8	5%		0%		0%	
	9	0%		0%		0%	
	10	0%		11%		0%	

3.5 *Summary*

Analysis related to the relative performances of the Two-Minute Rule, Highway Capacity Manual Method, and John T. Gard's equations in predicting the queue lengths at two-way STOP controlled intersections are presented in the chapter. For Major Left Turn (MJL) lane configurations Gard's equation performs well. For the remaining lane configurations, the Two-Minute Rule predicts the best. On average, the performance of matching with ± one vehicle variation does not exceed 70 percent. This triggers the effort to build a model to assess the queue length in consistent manner. The model development procedure is presented in the next chapter.

4 Model Development

Model development plays a key role for predicting vehicles in stopped queue. A well formulated model significantly explains the changes to the dependent variable for variations in the independent variable. Though a variety of models ranging from simple regression to complex simulation exists today, this study is limited to development of a regression model. This is intuitive because of the simple model structure and easy usage. Not only are regression models straight forward to understand, but also the model user can study model sensitivity by changing the values of the independent variables.

This chapter explains the model development process. First, the factors influencing the queue behavior for different lane configuration movements at unsignalized intersections are identified. Then, a scatter diagram of the observed data is analyzed to identify the appropriate regression model. Next, for the chosen regression model the combinations of influenced variables are identified to incorporate into the model. After that the model is formulated and developed for each lane configuration movement. Finally, statistical tests are performed to check the model reasonableness.

4.1 Factors Influencing Queue Behavior

Primarily geometry, operations, traffic flow, and human travel characteristics influence the queue behavior. Over the past decade, numerous models have been developed taking into consideration one or a combination of these characteristics. Influencing factors are listed in Table 4-1.

It is very difficult to capture the effects of all parameters. Instead, it is assumed that lane configuration, major and minor approach volume, number of conflicting lanes, volumes and speed on the conflicting lanes, presence of turn lanes, right turn channelization, flared right turn lanes, and presence of traffic signal within 1000ft of the study intersections are the most influencing parameters.

Table 4-1 Factors Influencing Queue Behavior

Category	List of Factors
Geometry	Number of approaches Number of lanes on both major and minor approaches Lane configuration (shared/separate) Chanallization / Flared approaches Median Type Grade Sight Distance Intersection Skewness
Operations	Traffic flow speed Upstream Signal
Traffic Flow	Approach volume Conflicting volume Arrival type Turning volume Percent of heavy vehicles Gap and follow-up times Time of day / seasonal variation
Human Factors	Reaction time

It is a fact that traffic movements will behave uniquely for the given lane configuration. Lane groups are identified according the definitions provided in the 2000 Highway Capacity Manual. This chapter reiterates the lane groups considered in this study as shown below and from this point forward models are designated with the lane group codes shown below.

- ***MNLTR*** is for minor street on four legged intersection having a single lane for left, through and right turn movements
- ***MNLR*** is for minor street on three legged intersection having a single lane for left, and right turn movements
- ***MJL*** is for a Major left turn movement irrespective of exclusive/median/TWTL configuration
- ***MNL*** is for an exclusive left turn lane on minor approach of either four or three legged intersection
- ***MNR*** is for an exclusive right turn lane on four legged intersection minor approach

The data collection and analysis chapter explains the preparation of the data sets for modeling purposes. Scatter plots for different lane group configurations are drawn to get the outlook of the data trend and type of model to choose. As such, the trends of the queue lengths (denoted as QL, is the maximum number of vehicles in the stopped queue for the same time unit of volume measurement) with respect to the changes in volume (VOL) and conflicting volume (CONVOL) were evaluated. Following sub-sections describe the findings for various lane group models.

4.2 *Poisson Regression Model*

A general description of the Poisson Regression Model is given as it is the model which is best suited for the study conditions. Number of vehicles in the queue is of count type, often called discrete type, taking only a finite number of values. The probability distribution that is specifically suited for count data is the Poisson probability distribution. An interesting feature of the Poisson distribution: its variance is the same as its mean value.

The Poisson regression model may be written as:

$$\mathrm{Ln}(\mathrm{Y}) = \beta_0 + \beta_1 \times \mathrm{X}_1 + \beta_2 \times \mathrm{X}_2 + \ldots\ldots\ldots\ldots\ldots\ldots + \beta n \times \mathrm{X}_\mathrm{n}$$

Where

Ln = Natural LogarithmY = Dependent Variable

X_1,X_2,……X_n = Independent or Explanatory Variables

β_0 = Constant

β_1 , β_2 , β_3 = Model coefficients corresponds X_1,X_2,……X_n

4.3 *Major Left Turn (MJL) Model*

Table 4-2 shows summary statistics for each of the selected data variables. It includes measures of central tendency, measures of variability, and measures of shape. Of particular interest here are the standardized skewness and standardized kurtosis, which can be used to determine whether the sample comes from a normal distribution. Values of these statistics outside the range of -2 to +2 indicate significant departures from normality, which would tend to invalidate many of the statistical procedures normally applied to this data. In this case, the variables show standardized skewness values outside the expected range are QL, VOL and VOLCONVOL. QL and VOL variables show standardized kurtosis values outside the expected range. QL value varies from 1 to 8 vehicles.

Table 4-2 Summary Statistics of Major Left Turn Data

	QL	VOL	CONVOL	VOLCONVOL
Count	219	219	219	219
Average	2.58904	29.9361	506.95	17163.2
Variance	2.61933	618.436	60516.9	3.03245E8
Standard deviation	1.61843	24.8684	246.002	17413.9
Minimum	1.0	1.0	2.0	51.0
Maximum	8.0	134.0	972.0	65715.0
Range	7.0	133.0	970.0	65664.0
Stnd. skewness	6.76384	6.45329	-0.729985	7.04459
Stnd. kurtosis	3.52603	3.8392	-1.97501	1.19911

Transformations are one of the methods to make the variables more normal. A scattered matrix plot among the variables shows no definite pattern between QL and explanatory variables VOL, CONVOL, and their product.

Figure 4-1 Scattered Matrix Plot between the variables for MJL

Though many variables listed in Table 4-1 adequately explain the queue behavior, only VOL, CONVOL, presence of upstream signal within 1000 ft distance from intersection (SIGNAL), and presence of exclusive left turn lane (Coded as LT, either median left turn lane or two-way left turn lane-TWTL) are selected, based upon lowest Mean Squared Error, highest R-squared value, and lowest Cp Statistics[7]. Results of this analysis are shown in Table 4-3.

After selecting the variables, analysis needs to be performed to identify the correlation among the independent variables. Table 4-4 shows Spearman rank correlations between each pair of variables. These correlation coefficients range between -1 and +1 and measure the strength of the association between the variables. In contrast to the more common Pearson correlations, the Spearman coefficients are computed from the ranks of the data values rather than from the values themselves. Consequently, they are less sensitive to outliers than the Pearson coefficients. Also shown in parentheses is the number of pairs of data values used to compute each coefficient. The third number in each location of the table is a P-value which tests the statistical significance of the estimated correlations. P-values below 0.05 indicate statistically significant "non-zero" correlations at the 95% confidence level.

[7] Mallows' Cp statistic is a measure of the bias in a model, based on a comparison of total mean squared error to the true error variance. Unbiased models have an expected value of approximately p, where p is the number of coefficients in the fitted model including constant

Table 4-3 Selection of Independent Variable for MJL Model

Regression Model Selection

Dependent variable: QL
Independent variables:
A=VOL
B=CONVOL
C=VOLCONVOL
D=SIGNAL
E=TWTL

Number of complete cases: 219
Number of models fit: 32

Model Results

MSE	R-Squared	Adjusted R-Squared	Cp	Included Variables
2.61933	0.0	0.0	502.988	
1.70135	35.3442	35.0463	250.514	A
1.97238	25.0443	24.6989	324.672	B
1.04799	60.1737	59.9902	71.7446	C
2.6145	0.642103	0.184232	500.365	D
2.15002	18.2936	17.9171	373.277	E
1.43358	45.7712	45.2691	177.441	AB
1.05279	60.1756	59.8068	73.7312	AC
1.7088	35.3602	34.7617	252.399	AD
1.53786	41.8265	41.2879	205.842	AE
1.00842	61.8539	61.5007	61.6472	BC
1.94017	26.6084	25.9288	315.411	BD
1.21427	54.0672	53.6419	117.711	BE
1.02662	61.1654	60.8058	66.6044	CD
0.822207	68.898	68.61	10.9306	CE
1.85755	29.7336	29.0829	292.91	DE
0.992793	62.619	62.0974	58.1386	ABC
1.40365	47.1491	46.4117	169.52	ABD
1.04388	60.6953	60.1469	71.9892	ABE
1.03133	61.1682	60.6264	68.5844	ACD
0.808386	69.5624	69.1377	8.14724	ACE
1.44915	45.4362	44.6748	181.853	ADE
1.0039	62.2008	61.6733	61.1501	BCD
0.824137	68.9693	68.5363	12.4173	BCE
1.16617	56.0909	55.4783	105.14	BDE
0.808903	69.5429	69.1179	8.28729	CDE
0.991387	62.8456	62.1511	58.5075	ABCD
0.811127	69.6012	69.033	9.86749	ABCE
1.02763	61.4872	60.7673	68.2879	ABDE
0.792048	70.3162	69.7614	4.7195	ACDE
0.811509	69.5869	69.0184	9.97064	BCDE
0.793088	70.4162	69.7217	6.0	ABCDE

Table 4-4 Spearman Rank Correlation Matrix

	QL	VOL	CONVOL	SIGNAL	TWTL
QL		0.6551	0.4673	0.1529	-0.4519
		(219)	(219)	(219)	(219)
		0.0000	0.0000	0.0239	0.0000
VOL	0.6551		0.4411	0.2112	-0.2724
	(219)		(219)	(219)	(219)
	0.0000		0.0000	0.0018	0.0001
CONVOL	0.4673	0.4411		0.3937	0.1713
	(219)	(219)		(219)	(219)
	0.0000	0.0000		0.0000	0.0114
SIGNAL	0.1529	0.2112	0.3937		0.4983
	(219)	(219)	(219)		(219)
	0.0239	0.0018	0.0000		0.0000
TWTL	-0.4519	-0.2724	0.1713	0.4983	
	(219)	(219)	(219)	(219)	
	0.0000	0.0001	0.0114	0.0000	

Correlation
(Sample Size)
P-Value

The model for predicting the maximum number of vehicles in the stopped queue is formulated as a Poisson regression equation with volume, conflicting volume, presence of upstream signal, and presence of left turn lane as the independent or explanatory variables.

$$\mathrm{Ln}(\mathrm{QL}_{\mathrm{MJL}}) = \beta_0 + \beta_1 \times \mathrm{VOL}_{\mathrm{MJL}} + \beta_2 \times \mathrm{CONVOL}_{\mathrm{MJL}} + \beta_3 \times SIGNAL + \beta_4 \times LT$$

Where

Ln = Natural Logarithm

VOL = Approach volume for MJL lane configuration

CONVOL = Conflicting volume for MJL

SIGNAL = Presence of upstream signal within 1000 ft of the intersection (1 if there is a signal, otherwise 0)

LT = Presence of left turn lane (1 if there is an exclusive left turn lane/median left turn lane/ two-way left turn lane, otherwise 0)

β_0 = Constant

β_1, β2, β3, β4 = Model coefficients corresponds to VOL, CONVOL, SIGNAL, and LT variables

The developed model with coefficient values and corresponding statistical tests are explained below:

$$\mathbf{Ln(QL) = 0.3925 + 0.0059 \times VOL + 0.00104 \times CONVOL + 0.49 * SIGNAL - 0.81 * LT}$$

$$QL = e^{(\mathbf{0.3925 + 0.0059 \times VOL + 0.00104 \times CONVOL + 0.49 * SIGNAL - 0.81 * LT})}$$

Percentage of deviance explained by model = 66.959

As the volume and conflicting volume increases, QL increases. Presence of an upstream signal within 1000 feet from the intersection increases QL. Moreover, presence of a separate left turn lane decreases QL as compared to not having a turn lane. These signs indicate the reasonableness of the model. Statistical analysis of the model is shown in Table 4-5.

Table 4-5 Statistical Analysis of MJL Model

```
Poisson Regression

Dependent variable: QL
Factors:
   VOL
   CONVOL
   SIGNAL
   TWTL

Estimated Regression Model (Maximum Likelihood)
------------------------------------------------------------
                                          Standard   Estimated
Parameter                Estimate            Error  Rate Ratio
------------------------------------------------------------
CONSTANT                 0.392535         0.119829
VOL                    0.00586284       0.00187515     1.00588
CONVOL                  0.0010416      0.000187399     1.00104
SIGNAL                    0.48999          0.16482      1.6323
TWTL                    -0.811879         0.140689    0.444023
------------------------------------------------------------

              Analysis of Deviance
---------------------------------------------------
Source          Deviance          Df      P-Value
---------------------------------------------------
Model           137.162            4       0.0000
Residual        67.6827          214       1.0000
---------------------------------------------------
Total (corr.)   204.845          218

Percentage of deviance explained by model = 66.959
Adjusted percentage = 62.0772

Likelihood Ratio Tests
---------------------------------------------------------
Factor                   Chi-Square          Df   P-Value
---------------------------------------------------------
VOL                      9.22746              1    0.0024
CONVOL                   33.0217              1    0.0000
SIGNAL                   8.98112              1    0.0027
TWTL                     37.2249              1    0.0000
---------------------------------------------------------

Residual Analysis
---------------------------------
      Estimation      Validation
n     219
MSE   2.78244
MAE   0.761339
MAPE  33.5926
ME    0.0189021
MPE   -16.1152
```

Because the P-value for the model in the Analysis of Deviance table is less than 0.01, there is a statistically significant relationship between the variables at the 99% confidence level. In addition, the P-value for the residuals is greater than or equal to 0.10, indicating that the model is not significantly worse than the best possible model for this data at the 90% or higher confidence level. The percentage of deviance in QL explained by the model equals 66.959%. This statistic is similar to the usual R-Squared statistic. The adjusted percentage, which is more suitable for comparing models with different numbers of independent variables, is 62.0772%.

In determining whether the model can be simplified, notice that the highest P-value for the likelihood ratio tests is 0.0027, belonging to SIGNAL. Because the P-value is less than 0.01, that term is statistically significant at the 99% confidence level. Consequently, it is not advisable to remove any variables from the model.

Table 4-6 shows estimated correlations between the coefficients in the fitted model. These correlations can be used to detect the presence of serious multicollinearity, i.e., correlation amongst the predictor variables. In this case, there is 1 correlation with an absolute value greater than 0.5.

Table 4-6 Correlation Matrix for Estimated Coefficients for MJL Model

Correlation matrix for coefficient estimates

	CONSTANT	VOL	CONVOL	SIGNAL
CONSTANT	1.0000	-0.3962	-0.6333	0.3000
VOL	-0.3962	1.0000	-0.3402	-0.1638
CONVOL	-0.6333	-0.3402	1.0000	-0.2150
SIGNAL	0.3000	-0.1638	-0.2150	1.0000
TWTL	-0.3673	0.3793	-0.0517	-0.7019

	TWTL
CONSTANT	-0.3673
VOL	0.3793
CONVOL	-0.0517
SIGNAL	-0.7019
TWTL	1.0000

The plot of the fitted model with 95% confidence limits (shown as red lines), predicted QL Vs Observed QL, and the Residual plot for QL, are shown in Figures 4-2 and 4-3, respectively. The Residual plot for QL shows most of the predictions have an error of ± 1 vehicle, due to rounding off to the nearest integer.

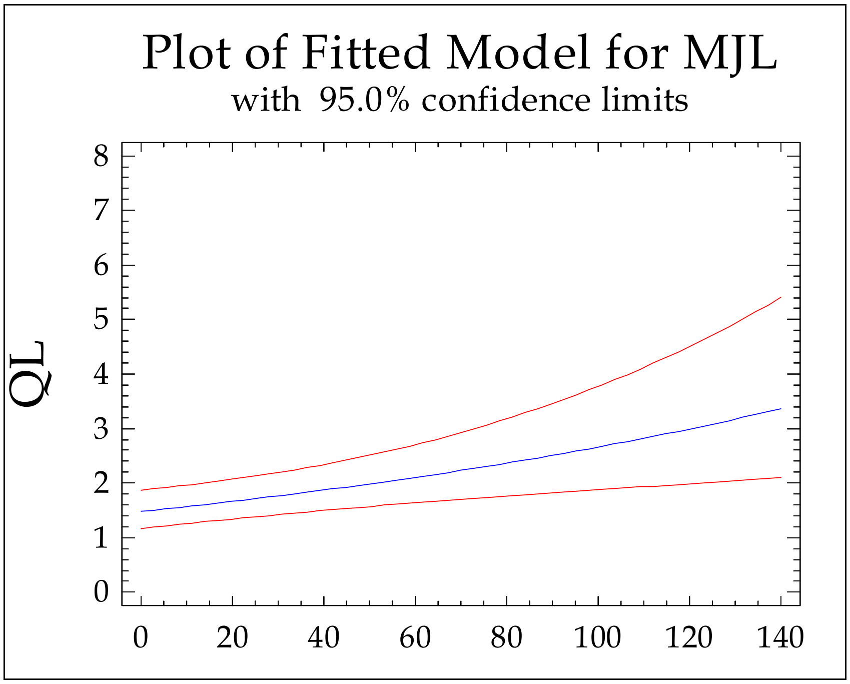


Figure 4-2 Plot of Fitted Model for MJL

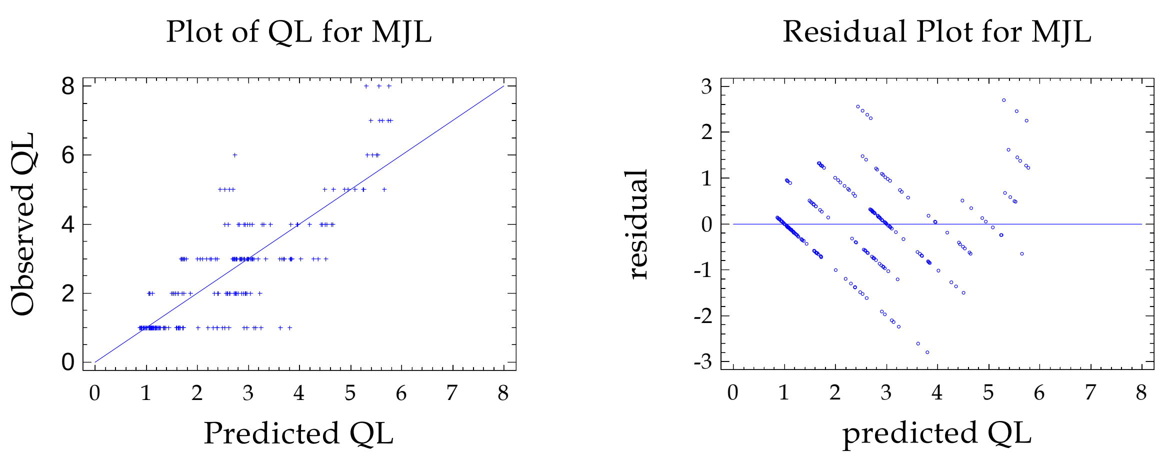


Figure 4-3 Residual Plot for MJL

4.4 *Minor, Shared LTR (MNLTR)*

There are 143 observations for this lane group category. Scatter plots are shown in Figure 4-4.

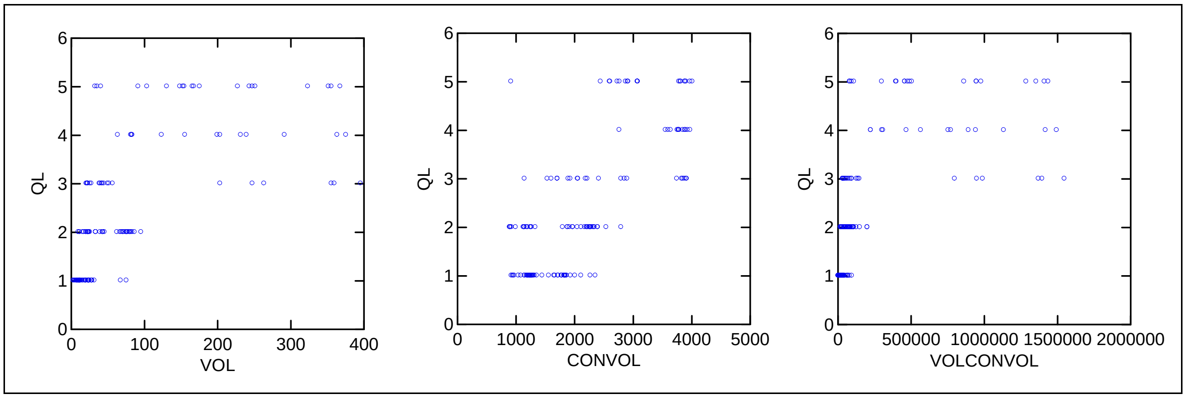


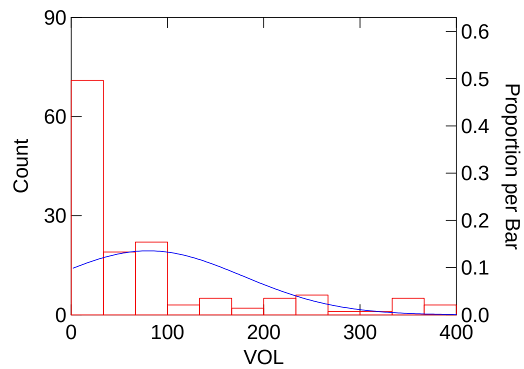
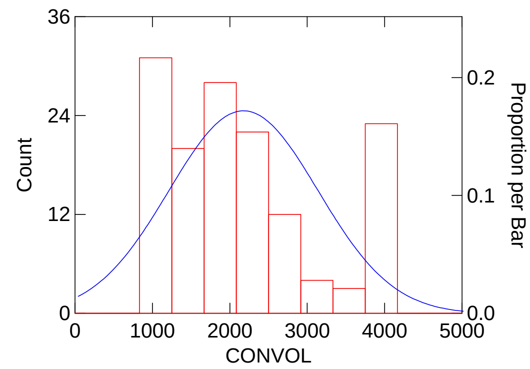
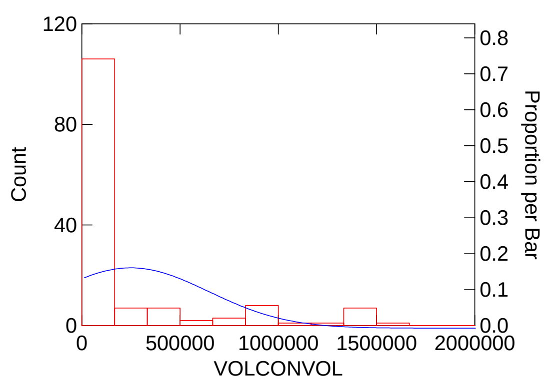
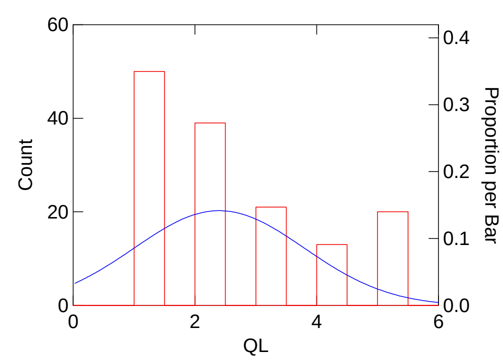
Figure 4-4 Scattered Plot of QL Vs VOL, CONVOL, and VOL*CONVOL for MNLTR

There exists no definite pattern among the variables, which shows the requirement of the transformation of the explanatory variables. This may be due to the random arrival of the minor street movements, which will influence the queue formation to be random. Various combinations of transformations are analyzed to recognize the patterns in the data, but none are explaining the queue behavior properly.

The distributions of volume, conflicting volume, product of these volumes, and queue lengths are following gamma distribution as listed in Table 4-7. In the Shapiro-Wilk test statistic for normality, the P-value for each distribution is less than the alpha value of 0.05. It is concluded that the data are not from the normally distribution population. So, VOL and CONVOL are not normally distributed.

It is assumed that the queue lengths tend to follow the random vehicle arrivals and therefore the Poisson regression model is tested. Before going further into the model development it is worthwhile to get a snap shot of the summary statistics for each variable in the model.

Table 4-7 Fitting Distributions for MNLTR Data

<table>
<tr>
<td>FITTED DISTRIBUTION
</td>
<td>FITTED DISTRIBUTION
</td>
</tr>
<tr>
<td>Distribution: Gamma
Kolmogorov-Smirnov test statistic = 0.246430
Lilliefors Probability (2-tail) = 0.000000
Shapiro-Wilk test statistic for normality = 0.714618
p-value = 0.000000
Estimated Shape (alpha) = 0.666946
Scale (beta) = 120.117511
Chi-square test statistic = 30.032026
df = 5, p-value = 0.000015</td>
<td>Distribution: Gamma
Kolmogorov-Smirnov test statistic = 0.118544
Lilliefors Probability (2-tail) = 0.000041
Shapiro-Wilk test statistic for normality = 0.895070
p-value = 0.000000
Estimated: Shape (alpha) = 5.017084
Estimated: Scale (beta) = 430.484357
Chi-square test statistic = 68.746965
df = 8, p-value = 0.000000</td>
</tr>
<tr>
<td>FITTED DISTRIBUTION
</td>
<td>FITTED DISTRIBUTION
</td>
</tr>
<tr>
<td>Distribution: Gamma
Kolmogorov-Smirnov test statistic = 0.339958
Lilliefors Probability (2-tail) = 0.000000
Shapiro-Wilk test statistic for normality = 0.620258
p-value = 0.000000
Variable Name: VOLCONVOL
Estimated: Shape (alpha) = 0.387296
Estimated: Scale (beta) = 636926.604762
Chi-square test statistic = 61.330933
df = 4 p-value = 0.000000</td>
<td>Distribution: Gamma
Kolmogorov-Smirnov test statistic = 0.234447
Lilliefors Probability (2-tail) = 0.000000
Shapiro-Wilk test statistic for normality = 0.831541
p-value = 0.000000
Variable Name: QL
Estimated: Shape (alpha) = 2.935354
Estimated: Scale (beta) = 0.817142
Chi-square test statistic = 192.914128
df = 8 p-value = 0.000000</td>
</tr>
</table>

As given in Table 4-8, the maximum queue length observed is 5 with minimum of 1 vehicle in the queue. Volumes range from 3 vehicles per hour to 396 vehicles with a mean arrival rate of 80 vph. A mean conflicting flow rate of 2160 vph is observed. The distribution of the product of volume and conflicting volume is skewed more to the right side, and all data sets are showing a trend that is not normally distributed.

Table 4-8 Summary Statistics for MNLTR data

Item	VOL	CONVOL	VOLCONVOL	QL
N of cases	143	143	143	143
Minimum	3.000	897.000	3600.000	1.000
Maximum	396.000	4016.000	1550149.000	5.000
Range	393.000	3119.000	1546549.000	4.000
Sum	11456.000	308848.000	3.52752E+07	343.000
Median	34.000	1935.000	69315.000	2.000
Mean	80.112	2159.776	246679.392	2.399
95% CI upper	96.385	2319.734	312434.895	2.631
95% CI lower	63.839	1999.819	180923.888	2.166
Std. Error	8.232	80.917	33263.424	0.117
Standard Dev	98.441	967.625	397772.702	1.405
Variance	9690.607	936297.414	1.58223E+11	1.974
C.V.	1.229	0.448	1.613	0.586
Skewness(G1)	1.790	0.630	2.002	0.697
SE Skewness	0.203	0.203	0.203	0.203
Kurtosis(G2)	2.248	-0.821	2.846	-0.808
SE Kurtosis	0.403	0.403	0.403	0.403
SW Statistic	0.715	0.895	0.620	0.832
SW P-Value	0.000	0.000	0.000	0.000

The next step after choosing the model is to select the appropriate independent variable(s) from the pool of identified influencing variables. The combinations of the variables are tested and chosen based upon the largest R^2 Value[8], lowest Mallow`s Cp statistic [9] value, and lowest Mean Square Error (MSE). The analysis results are shown in Table 4-9. Analysis indicates that VOL, CONVOL, and the product of VOL and CONVOL may explain the queue behavior significantly. This step leads to the basic model formulation and development.

---

[8] The adjusted R-Squared statistic measures the proportion of the variability in QL which is explained by the model.

9 Mallows' Cp statistic is a measure of the bias in a model, based on a comparison of total mean squared error to the true error variance. Unbiased models have an expected value of approximately p, where p is the number of coefficients in the fitted model including constant

Table 4-9 Selection of Dependent Variables for Poisson Regression

Regression Model Selection

Dependent variable: QL
Independent variables:
A=VOL
B=CONVOL
C=RATIO
D=CONLANES
E=CONSPEED
F=VOLCONVOL

Number of complete cases: 143
Number of models fit: 63

Model Results

MSE	R-Squared	Adjusted R-Squared	Cp	Included Variables
0.611827	69.6575	69.0026	1.439	ABF
0.614559	69.7412	68.8642	3.06236	ABDF
0.615163	69.7115	68.8336	3.19602	ABEF
0.615628	69.6886	68.81	3.29892	ABCF
0.618901	69.7483	68.6442	5.03076	ABCDF
0.618946	69.7461	68.6419	5.04066	ABDEF
0.619454	69.7212	68.6162	5.15227	ABCEF
0.76147	62.2362	61.4212	34.8098	BCE
0.762063	62.4787	61.3911	35.7194	ABCE
0.764151	61.8306	61.2853	34.6337	BC
0.765162	62.3261	61.2341	36.4056	BCDE
0.765886	62.0172	61.1974	35.7946	BCD
0.766978	61.963	61.1421	36.0382	ABC
0.767159	62.5015	61.1329	37.617	ABCDE
0.768823	61.8715	61.0486	36.4495	BCF
0.770255	62.3501	60.976	38.2975	BCDEF
0.848438	57.6204	57.015	53.5652	AB
0.877404	56.1736	55.5475	60.0711	BD
0.884581	55.4995	55.1839	61.1023	B
0.885017	55.7933	55.1618	61.7809	BF
0.88622	55.7332	55.1008	62.0512	BE
1.08969	45.1812	44.7924	107.5	A
1.16248	41.5195	41.1047	123.965	F
1.44374	27.3698	26.8546	187.591	C
1.85577	6.64192	5.9798	280.797	D
1.9738	0.0	0.0	308.663	

A correlation matrix between QL and the independent variables is given in Table 4-10. A correlation coefficient of either +1 or -1 shows perfect correlation in positive or negative manner. A 0 correlation coefficient shows that independent variables can not explain queue behavior sufficiently. A positive sign in the matrix represents a positive correlation which indicates that QL increases as the independent or explanatory variable increases. Moreover, all explanatory variables are correlated with QL. There exists correlation between VOL and CONVOL. This may be due to the fact that CONVOL and VOL are volumes on different lanes, and as VOL increase from the off-peak period to the peak period, CONVOL may increase in volume to represent the peak condition. Finally, the correlation between the product of VOL and CONVOL with VOL and CONVOL will be expected as the product comes from both VOL and CONVOL. Though consideration of all three explanatory variables seems to dampen the model performance as a result of correlation, the comparison of models with various combinations of variables as shown in Table 4-11 indicates that these variables perform better for the observed data. The other reason to consider the product term is to capture the QL for the corresponding pair of VOL and CONVOL.

Table 4-10 Correlation Matrix of Variables for MNLTR Model

Variable	VOL	CONVOL	VOLCONVOL	QL
VOL	1.000	0.780	0.986	0.672
CONVOL	0.780	1.000	0.824	0.745
VOLCONVOL	0.986	0.824	1.000	0.644
QL	0.672	0.745	0.644	1.000

The model for predicting the maximum number of vehicles in the stopped queue is formulated as a Poisson regression equation with volume, conflicting volume, and their product as independent or explanatory variables.

$$\mathrm{Ln}(\mathrm{QL}_{\mathrm{MNLTR}}) = \beta_0 + \beta_1 \times \mathrm{VOL}_{\mathrm{MNLTR}} + \beta_2 \times \mathrm{CONVOL}_{\mathrm{MNLTR}} + \beta_3 \times (\mathrm{VOL}_{\mathrm{MNLTR}} * \mathrm{CONVOL}_{\mathrm{MNLTR}})$$

Where

Ln = Natural Logarithm

VOL = Approach volume for MNLTR lane configuration

CONVOL = Conflicting volume for MNLTR

β_0 = Constant

β_1 , β_2 , β_3 = Model coefficients corresponds to VOL, CONVOL, and VOL*CONVOL

The Poisson Regression model with coefficient values and corresponding statistical tests are explained below:

$$\mathbf{Ln(QL) = -0.7844 + 0.01636 \times VOL + 0.0006 \times CONVOL - 0.0000043 \times (VOL * CONVOL)}$$

$$QL = e^{(-0.7844 + 0.01636 \times VOL + 0.0006 \times CONVOL - 0.0000043 \times (VOL * CONVOL))}$$

Percentage of Deviation Explained by Model = 71.643

Signs of the independent variables show reasonableness in the models, as a positive change in the VOL and CONVOL results in increasing QL. But the occurrence of VOL and the corresponding CONVOL as a product triggers a decrease in QL, due to the fact that lower approach volume may not yield larger queue lengths, rather more waiting time in the stopped queue. As approach volumes increase to capacity, the approach volume has a greater higher impact on queue than the conflicting volume.

Table 4-11 Possion Regression Model for MNLTR

```
Poisson Regression
Dependent variable: QL
Factors:   VOL,   CONVOL,   VOLCONVOL
Estimated Regression Model (Maximum Likelihood)
------------------------------------------------------------
                                         Standard    Estimated
Parameter                Estimate           Error   Rate Ratio
------------------------------------------------------------
CONSTANT                -0.784374        0.247345
VOL                     0.0163606      0.00357521       1.0165
CONVOL                0.000598612     0.000102971       1.0006
VOLCONVOL           -0.0000043145      9.55645E-7     0.999996
------------------------------------------------------------
              Analysis of Deviance
---------------------------------------------------
Source          Deviance           Df      P-Value
---------------------------------------------------
Model           80.5605             3       0.0000
Residual        31.8861           139       1.0000
---------------------------------------------------
Total (corr.)   112.447           142
Percentage of deviance explained by model = 71.6433
Adjusted percentage = 64.5289
Likelihood Ratio Tests
---------------------------------------------------------
Factor                   Chi-Square          Df   P-Value
---------------------------------------------------------
VOL                         19.6951           1    0.0000
CONVOL                      33.5012           1    0.0000
VOLCONVOL                   19.6016           1    0.0000
---------------------------------------------------------
Residual Analysis
---------------------------------
      Estimation       Validation
n     143
MSE   1.53264
MAE   0.581838
MAPE  24.0153
ME    -0.00423549
MPE   -9.38096
```

Statistical significance is given in Table 4-11. The model accounts for 71.6 percent of deviance[10] explained in QL. Because the P-value for the model in the Analysis of Deviance table is less than 0.01, there is a statistically significant relationship between the variables at the 99% confidence level. In addition, the P-value for the residuals is greater than or equal to 0.10, indicating that the model is not significantly worse than the best possible model for this data at the 90% or higher confidence level.

[10] The percentage of deviance statistic is similar to the usual R-Squared statistic.

The adjusted percentage, which is more suitable for comparing models with different numbers of independent variables, is 64.5289%. In determining whether the model can be simplified, notice that the highest P-value for the likelihood ratio tests is 0.0000, belonging to VOLCONVOL. Because the P-value is less than 0.01, that term is statistically significant at the 99% confidence level. Consequently, it is not advisable to remove any variables from the model. The plot of predicted QL and Observed QL is shown in Figure 4-5. The Residual plot for QL given in Figure 4-6 shows most of the predictions have an error of ± 1 vehicle, due to rounding-off error to the nearest integer.

Plot of QL for MNLTR

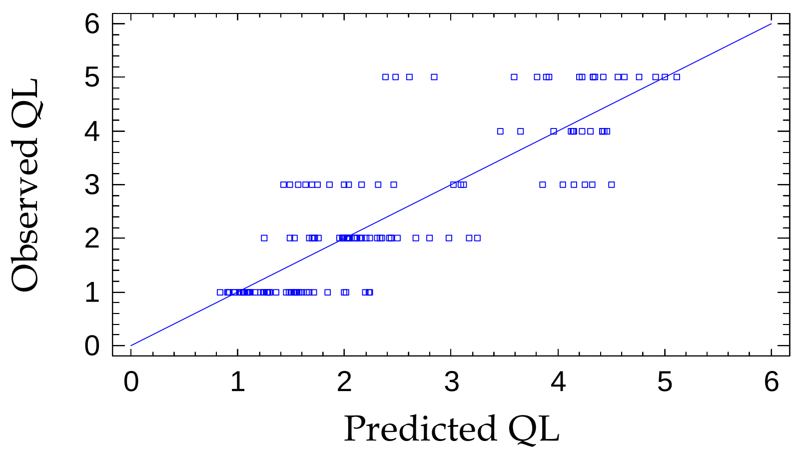


Figure 4-5 Plot of Predicted QL and Observed QL for MNLTR Model

Residual Plot for MNLTR

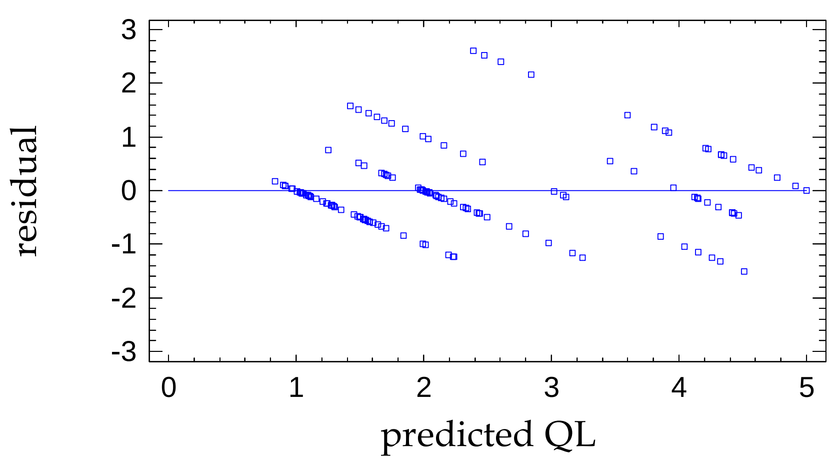


Figure 4-6 Residual Plot of Predicted QL and Observed QL for MNLTR Model

4.5 Minor, LR (MNLR)

This section explains model development for minor approach queue length estimation for a T-intersection where there exists a single lane for both left and right turn movements. There are 81 data point for model development. Summary statistics of the observed data corresponding to VOL, CONVOL, and their product is given in Table 4-12.

Table 4-12 Summary Statistics for MNLR lane group

	VOL	CONVOL	VOLCONVOL	QL
Count	81	81	81	81
Average	66.7654	1490.32	91333.1	2.39506
Variance	2683.03	282928.0	4.53467E9	1.96698
Standard deviation	51.798	531.909	67340.0	1.40249
Minimum	12.0	802.0	11250.0	1.0
Maximum	224.0	2489.0	273273.0	7.0
Range	212.0	1687.0	262023.0	6.0
Stnd. skewness	5.40679	1.60496	4.51837	5.77836
Stnd. kurtosis	3.87235	-1.99022	1.18442	5.03514

The observed maximum number of vehicles in the stopped queue (QL) varies from a minimum of 1 to a maximum of 7. Standardized skewness and standardized kurtosis statistics outside the range of -2 to +2 indicate significant departures from normality. So, QL is believed to be not from a Normal Distribution. Also VOL and VOLCONVOL show standardized skewness values outside the expected range. Standardized kurtosis values for VOL are also outside the expected range. Transformations may make these variables to be normal.

Analysis of the scatter plot between QL and VOL, and CONVOL shows the possible relation among variables. A box-and-whisker diagram or plot [11] is shown as the diagonal of the matrix in Figure 4-7. No definite pattern is observed among these plots. Therefore QL is assumed to follow the random process of vehicle arrivals. A Poisson regression model is tested as the initial step in model development.

[11] is a convenient way of graphically depicting groups of numerical data through their five-number summaries: the smallest observation (sample minimum), lower quartile (Q1), median (Q2), upper quartile (Q3), and largest observation (sample maximum). A boxplot may also indicate which observations, if any, might be considered outliers.

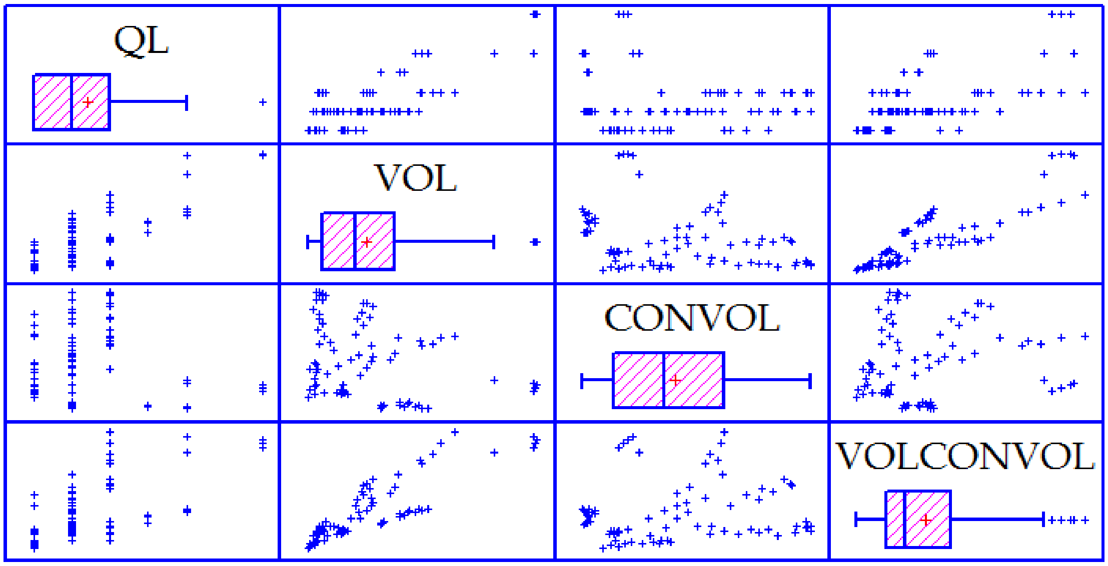


Figure 4-7 Scattered Matrix Plot for MNLR Lane Group Data

The next step is to choose appropriate set of variables from the identified list of variables from Table 4-13. Based on the lowest MSE, highest R-squared, and lowest Cp Statistic, VOL, CONVOL, and the product of VOL and CONVOL are considered in the model.

Variable selection leads to the step of correlation analysis to check for the serial correlation among the independent variables. Due to the fact that VOL and CONVOL are simply volume occurring on different lanes, there may be some correlation between them. Likewise, the product term has either VOL or CONVOL which will trigger the correlation. This has an impact on the model, but they can not be excluded to capture the impact on QL. The correlation matrix is shown in Table 4-14.

Table 4-14 Pearson Correlation Matrix of Variables in MNLR Model

Variable	QL	VOL	CONVOL	VOLCONVOL
QL	1.000	0.771	-0.064	0.643
VOL	0.771	1.000	-0.300	0.869
CONVOL	-0.064	-0.300	1.000	0.123
VOLCONVOL	0.643	0.869	0.123	1.000

Table 4-13 Variables Selection for MNLR Model

```
Regression Model Selection

Dependent variable: QL
Independent variables:
   A=VOL
   B=CONVOL
   C=RATIO
   D=VOLCONVOL

Number of complete cases: 81
Number of models fit: 16

Model Results
```

MSE	R-Squared	Adjusted R-Squared	Cp	Included Variables
1.96698	0.0	0.0	220.885	
0.806398	59.5156	59.0031	44.4064	A
1.96698	1.25	0.0	221.638	B
1.77722	10.7766	9.64718	190.567	C
1.16805	41.3593	40.617	98.8545	D
0.755034	62.5741	61.6145	37.2345	AB
0.625702	68.9849	68.1896	18.0095	AC
0.810442	59.8276	58.7975	45.4708	AD
1.74266	13.6189	11.404	184.044	BC
1.14078	43.4533	42.0034	94.5747	BD
1.18154	41.433	39.9313	100.633	CD
0.633765	68.988	67.7797	20.0002	ABC
0.526761	74.224	73.2197	4.29828	ABD
0.604781	70.4063	69.2533	15.7471	ACD
1.03289	49.4574	47.4882	78.5694	BCD
0.524729	74.6569	73.3231	5.0	ABCD

The model for predicting the maximum number of vehicles in the stopped queue is formulated as a Poisson regression equation with volume, conflicting volume, and their product as independent or explanatory variables.

$$Ln(QL_{MNLR}) = \beta_0 + \beta_1 \times VOL_{MNLR} + \beta_2 \times CONVOL_{MNLR} + \beta_3 \times (VOL_{MNLR} * CONVOL_{MNLR})$$

Where

Ln = Natural Logarithm

VOL = Approach volume for MNLR lane configuration

CONVOL = Conflicting volume for MNLR

β_0 = Constant

β_1 , β_2 , β_3 = Model coefficients corresponds to VOL, CONVOL, and VOL*CONVOL

The developed model with coefficient values and corresponding statistical tests are explained below:

$$\mathbf{Ln(QL) = -0.6319 + 0.0173 \times VOL + 0.00066 \times CONVOL - 0.000007913\,(VOL * CONVOL)}$$
$$QL = e^{(-0.6319 + 0.0173 \times VOL + 0.00066 \times CONVOL - 0.000007913\,(VOL * CONVOL))}$$

Percentage of deviance explained by model = 69.25

The signs of the independent variables show reasonableness in the models, as a positive change in the VOL and CONVOL results in increasing QL. But the occurrence of VOL and the corresponding CONVOL as a product triggers a decrease in QL, due to the fact that lower approach volume may not yield larger queue lengths, rather increase waiting time in the stopped queue. As volumes increase to capacity, the approach volume has a higher impact on queue than the conflicting volume. The statistics of model development are given in Table 4-15.

The model accounts for 69.25 percent of deviance[12] explained in QL. Because the P-value for the model in the Analysis of Deviance table is less than 0.01, there is a statistically significant relationship between the variables at the 99% confidence level. In addition, the P-value for the residuals is greater than or equal to 0.10, indicating that the model is not significantly worse than the best possible model for this data at the 90% or higher confidence level.

Table4-15 Poisson Regression Model for MNLR Data

[12] The percentage of deviance statistic is similar to the usual R-Squared statistic.

```
Poisson Regression

Dependent variable: QL
Factors:
   VOL
   CONVOL
   VOLCONVOL

Estimated Regression Model (Maximum Likelihood)
------------------------------------------------------------
                                      Standard    Estimated
Parameter               Estimate         Error   Rate Ratio
------------------------------------------------------------
CONSTANT               -0.631869      0.399069
VOL                    0.0172923    0.00455457      1.01744
CONVOL               0.000662669   0.000246615      1.00066
VOLCONVOL        -0.00000791327  0.00000352853     0.999992
------------------------------------------------------------

              Analysis of Deviance
---------------------------------------------------
Source          Deviance           Df      P-Value
---------------------------------------------------
Model           39.7281             3       0.0000
Residual        17.6416            77       1.0000
---------------------------------------------------
Total (corr.)   57.3698            80

Percentage of deviance explained by model = 69.2492
Adjusted percentage = 55.3046

Likelihood Ratio Tests
---------------------------------------------------------
Factor                   Chi-Square          Df   P-Value
---------------------------------------------------------
VOL                      14.5754              1    0.0001
CONVOL                   6.88596              1    0.0087
VOLCONVOL                5.17992              1    0.0228
---------------------------------------------------------

Residual Analysis
---------------------------------
      Estimation      Validation
n     81
MSE   1.55813
MAE   0.618471
MAPE  26.7428
ME    -0.0326626
MPE   -9.84617
```

The adjusted percentage of deviance, which is more suitable for comparing models with different numbers of independent variables, is 55.3%. In determining whether the model can be simplified, notice that the highest P-value for the likelihood ratio tests is 0.0228, belonging to VOLCONVOL. Because the P-value is less than 0.05, that term is statistically significant at the 95% confidence level. Consequently, it is not advisable to remove any variables from the model. The plot of predicted QL and Observed QL is shown in Figure 4-8. The Residual plot for QL given in Figure 4-9 shows most of the predictions have an error of ± 1 vehicle, due to rounding-off error to the nearest integer.

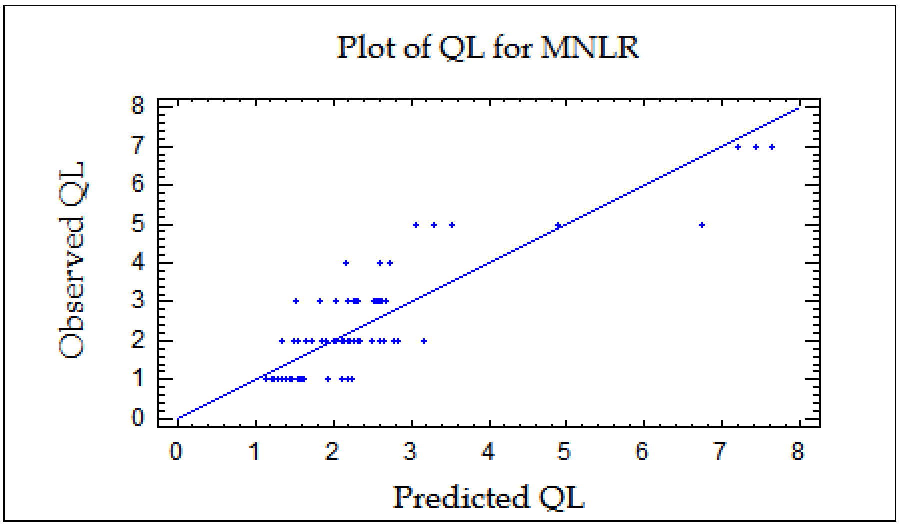


Figure 4-8 Plot of QL between Predicted Vs Observed QL for MNLR

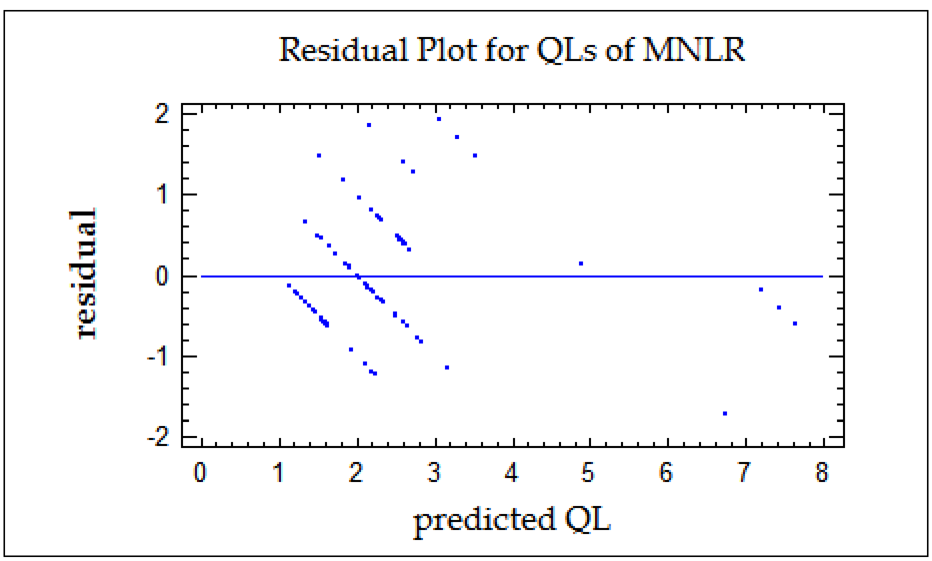


Figure 4-9 Residual Plot between Predicted QL and Residual for MNLR

4.6 *Minor Left (MNL)*

There are 34 data point available for model development. Table 4-16 shows the maximum QL observed ranges from one vehicle to 6 vehicles. Sample QL, VOL and CONVOL come from a normal distribution except the ratio of conflicting volume to the approach volume (designated as RATIO). Intuition for taking RATIO as the explanatory variable is partly explained through the scattered diagram shown in Figure 4-10, where a pattern is observed between QL and RATIO.

Table 4-16 Summary Statistics of MNL Data

	QL	VOL	CONVOL	RATIO
Count	34	34	34	34
Average	3.67647	66.2353	949.412	23.2288
Variance	2.40731	1467.16	65609.0	350.695
Standard deviation	1.55155	38.3035	256.143	18.7268
Minimum	1.0	7.0	457.0	5.03
Maximum	6.0	132.0	1286.0	83.35
Range	5.0	125.0	829.0	78.32
Stnd. skewness	-0.589831	0.183785	-1.09969	3.62622
Stnd. kurtosis	-1.08621	-1.27444	-1.42946	2.37195

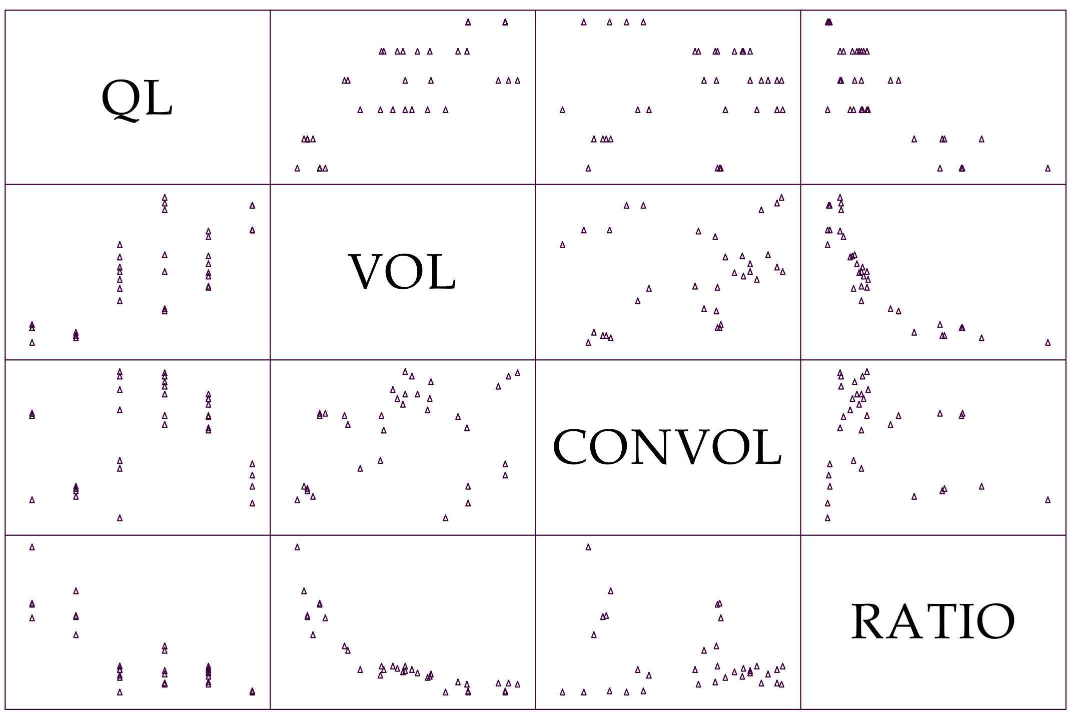


Figure 4-10 Scattered Matrix Plot of Variables for MNLR

Table 4-17 shows the results of fitting various multiple regression models to describe the relationship between QL and 4 predictor variables. Models have been fit containing all combinations of from 0 to 4 variables. The statistics tabulated include the mean squared error (MSE), the adjusted and unadjusted R-Squared values, and Mallows' Cp statistic. Models are determined to be best according to lowest values for all three criteria.

Table 4-17 Selection of Independent Variables for MNL Model

```
Regression Model Selection

Dependent variable: QL
Independent variables:
   A=VOL
   B=CONVOL
   C=VOLCONVOL
   D=RATIO

Number of complete cases: 34
Number of models fit: 16

Model Results
```

MSE	R-Squared	Adjusted R-Squared	Cp	Included Variables
2.40731	0.0	0.0	57.7769	
1.10936	55.3133	53.9169	10.1183	A
2.40731	3.0303	0.0	58.6512	B
1.69847	31.5832	29.4452	31.4225	C
0.91006	63.3415	62.196	2.91082	D
1.12208	56.2137	53.3888	11.31	AB
1.04489	59.2256	56.595	8.60595	AC
0.89479	65.083	62.8303	3.34741	AD
1.49105	41.8154	38.0616	24.2363	BC
0.910698	64.4622	62.1695	3.90472	BD
0.939369	63.3434	60.9784	4.90918	CD
1.01232	61.7708	57.9479	8.321	ABC
0.887672	66.4781	63.126	4.09488	ABD
0.861009	67.485	64.2336	3.19092	ACD
0.921012	65.2191	61.741	5.22521	BCD
0.884873	67.6977	63.2422	5.0	ABCD

Table 4-18 shows Spearman rank correlations between each pair of variables. These correlation coefficients range between -1 and +1 and measure the strength of the association between the variables. In contrast to the more common Pearson correlations, the Spearman coefficients are computed from the ranks of the data values rather than from the values themselves. Consequently, they are less sensitive to outliers than the Pearson coefficients. Also shown in parentheses is the number of pairs of data values used to compute each coefficient. The third number in each location of the table is a P-value which tests the statistical significance of the estimated correlations. P-values below 0.05 indicate statistically significant non-zero correlations at the 95% confidence level. The following pairs of variables have P-values below 0.05: QL and VOL, QL and RATIO, and VOL and RATIO.

Table4-18 Spearman Rank Correlation Matrix for MNL Variables

Spearman Rank Correlations

	QL	VOL	CONVOL	RATIO
QL		0.7117	0.0560	-0.7294
		(34)	(34)	(34)
		0.0000	0.7478	0.0000
VOL	0.7117		0.3265	-0.9276
	(34)		(34)	(34)
	0.0000		0.0607	0.0000
CONVOL	0.0560	0.3265		-0.0393
	(34)	(34)		(34)
	0.7478	0.0607		0.8215
RATIO	-0.7294	-0.9276	-0.0393	
	(34)	(34)	(34)	
	0.0000	0.0000	0.8215	

Correlation
(Sample Size)
P-Value

The model for predicting the maximum number of vehicles in the stopped queue is formulated as a Poisson regression equation with volume and conflicting volume as independent or explanatory variables.

$$\mathbf{Ln}\left(\mathbf{QL_{MNL}}\right) = \boldsymbol{\beta_0} + \boldsymbol{\beta_1} \times \left(\boldsymbol{CONVOL_{MNL}/VOL_{MNL}}\right)$$

Where

Ln = Natural Logarithm

QL = Maximum number of vehicles in the stopped queue

VOL = Approach volume for MNL lane configuration

CONVOL = Conflicting volume for MNL

β_0 = Constant

β_1 = Model coefficients corresponds to RATIO (CONVOL/VOL)

The Poisson regression model developed with coefficient values and corresponding statistical tests are explained below:

$$\mathbf{Ln}\left(\mathbf{QL}\right) = \mathbf{1.7934 - 0.025} \times \left(\boldsymbol{CONVOL/VOL}\right)$$

$$QL = e^{\left(\mathbf{1.7934 - 0.025} \times \left(\boldsymbol{CONVOL/VOL}\right)\right)}$$

Percentage of deviance explained by model = 69.404

Because the P-value for the model in the Analysis of Deviance table shown in Table 4-19 is less than 0.01, there is a statistically significant relationship between the variables at the 99% confidence level. In addition, the P-value for the residuals is greater than or equal to 0.10, indicating that the model is not significantly worse than the best possible model for this data at the 90% or higher confidence level.

Table 4-19 Estimated Regression Model for MNL Data

```
Poisson Regression

Dependent variable: QL
Factors:
   RATIO

Estimated Regression Model (Maximum Likelihood)
------------------------------------------------------------
                                       Standard    Estimated
Parameter                Estimate         Error   Rate Ratio
------------------------------------------------------------
CONSTANT                  1.79343       0.14742
RATIO                  -0.0247881    0.00684342     0.975517
------------------------------------------------------------

              Analysis of Deviance
---------------------------------------------------
Source          Deviance            Df      P-Value
---------------------------------------------------
Model           16.7513              1       0.0000
Residual        7.38449             32       1.0000
---------------------------------------------------
Total (corr.)   24.1357             33

Percentage of deviance explained by model = 69.4043
Adjusted percentage = 52.8314

Likelihood Ratio Tests
---------------------------------------------------------
Factor                   Chi-Square         Df   P-Value
---------------------------------------------------------
RATIO                    16.7513             1    0.0000
---------------------------------------------------------

Residual Analysis
---------------------------------
      Estimation       Validation
n     34
MSE   3.41565
MAE   0.820284
MAPE  23.642
ME    -0.00104352
MPE   -6.81105
```

The above Table also shows that the percentage of deviance in QL explained by the model equals 69.4043%. This statistic is similar to the usual R-Squared statistic. The adjusted percentage, which is more suitable for comparing models with different numbers of independent variables, is 52.8314%.

In determining whether the model can be simplified, notice that the highest P-value for the likelihood ratio tests is 0.0000, belonging to RATIO. Because the P-value is less than 0.01, that term is statistically significant at the 99% confidence level. Consequently, it is not advisable to remove any variables from the model. Plots of the predicted vs observed QL and their residuals are shown in Figure 4-11 and 4-12.

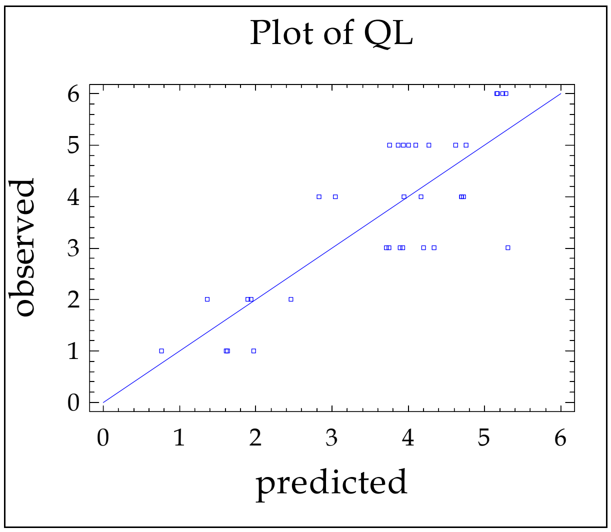


Figure 4-11 Plot of QL between Predicted Vs Observed QL for MNL

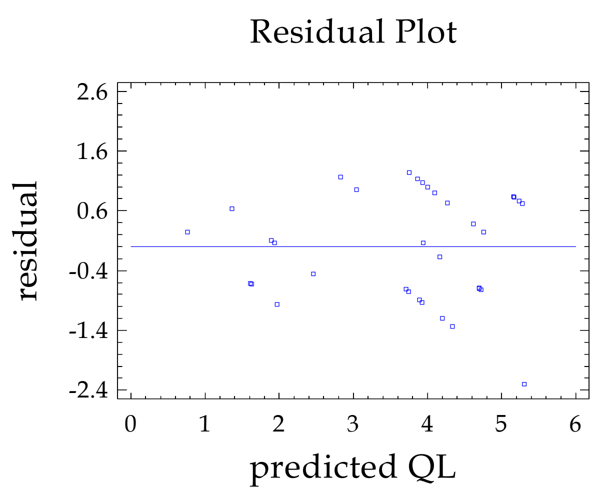


Figure 4-12 Residual Plot between Predicted QL and Residual for MNL

4.7 *Minor Right (MNR)*

Unfortunately, there are only 18 data points available for model development. A minimum of 2 vehicle in the queue and maximum of 5 vehicles are observed for this lane configuration. Table 4-20 shows summary statistics of data.

Table 4-20 Summary Statistics of MNR Data

	QL	VOL	CONVOL	VOLCONVOL
Count	18	18	18	18
Average	3.22222	56.1667	304.167	17124.6
Variance	1.12418	77.9118	7107.79	2.75093E7
Standard deviation	1.06027	8.82676	84.3077	5244.94
Minimum	2.0	44.0	141.0	6166.0
Maximum	5.0	71.0	412.0	23323.0
Range	3.0	27.0	271.0	17157.0
Stnd. skewness	0.871404	0.273118	-0.594507	-1.09394
Stnd. kurtosis	-0.702281	-0.991707	-0.862272	-0.426578

Only the developed model is shown with out further explanation of the model. This model is not tested for validation, as more data is required to obtain a significant model.

The model for predicting the maximum number of vehicles in the stopped queue is formulated as a Poisson regression equation with onlythe product of volume and conflicting volume as the independent or explanatory variable.

$$\mathbf{Ln}(\mathbf{QL}_{\mathbf{MNR}}) = \beta_0 + \beta_1 \times (VOL^{*}CONVOL)_{MNR}$$

Where

Ln = Natural Logarithm

QL = Maximum number of vehicles in the stopped queue

VOL = Approach volume for MNR lane configuration

CONVOL = Conflicting volume for MNR

β_0 = Constant

β_1 = Model coefficients corresponds to CONVOL

Poisson Regression is shown below:

$$\mathbf{Ln(QL) = 0.225058 + 0.00005316 \times (VOL^{*}\,CONVOL)}$$

$$\mathbf{QL = e^{(0.225058 + 0.00005316 \times (VOL^{*}CONVOL))}}$$

$$\mathbf{Percentage\ of\ deviance\ explained\ by\ model = 64.7485}$$

4.8 *Summary*

This chapter summarizes the step by step procedure in the development of a model. It is not practically possible to consider all explanatory variables in the model development. Only volume, conflicting volume, either their product or ratio between them, presence of a signal, and presence of a separate turn lane are assumed to have a significant impact on queue. Scatter diagrams of these identified variables show the random phenomenon which triggers the development of Poisson regression models. Model development steps are presented in a detailed manner for each lane configuration except for minor right turn configuration (MNR), due to presence of only 18 data points. Accuracy of these models is validated through data validation presented in the next chapter.

5 Model Validation

Validation gives the estimation of the model accuracy in predicting the maximum number of vehicles in the stopped queue. Validation can be done by using the subset of data prepared for model development process but not used for model building. If there is a possibility to collect separate data under similar conditions where the model is developed, those data will be preferred.

This chapter explains the data collection efforts for model validation. Collected raw data need to be processed to be used in model. For each model category, observed queue lengths are compared with predicted queues to check the consistency. Later, this step is extended to compare other methods with the existing methodology.

5.1 Data Collection

Intersections are chosen to cover good proportions of various lane configurations, geographic regions, functional classifications, and traffic conditions. In total, 25 intersections shown in Table C1 in the Appendix C are used for data collection.

Out of 25 intersections: 17 of them are from Region 1, and 8 are from Region 2. 12 (48%) are within the urban growth boundary, and the remaining 13 (52%) are rural. 24 intersections have either OR or US route as the major approach. Ten of the intersections have an upstream signal within 1000 ft. Thirteen intersections have either an exclusive or two-way left turn lane. Only 7 of the intersections have skewed approaches. None of the intersection approaches are off-set from the major approach. 17 intersections are 3 legged (68%), and 8 of them are 4 legged intersections. Finally, two intersections major approaches have flaredness.

Table C2 represents the time frame of the data collection. All the data were collected in 2010, on typical weekdays of either last week of August or the first week of September, but before the Labor Day Weekend.

5.2 *MJL*

There are 41 data points available for validation. Although there are many indicators of the strength of the model in predicting the intended behavior, residual analysis is primarily to document the accuracy. The difference between the observed and model predicted value is used to assess the model performance. The following Table 5-1 shows the difference between various models. 39% of observed values are exactly predicted by the new model. Gard's equation and the Two-Minute Rule are behind the new model with 22% and 20% matching. If the error is relaxed to either +1 or -1 vehicles, 79% are matched by the new model and nearly the same percentage of match by Gard's and Two-Minute Rule. HCM consistently yields lower estimates. None of the model outputs underestimate queue length by more than 3 vehicles.

Table 5-1 Comparison of Queue Length Estimation Differences for MJL

Type of Estimation	Difference (against Observed)	**Observed - 2 min rule**		**Observed - HCM method**		**Observed - Gard's Equation**		**Observed - Model**	
		% Obs	Cum % of Obs	% Obs	Cum % of Obs	% Obs	Cum % of Obs	% Obs	Cum % of Obs
Over Estimated	< = -5	0%	2%	0%	0%	0%	12%	5%	17%
	-4	0%		0%		0%		0%	
	-3	0%		0%		7%		2%	
	-2	2%		0%		5%		10%	
Acceptable	-1	20%	**61%**	0%	**29%**	20%	**56%**	20%	**78%**
	0	**20%**		**0%**		**22%**		**39%**	
	1	22%		29%		15%		20%	
Under Estimated	2	17%	37%	29%	71%	10%	32%	5%	5%
	3	12%		12%		15%		0%	
	4	0%		17%		2%		0%	
	> = 5	7%		12%		5%		0%	

5.3 *MNLTR*

Overall 15 observations are available for this category. 33% of predicted queue lengths are exactly matched with the observed values. Two-Minute rule matched 13% of the observations. The new model matches 60% with a variation of a single vehicle on both sides. For the same variation 53% are matches for the Two-Minute Rule. Gard's equation is overestimating the queues, while the HCM is under predicting. The variation of the error is shown through both Table 5-2.

Table 5-2 Comparison of Queue Length Estimation Differences for MNLTR

Type of Estimation	Difference (against Observed)	Observed - 2 min rule		Observed - HCM method		Observed - Gard's Equation		Observed - Model	
		% of Obs	Cum % of Obs	% of Obs	Cum % of Obs	% of Obs	Cum % of Obs	% of Obs	Cum % of Obs
Over Estimated	< = -5	0%	0%	0%	0%	60%	73%	0%	20%
	-4	0%		0%		0%		0%	
	-3	0%		0%		7%		7%	
	-2	0%		0%		7%		13%	
Acceptabl e	-1	7%	53%	7%	7%	13%	20%	0%	60%
	0	13%		0%		0%		33%	
	1	33%		0%		7%		27%	
Under Estimated	2	13%	47%	27%	93%	7%	7%	13%	20%
	3	33%		47%		0%		7%	
	4	0%		13%		0%		0%	
	> = 5	0%		7%		0%		0%	

5.4 *MNLR*

Only 25% of predicted queue lengths are exactly matched with the observed queues, while 42% are matched by the Two-Minute Rule. With a variation of 1 vehicle, almost 92% are matched by Two-Minute Rule. For the same situation, 67% are matched by the developed model. The HCM method is matching 58% of the time and 33% are matched by Gard's equation. The results are shown inTable 5-3. The predicted model seems to be underestimating. One reason may be only 12 data points are available for data validation purpose. As the sample size increases, there is a good chance of model convergence with the observed values.

Table 5-3 Comparison of Queue Length Estimation Differences for MNLR

Type of Estimation	Difference (against Observed)	Observed - 2 min rule		Observed - HCM method		Observed - Gard's Equation		Observed - Model	
		% of Obs	Cum % of Obs	% of Obs	Cum % of Obs	% of Obs	Cum % of Obs	% of Obs	Cum % of Obs
Over Estimated	< = -5	0%	0%	8%	8%	50%	58%	0%	0%
	-4	0%		0%		0%		0%	
	-3	0%		0%		0%		0%	
	-2	0%		0%		8%		0%	
Acceptable	-1	0%	92%	8%	58%	17%	33%	25%	67%
	0	42%		8%		8%		25%	
	1	50%		42%		8%		17%	
Under Estimated	2	8%	8%	25%	33%	8%	8%	25%	33%
	3	0%		0%		0%		0%	
	4	0%		0%		0%		8%	
	> = 5	0%		8%		0%		0%	

5.5 *MNL*

Only 10 observations are available for the MNL lane configuration. 50% are exactly matched for the developed model with 90% for one vehicle variation. 70% are matched by Two-Minute Rule, and 60% by Gard's Equation. The HCM methodology underestimates the queue lengths. Error distribution is shown in Table 5-4.

Table 5-4 Comparison of Queue Length Estimation Differences for MNR

Type of Estimation	Difference (against Observed)	**Observed - 2 min rule**		**Observed - HCM method**		**Observed - Gard's Equation**		**Observed - Model**	
		% of Obs	Cum % of Obs	% of Obs	Cum % of Obs	% of Obs	Cum % of Obs	% of Obs	Cum % of Obs
Over Estimated	< = -5	0%	0%	0%	0%	0%	40%	0%	10%
	-4	0%		0%		0%		0%	
	-3	0%		0%		40%		0%	
	-2	0%		0%		0%		10%	
Acceptable	-1	0%	**70%**	0%	**0%**	20%	**60%**	40%	**90%**
	0	**10%**		**0%**		**20%**		**50%**	
	1	60%		0%		20%		0%	
Under Estimated	2	20%	30%	30%	100%	0%	0%	0%	0%
	3	10%		50%		0%		0%	
	4	0%		20%		0%		0%	
	> = 5	0%		0%		0%		0%	

5.6 *Summary*

The developed models are predicting consistently closer values. Although error varies from one lane group model to other, the percent of matching varies from 60% to 90% with error of ±1 vehicle. Importantly, the data set available for most of the lane groups is not sufficient for analysis. As more data is made available, the predictions should be closer.

6 Model Sensitivity Analysis

The sensitivity of the model with respect to changes in the independent or explanatory variables gives an idea how well models will perform for varying conditions. As such, one can check the model reasonableness by the sign of the change of dependent variable for the corresponding change in the explanatory variables. The magnitude of the change will also be obtained. In addition, one can check the limit(s) or range(s) of the independent variables where the model will adequately explain the queue behavior.

This chapter presents the explanations for sensitivity of the models developed in Chapter 4. For each model, limits are set for the independent variables based on the outcome of the model queue lengths which indicate the model stability.

6.1 MJL

Volume (VOL), conflicting volume (CONVOL), presence of upstream signal within 1000 feet of the intersection (SIGNAL), and presence of left turn lane (LT) are considered for modeling queue lengths for major left turns. The developed model is shown below:

$$\mathbf{Ln(QL) = 0.3925 + 0.0059 \times VOL + 0.00104 \times CONVOL + 0.49 * SIGNAL - 0.81 * LT}$$

$$QL = e^{\left(\mathbf{0.3925 + 0.0059 \times VOL + 0.00104 \times CONVOL + 0.49 * SIGNAL - 0.81 * LT}\right)}$$

Percentage of deviance explained by model = 66.959

As VOL and CONVOL increases, QL increases. Presence of an upstream signal increases QL due to vehicle arrivals in platoon. If there is a separate left turn lane, either exclusive or two-way left turn lane, it decreases queue length compared to a shared left turn lane. The bounds for VOL and CONVOL are set by drawing a 2-D contour map of the QL.

As shown in Figure 6-1, as the VOL and CONVOL pair reaches MUTCD 2009 edition warranted volumes given in Table A1, for condition A (Minimum vehicular volume for 2 or more lanes on major street and 2 or more lanes on minor street), a maximum of fifteen vehicles are predicted to be in the stopped queue. For condition B (interruption of continuous traffic condition for the same lane configuration), eleven vehicles at maximum are in the stopped queue condition.

Volumes exceeding these points trigger a substantial increase in queue lengths. As such, unacceptable queue lengths are obtained for volumes greater than 300 VPH and corresponding conflicting volumes of 2000 VPH. Beyond these points the model is unstable for queue length prediction. Caution - the outcomes shown in Figure 6-1 are obtained by assuming an upstream signal and shared left turn lane, which will test the worst possible scenario.

6.2 MNLTR

QL on a single shared left, through, and right turn movement on minor approach is affected by volume, conflicting volume, and their product.

$$\mathbf{Ln(QL) = -0.7844 + 0.01636 \times VOL + 0.0006 \times CONVOL - 0.0000043 \times (VOL * CONVOL)}$$

$$QL = e^{(\mathbf{-0.7844+0.01636 \times VOL+0.0006 \times CONVOL-0.0000043 \times (VOL * CONVOL)})}$$

Percentage of Deviation Explained by Model = 71.643

Signs of the independent variables show reasonableness in the models, as a positive change in the VOL and CONVOL increases QL. But the occurrence of VOL and the corresponding CONVOL as a product triggers a decrease in QL, due to the fact that a lower approach volume may not yield larger queue lengths, rather triggers more waiting time in the stopped queue.

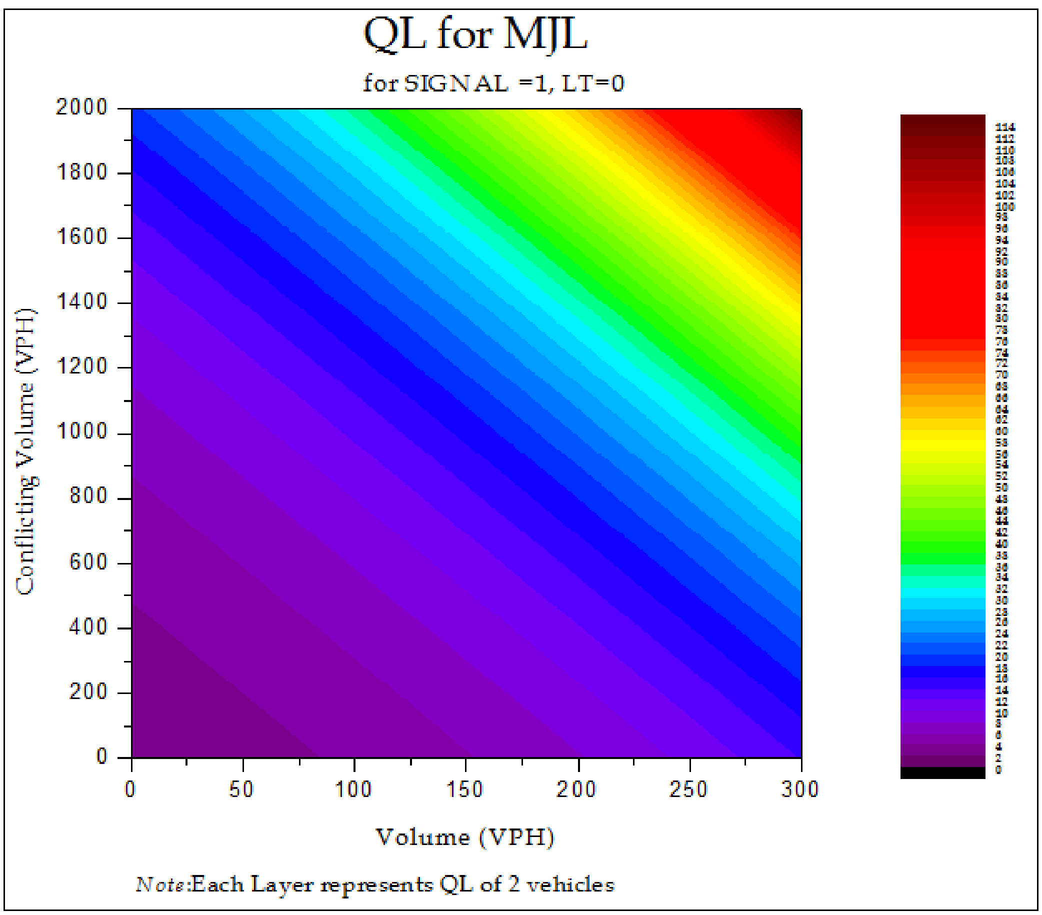


Figure 6-1 Plot of QL for VOL and CONVOL at Major Left Turn (MJL)

As approach volume increase to capacity, approach volume has a higher impact on queue than the conflicting volume. This behavior can be seen in Figure 6-2. As Highway Capacity Manual (HCM) limits the maximum 95th percentile queue lengths to be 100, the limits for VOL and CONVOL are set such that predicted queue length is not exceeding 100 vehicles. This is not a bad idea because, if the volume on the subject approach and the corresponding volume on major street reach the MUTCD Chapter 4C, section 4C.02 thresholds, it will warrant the installation of a signal.

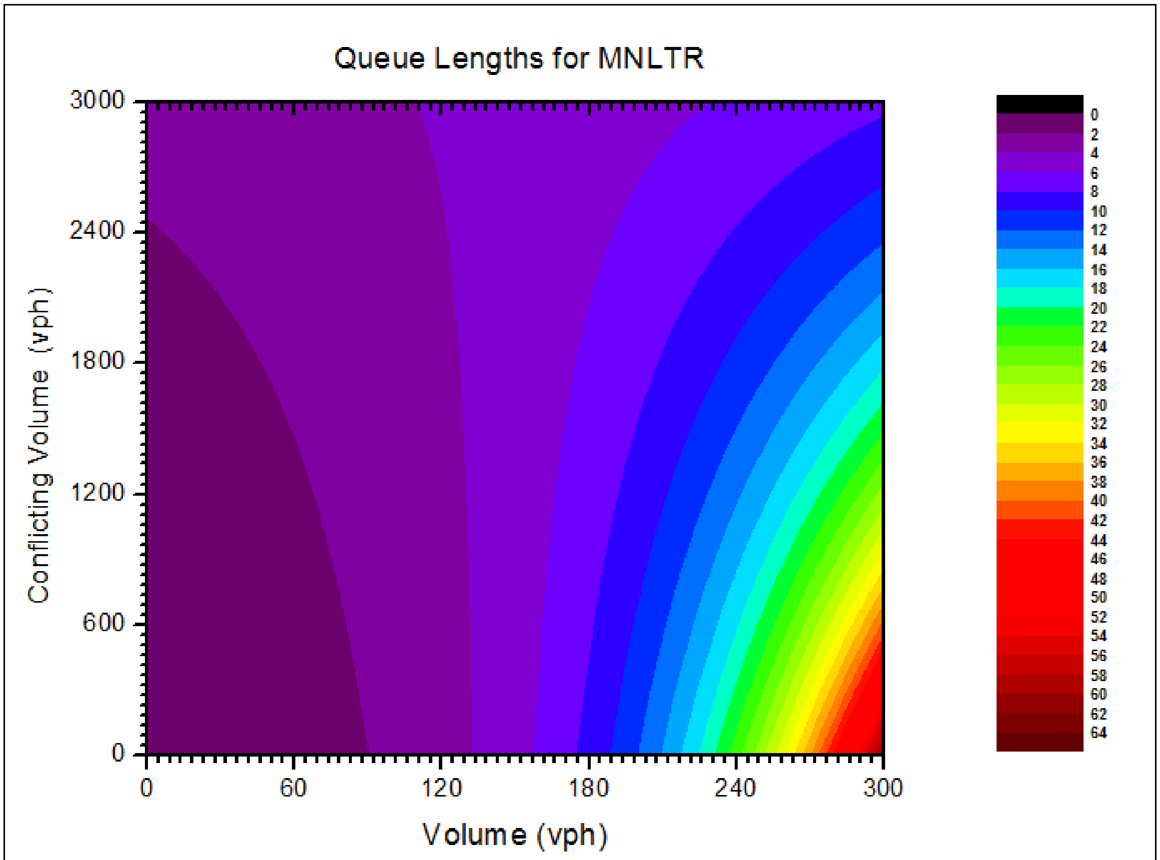


Figure 6-2 Plot of QL for VOL and CONVOL at Minor LTR (MNLTR)

6.3 MNLR

QL on a single shared left, through, and right turn movement on minor approach is affected by volume, conflicting volume, and their product.

$$\mathbf{Ln(QL) = -0.6319 + 0.0173 \times VOL + 0.00066 \times CONVOL - 0.000007913\,(VOL * CONVOL)}$$

$$QL = e^{(\mathbf{-0.6319 + 0.0173 \times VOL + 0.00066 \times CONVOL - 0.000007913\,(VOL * CONVOL)})}$$

Percentage of deviance explained by model = 69.25

Signs of the independent variables show reasonableness in the models, as a positive change in the VOL and CONVOL increases QL. But the occurrence of VOL and the corresponding CONVOL as a product triggers a decrease in QL, due to the fact that lower approach volume may not yield larger queue lengths, rather triggers more waiting time in the stopped queue. As approach volume increase to capacity, approach volume has higher impact on queue than the conflicting volume. This behavior can be seen in Figure 6-3. As Highway Capacity Manual (HCM) limits the maximum 95th percentile queue lengths to be 100, the limits for VOL and CONVOL are set such that predicted queue length is not exceeding 100 vehicles. This is not a bad idea because, if the volume on the subject approach and the corresponding volume on major street reach the MUTCD Chapter 4C, section 4C.02 thresholds, it will warrant the installation of a signal.

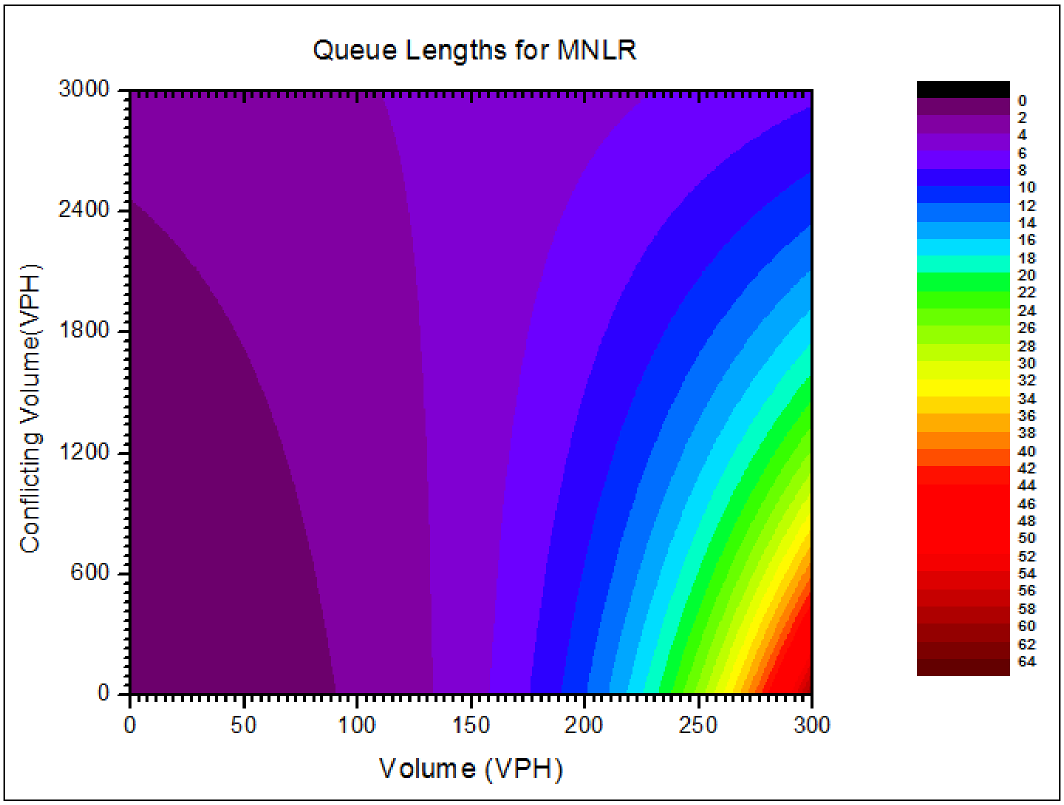


Figure 6-3 Plot of QL for VOL and CONVOL at Minor LR (MNLR)

6.4 MNL

Poisson Regression developed is shown below:

$$\mathbf{Ln(QL) = 1.7934 - 0.025 \times} (CONVOL/VOL)$$

$$QL = e^{(1.7934 - 0.025 \times (CONVOL/VOL))}$$

Percentage of deviance explained by model = 69.404

As the volume and conflicting volume increases, QL increases as shown in Figure 6-4. Beyond the volume of 300 VPH and conflicting volume of 3000 VPH, queue lengths are not realistically represented by the model. These are used as the limits for the models. MNL model is developed using only 34 data points, which may limit the strength of the model. This is evident from MNL model form, which has 1.7934[13] as a constant. So, the output is greatly affected by a constant value rather than variation of explanatory variables. MNL model needs to be improved by collecting more data.

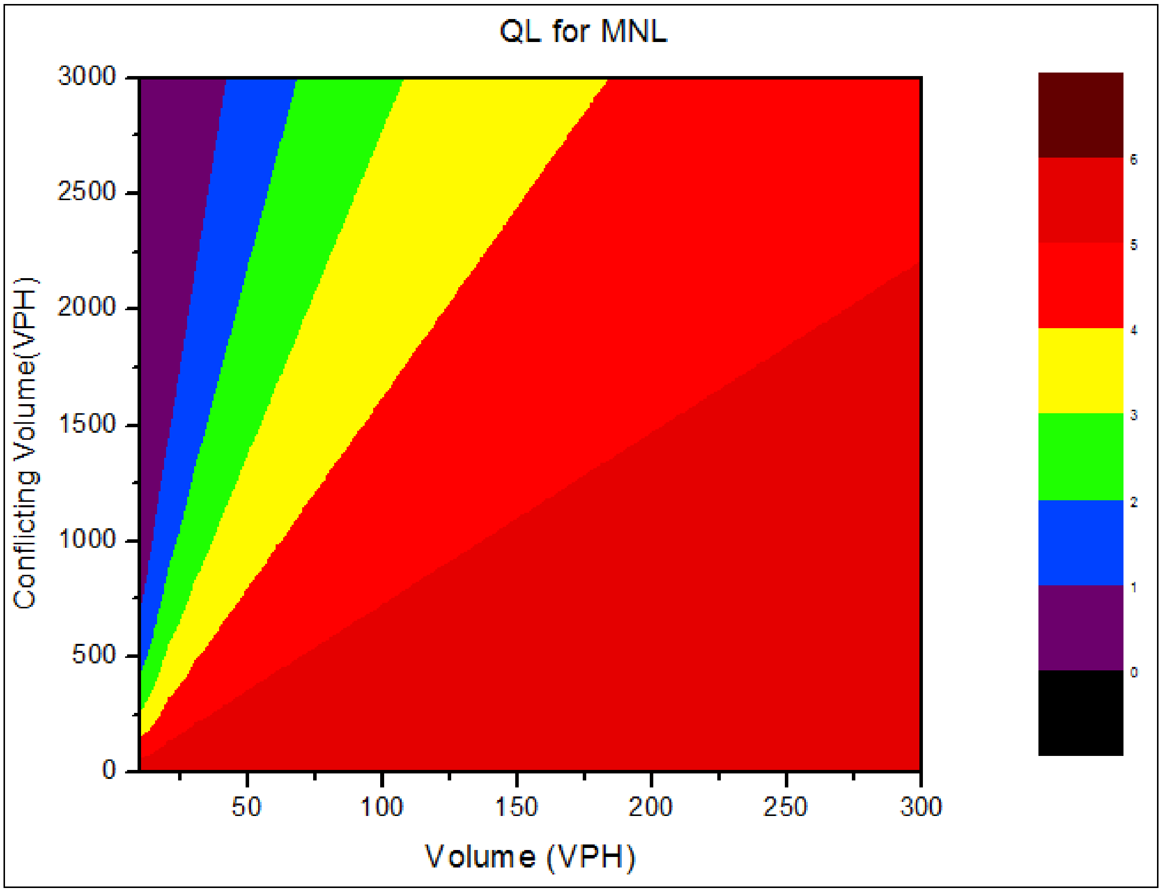


Figure 6-4 Plot of QL for VOL and CONVOL at Minor Left (MNL)

6.5 *Summary*

Sensitivity Analysis is used to test the model for the all possible ranges of the input variables. During the model development only certain range for input variables are represented. Sensitivity analysis gives an opportunity, as explained in the above sections, to test the model behavior for most of the combinations of inputs. Following table summaries models for each lane groups with the limitation to the input variables.

---

[13] In the absence of VOL, and CONVOL , QL = $e^{1.7934} \equiv 6$ vehicles

7 Summary, Conclusions & Scope for Future Study

7.1 Summary

The following table summarizes the developed models, the applicable ranges for input data, and the percentage of deviation for each model:

VOL = Traffic flow rate on the subject approach in vehicles per hour

CONVOL = Conflicting traffic flow rate calculated according HCM methodology, expressed as vehicles per hour

SIGNAL = Presence of Upstream Signal with in 1000 ft of the intersection, Applicable for Major Left Turn only, 1 if there is a signal, otherwise 0

LT = Presence of a separate left turn lane, Applicable for Major Left Turn only (1 if there is an exclusive left turn lane/median left turn lane/ two-way left turn lane, otherwise 0)

Lane Group	Model Equation & Ranges	Percent of Deviation
MJL	$QL = e^{(0.3925+0.0059\times VOL+0.00104\times CONVOL+0.49*SIGNAL-0.81*LT)}$ VOL = (0, 300] ; CONVOL = (0,2000] SIGNAL = 0 or 1 ; LT = 0 or 1	66.96
MNLTR	$QL = e^{(-0.7844+0.01636\times VOL+0.0006\times CONVOL-0.0000043\times(VOL*CONVOL))}$ VOL = (0, 300] ; CONVOL = (0,3000]	71.64
MNLR	$QL = e^{(-0.6319+0.0173\times VOL+0.00066\times CONVOL-0.000007913\times(VOL*CONVOL))}$ VOL = (0, 300] ; CONVOL = (0,3000]	69.25

7.2 Conclusions

Following conclusions are made from this study:

- The Two-Minute Rule performs better than other existing methods except for the Major Left Turn (MJL) configuration where Gard's Equation does better for the reason that the Two-Minute Rule does not include opposing volume.
- Existing methods are not exactly predicting queue lengths for more than 50 percent of the cases.
- A Poisson regression model is developed to improve the queue length estimations.
- The developed models accurately predict more than 65 percent of the cases.

- Improvements to the model predictions may achieved by expanding the data sample size.
- As presented these models are a proto type, one may not adopt this model until they are validated for a wide range of conditions.

7.3 Future Study Scope

It is a well known fact that model development is an iterative process, as such there is always room for improvement.

- The current study may be elaborated with expanded data sets. Special attention need to be given to collect MNL lane configuration data to refine the model performance.
- A good proportion of data groups are adopted for study, but expansion to different geographic regions, highway networks, traffic loads as time of day and seasonal variations, multi-year data sets is possible
- The model form may be scoped to suit various traffic and queue conditions
- The methods of model development may be varied to capture dynamics of queues
- Even explanatory variables and their combination may be studied

Appendix A – HCM & MUTCD Exhibits

Figure A1 Traffic Streams at a Two-Way STOP Controlled Intersection
(*Source: HCM, Exhibit 17-3*)

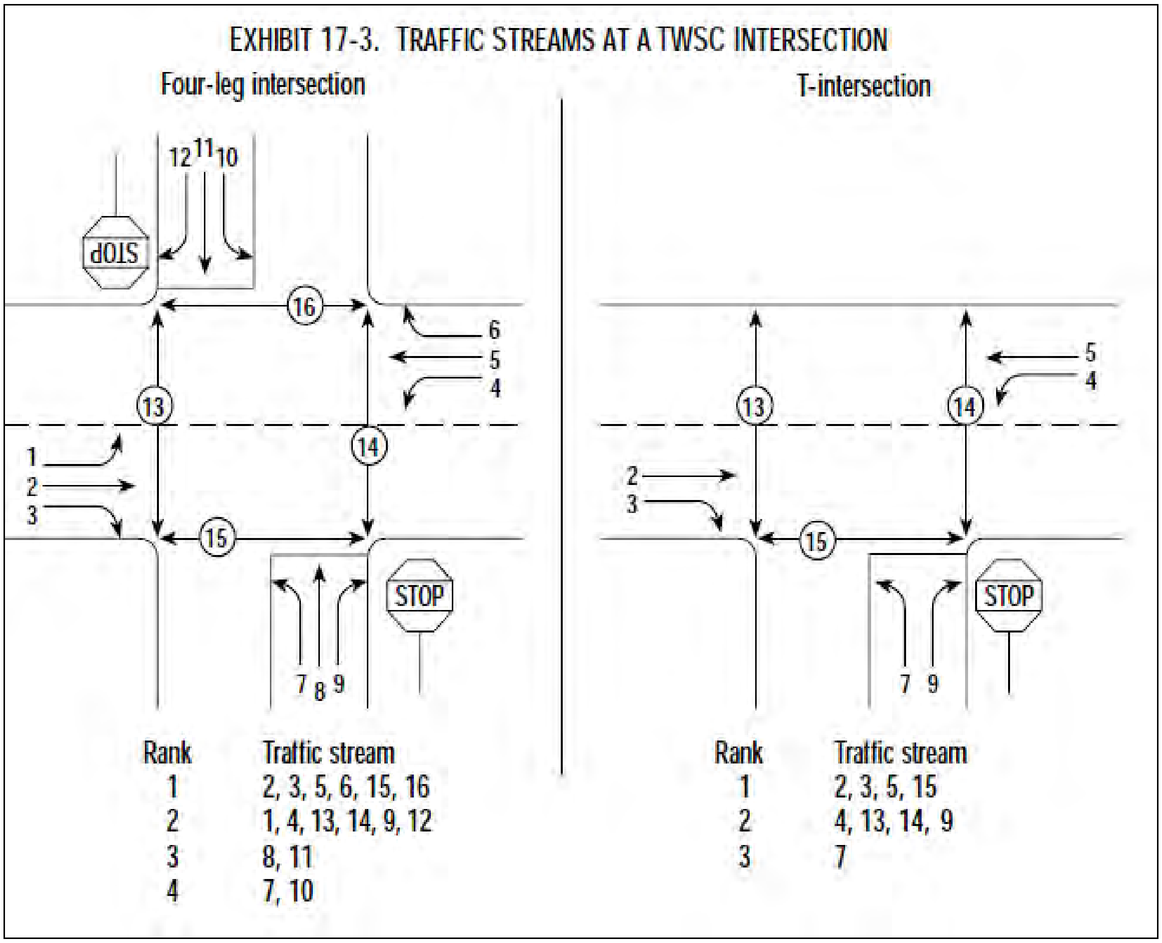
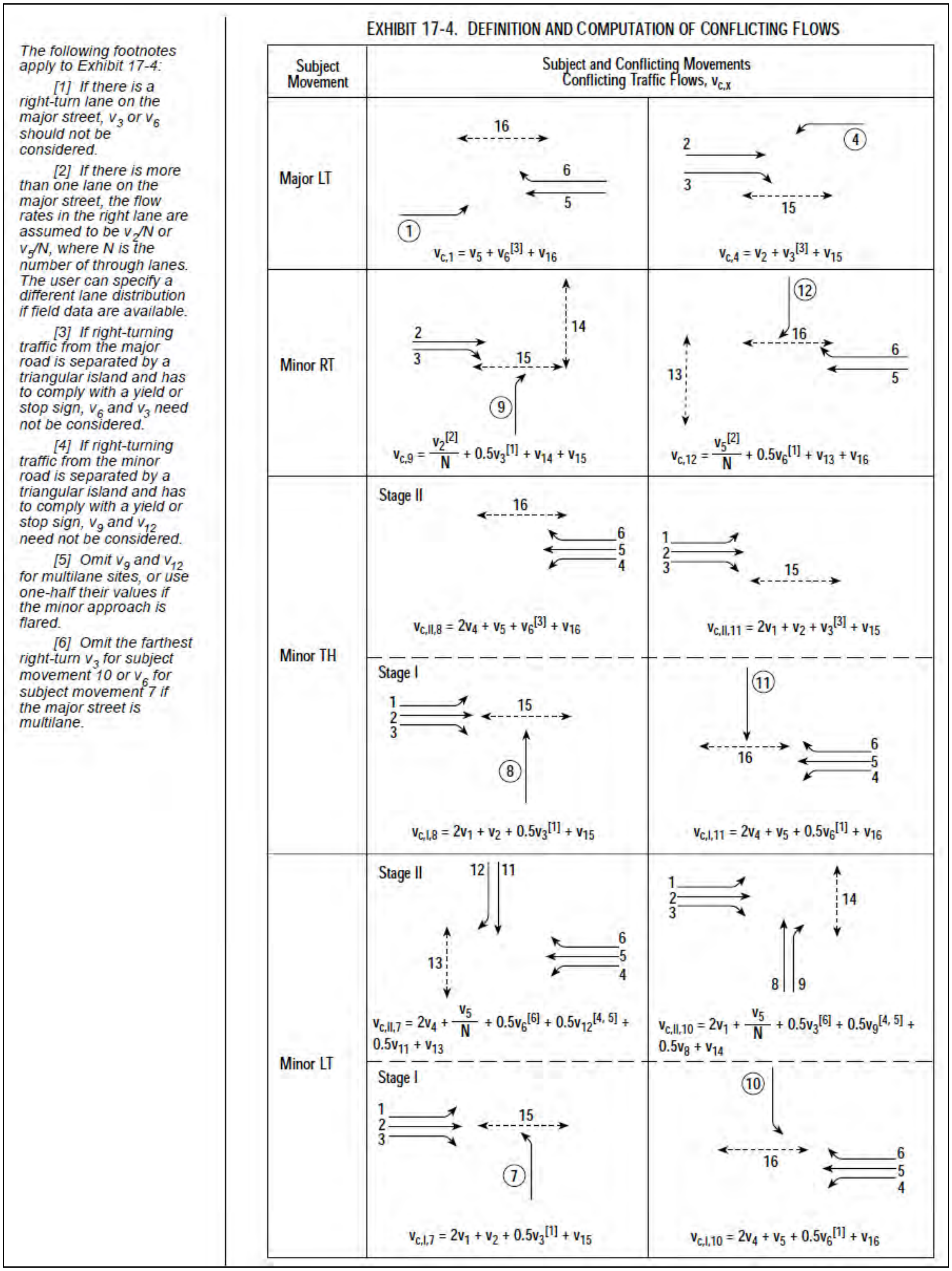


Figure A2 Method for Computing Lane Group Conflicting Flow Rates
(*Source: HCM, Exhibit 17-4*)



The following footnotes apply to Exhibit 17-4:

[1] If there is a right-turn lane on the major street, v_3 or v_6 should not be considered.

[2] If there is more than one lane on the major street, the flow rates in the right lane are assumed to be v_2/N or v_5/N, where N is the number of through lanes. The user can specify a different lane distribution if field data are available.

[3] If right-turning traffic from the major road is separated by a triangular island and has to comply with a yield or stop sign, v_6 and v_3 need not be considered.

[4] If right-turning traffic from the minor road is separated by a triangular island and has to comply with a yield or stop sign, v_9 and v_{12} need not be considered.

[5] Omit v_9 and v_{12} for multilane sites, or use one-half their values if the minor approach is flared.

[6] Omit the farthest right-turn v_3 for subject movement 10 or v_6 for subject movement 7 if the major street is multilane.

EXHIBIT 17-4. DEFINITION AND COMPUTATION OF CONFLICTING FLOWS

Subject Movement	Subject and Conflicting Movements Conflicting Traffic Flows, $v_{c,x}$	
Major LT	$v_{c,1} = v_5 + v_6^{[3]} + v_{16}$	$v_{c,4} = v_2 + v_3^{[3]} + v_{15}$
Minor RT	$v_{c,9} = \frac{v_2^{[2]}}{N} + 0.5v_3^{[1]} + v_{14} + v_{15}$	$v_{c,12} = \frac{v_5^{[2]}}{N} + 0.5v_6^{[1]} + v_{13} + v_{16}$
Minor TH	Stage II: $v_{c,II,8} = 2v_4 + v_5 + v_6^{[3]} + v_{16}$	$v_{c,II,11} = 2v_1 + v_2 + v_3^{[3]} + v_{15}$
	Stage I: $v_{c,I,8} = 2v_1 + v_2 + 0.5v_3^{[1]} + v_{15}$	$v_{c,I,11} = 2v_4 + v_5 + 0.5v_6^{[1]} + v_{16}$
Minor LT	Stage II: $v_{c,II,7} = 2v_4 + \frac{v_5}{N} + 0.5v_6^{[6]} + 0.5v_{12}^{[4,5]} + 0.5v_{11} + v_{13}$	$v_{c,II,10} = 2v_1 + \frac{v_5}{N} + 0.5v_3^{[6]} + 0.5v_9^{[4,5]} + 0.5v_8 + v_{14}$
	Stage I: $v_{c,I,7} = 2v_1 + v_2 + 0.5v_3^{[1]} + v_{15}$	$v_{c,I,10} = 2v_4 + v_5 + 0.5v_6^{[1]} + v_{16}$

Figure A3 Lane Groups (*Source: HCM, Exhibit 16-5*)

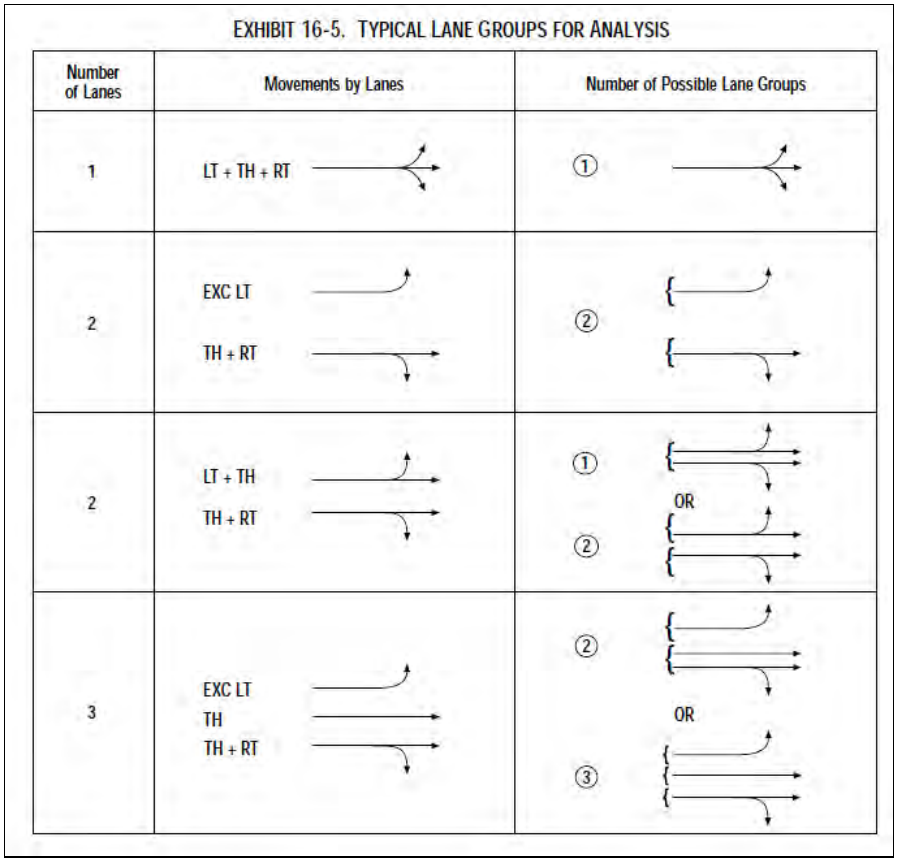


Table A1 MUTCD Warrant 1 Table (Source: *2009 MUTCD, section 4C.02*)

Table 4C-1. Warrant 1, Eight-Hour Vehicular Volume

Condition A—Minimum Vehicular Volume

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	500	400	350	280	150	120	105	84
2 or more	1	600	480	420	336	150	120	105	84
2 or more	2 or more	600	480	420	336	200	160	140	112
1	2 or more	500	400	350	280	200	160	140	112

Condition B—Interruption of Continuous Traffic

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	750	600	525	420	75	60	53	42
2 or more	1	900	720	630	504	75	60	53	42
2 or more	2 or more	900	720	630	504	100	80	70	56
1	2 or more	750	600	525	420	100	80	70	56

[a] Basic minimum hourly volume
[b] Used for combination of Conditions A and B after adequate trial of other remedial measures
[c] May be used when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000
[d] May be used for combination of Conditions A and B after adequate trial of other remedial measures when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000

Appendix B – Data Collection Sites for Model Development

Table B1 Description of Study Intersection Used for Model Development

Sl. No	Name	City	Region	Geographic Area(R/U)	No of Legs (3/4)	Presence of Skewness/ Presence of curve at the intersection	Presence of Offset on Minor Approaches	No of Lanes on Major Street (2/3/4/5)	Functional Classification of the Major ST	Functional Classification of the Minor ST	Major Approach: Approach Speed	Major Approach: Presence of Upstream Signal	Major Approach: Presence of TWLT Lane/ Median Left Turn Lane	Major Approach: Presence of Right Turn Lane	Major Approach: Presence of Flared Right Turn Lane
1	Nevada ST and Oak ST	Ashland	3	U	4	0	0	2	C	L	25	0	0	0	0
2	Tolman Creek Rd. and Mistletoe Rd + Takelma Wy	Ashland	3	U	4	1	0	2	C	L	25	0	0	0	0
3	OR 99 E and NE Carl Rd	Woodburn	2	U	3	1	0	5	OR	C	45	0	1	0	0
4	US 101 and 20th ST	Reedsport	3	U	4	0	0	4	US	C	30	0	0	0	0
5	US 730 and Umatilla River Rd	Umatilla	5	U	3	0	0	3	US	C	25	0	1	0	0
6	OR 99 E and E Cleveland ST	Woodburn	2	U	4	1	1	4	OR	C	35	0	0	0	0
7	OR 214 / OR 211 and Lawson Avenue	Woodburn	2	U	3	0	0	3	OR	C	30	1	1	0	0
8	OR 99 E with Industrial Ave and Mc Laren School Rd NE	Woodburn	2	R	4	1	0	5	OR	L	45	1	1	0	1
9	US 97 vs Lakeport Blvd	Klamath Falls	4	R	3	1	0	3	US	L	50	0	1	0	0
10	OR 99 E and Food Services Road	Woodburn	2	R	3	1	0	2	OR	L	45	0	0	0	0
11	US 20 and 17th ST	Philomath	2	U	4	0	0	4	US/OR	C	30	1	0	0	0
12	OR 20 and Dead Indian Memorial Road	Ashland	3	R	3	1	0	2	OR	C	35	0	0	0	1
13	US 395 and Power City Rd	Umatilla	5	R	4	1	1	5	US/OR	C	50	0	1	0	1
14	OR 99 and (W Hersey Road & Wimer ST)	Ashland	3	U	4	1	1	4	OR	C	25	0	0	0	0
15	US 20 (OR 34) and (S 7th ST & N 7th ST)	Philomath	2	U	3	0	0	2	US	C	35	0	0	0	0

Note: U=Urban, R=Rural, OR= Oregon Route, US = US Route, C= Collector, L=Local, C/L = Collector/Local, N/A = Not available, 0/1 = Flag showing Yes/No

Table B2 Periods of Data Collection Used for Model Development

Sl. No	Name	City	Region	Data Collection Date	Data Collection Day	Data Collection Time	Data Collection Duration (Hrs)
1	Nevada ST and Oak ST	Ashland	3	9/14/2009	Monday	3 - 6 PM	3
2	Tolman Creek Rd. and Mistletoe Rd + Takelma Wy	Ashland	3	9/16/2009	Wednesday	4 - 6 PM; 7-8 AM	3
3	OR 99 E and NE Carl Rd	Woodburn	2	2/23/2010	Tuesday	3 - 6 PM ; 6-9 AM	6
4	US 101 and 20th ST	Reedsport	3	11/4/2009	Wednesday	3 - 6 PM ; 7:30-9 AM	4.5
5	US 730 and Umatilla River Rd	Umatilla	5	1/13/2010	Wednesday	3 - 6 PM ; 7:30-9 AM	4.5
6	OR 99 E and E Cleveland ST	Woodburn	2	2/23/2010	Tuesday	3 - 6 PM ; 6-9 AM	6
7	OR 214 / OR 211 and Lawson Avenue	Woodburn	2	3/1/2010	Monday	3 - 6 PM ; 6-9 AM	6
8	OR 99 E with Industrial Ave and Mc Laren School Rd NE	Woodburn	2	2/22/2010	Monday	3 - 6 PM ; 6-9 AM	6
9	US 97 vs Lakeport Blvd	Klamath Falls	4	4/7/2010	Wednesday	3 - 6 PM	3
10	OR 99 E and Food Services Road	Woodburn	2	2/22/2010	Monday	3 - 6 PM ; 6-9 AM	6
11	US 20 and 17th ST	Philomath	2	3/9/2010	Tuesday	3 - 6 PM ; 6-9 AM	6
12	OR 20 and Dead Indian Memorial Road	Ashland	3	9/15/2009	Tuesday	3 - 6 PM	3
13	US 395 and Power City Rd	Umatilla	5	1/19/2010	Tuesday	3 - 6 PM ; 6-9 AM	6
14	OR 99 and (W Hersey Road & Wimer ST)	Ashland	3	9/14/2009	Monday	2 - 6 PM	4
15	US 20 (OR 34) and (S 7th ST & N 7th ST)	Philomath	2	3/9/2009	Tuesday	3 - 6 PM ; 6-9 AM	6

Appendix C - Data Collection Sites for Model Validation

Table C1 Description of Study Intersections Used for Data Validation

Sl.No	Name	Region	No of Legs (3/4)	Presence of Skewness/ Presence of curve	Presence of Offset on Minor Approaches	Geographic Area(R/U/SU)	No of Lanes on Major Street (2/4/4/5)	Functional Classification of the Major ST	Functional Classification of the Minor ST	Major Approach: Approach Speed	Major Approach: Presence of Upstream Signal	Major Approach: Presence of TWLT / Exclusive Left	Major Approach: Presence of Right Turn Lane	Major Approach: Presence of Flared Right Turn Lane
1	US 30 and NE 185th Ave	1	3	1	0	U	4	US	C	40	1	0	0	0
2	NE 185th Ave and NE Portal Way	1	4	1	0	U	4	C	L	40	0	1	0	1
3	US 30 and NE 172ND PL	1	3	0	0	U	4	US	L	N/A	0	0	0	0
4	US 26 and SE 79th Ave	1	4	0	0	U	5	US	C/L	35	1	1	0	0
5	US 26 / SE 99th Ave	1	3	0	0	U	2	US	L	35	1	0	0	0
6	US 26 / SE 130th Ave	1	3	0	0	U	2	US	L	35	1	0	0	0
7	OR 99 E / SE HULL AVE	1	4	0	0	U	5	OR	C	40	1	1	0	0
8	OR 99 E / SE VINEYARD RD	1	4	0	0	U	5	OR	C	40	1	1	0	0
9	OR 99 E / SE HOLLY AVE	1	3	0	0	U	5	OR	L	40	1	1	0	0
10	OR 224 & OR 212 / SE 106th ST	1	3	0	0	U	5	OR	L	N/A	1	1	0	0
11	OR 224 & OR 212 / SE 114th AVE	1	3	0	0	U	5	OR	L	N/A	0	1	0	0
12	OR 224 & OR 212 / SE 122nd AVE	1	3	0	0	U	5	OR	C	45	1	1	0	0
13	OR 22 W / PERRYDALE RD	2	3	1	0	R	2	OR	L	N/A	0	1	0	0
14	OR 22 W / DOAKS FERRY RD	2	3	1	0	R	5	OR	C	N/A	0	1	0	0
15	OR 221 / DOAKS FERRY RD NW	2	3	1	0	R	2	OR	C	N/A	1	1	0	0
16	OR 221 / SE PALMER CREEK RD	2	3	1	0	R	2	OR	C	50	0	0	0	0
17	OR 219/ SE FARMINGTON RD	1	3	0	0	R	3	OR	OR	55	0	0	1	0
18	OR 219/ SE SCHOLLS FERRY RD(OR 210)	1	3	1	0	R	2	OR	OR	45	0	0	0	1
19	OR 219/ BELL RD / N VALLE RD	2	4	0	0	R	2	OR	C/L	N/A	0	0	0	0
20	OR 219/ SW UNGER RD	1	3	0	0	R	3	OR	C	45	0	1	0	0
21	OR 219/ Tongue Ln	1	3	0	0	R	4	OR	C	N/A	0	1	0	0
22	OR 211 / S Kropff Rd or Canby - Marquam Rd	2	4	0	0	R	2	OR	OR	N/A	0	0	1	0
23	OR 211 / S Meridian Rd	2	4	0	0	R	2	OR	C	55	0	0	0	0
24	OR 214 / Howell Prairie Rd	2	3	0	0	R	2	OR	C	40	0	0	0	0
25	OR 213 / S CARUS RD	1	4	0	0	R	2	OR	C	20	0	0	0	0

Note:
U=Urban, R=Rural, OR= Oregon Route, US = US Route, C= Collector, L=Local, C/L = Collector/Local,
N/A = Not available, 0/1 = Flag showing Yes/No

Table C2 Duration of Traffic Counts Used for Data Validation

Sl.No	Name	Region	No of Legs (3/4)	Duration	Date	Day	Time of Day
1	US 30 and NE 185th Ave	1	3	1 Hr	8/24/2010	Tuesday	8:25 AM - 9:25 AM
2	NE 185th Ave and NE Portal Way	1	4	1 Hr	8/24/2010	Tuesday	9:40 AM - 10:41 AM
3	US 30 and NE 172ND PL	1	3	1 Hr	8/24/2010	Tuesday	11:07 AM - 12:07 AM
4	US 26 and SE 79th Ave	1	4	1 Hr	8/24/2010	Tuesday	1:22 PM - 2:22 PM
5	US 26 / SE 99th Ave	1	3	1 Hr	8/24/2010	Tuesday	2:40 PM - 3:40 PM
6	US 26 / SE 130th Ave	1	3	1 Hr	8/24/2010	Tuesday	4:05 PM - 5:05 PM
7	OR 99 E / SE HULL AVE	1	4	1 Hr	8/25/2010	Wednesday	8:20 AM - 9:20 AM
8	OR 99 E / SE VINEYARD RD	1	4	1 Hr	8/25/2010	Wednesday	9:49 AM - 10:49 AM
9	OR 99 E / SE HOLLY AVE	1	3	1 Hr	8/25/2010	Wednesday	11:01 AM - 12:01 AM
10	OR 224 & OR 212 / SE 106th ST	1	3	1 Hr	8/25/2010	Wednesday	1:18 PM - 2:18 PM
11	OR 224 & OR 212 / SE 114th AVE	1	3	1 Hr	8/25/2010	Wednesday	2:49 PM - 3:49 PM
12	OR 224 & OR 212 / SE 122nd AVE	1	3	1 Hr	8/25/2010	Wednesday	4:00 PM - 5:00 PM
13	OR 22 W / PERRYDALE RD	2	3	1 Hr	8/26/2010	Thursday	8:56 AM - 9:56 AM
14	OR 22 W / DOAKS FERRY RD	2	3	1 Hr	8/26/2010	Thursday	10:14 AM - 11:14 AM
15	OR 221 / DOAKS FERRY RD NW	2	3	1 Hr	8/26/2010	Thursday	12:11 PM - 1:26 PM
16	OR 221 / SE PALMER CREEK RD	2	3	1 Hr	8/26/2010	Thursday	1:40 PM - 2:40 PM
17	OR 219/ SE FARMINGTON RD	1	3	2 Hr	8/31/2010	Tuesday	7:45 AM - 9:45 AM
18	OR 219/ SE SCHOLLS FERRY RD (OR 210)	1	3	1 Hr	8/31/2010	Tuesday	10:21 AM - 11:21AM
19	OR 219/ BELL RD / N VALLE RD	2	4	1 Hr	8/31/2010	Tuesday	12:46 PM - 1:46 PM
20	OR 219/ SW UNGER RD	1	3	1 Hr	8/31/2010	Tuesday	2:44 PM - 3:44 PM
21	OR 219/ Tongue Ln	1	3	2 Hr	8/31/2010	Tuesday	4:00 PM - 6:00 PM
22	OR 211 / S Kropff Rd	2	4	1 Hr	9/1/2010	Wednesday	7:10 AM - 8:10 AM
23	OR 211 / S Meridian Rd	2	4	1 Hr	9/1/2010	Wednesday	8:27 AM - 9:27 AM
24	OR 214 / Howell Prairie Rd	2	3	1 Hr	9/1/2010	Wednesday	10:28 AM - 11:28 AM
25	OR 213 / S CARUS RD	1	4	1 Hr	9/1/2010	Wednesday	4:40PM - 5:40 PM

Re-validating and Improving Queue Length Models at Two-Way Stop Controlled Intersections

Transportation Planning and Analysis Unit
Transportation Development Division
Oregon Department of Transportation

December 2013

Table of Contents

Table of Figures

Table of Tables

Executive Summary

An effort to document the inconsistencies among the queue length estimation methods at two-way stop controlled intersections took place in the year 2010. The 2010 study models significantly improved the queue estimates except for major left turn configuration. The models are predicting over 65 percent of variability and most part used data from ODOT region 1 and region 2. One of the study recommendations is to expand the database and improve the model estimates.

Hence, the study again revisited with the aim to improve queue length models at two-way stop controlled intersections. Data covering various functional classifications of highways, geometric configurations, and geographic regions were collected by using video tapes. Still, previous models are performing well for 2013 data over other methods and models. The present study develops new models using 2013 data to foster the understanding of queue behavior on both major and minor approach lane groups. Various regression models were fitted to explain the random process in queue lengths. A model comparison shows significantly improved performance of the new models in predicting maximum queue lengths except for few lane groups where the 2010 model is predicting better queue lengths.

After introducing the problem, data collection and analysis efforts are presented. Next few sections explain the model selection and validation for each lane group configuration following the methodology outline. Last section concludes with recommendation of model forms for QL estimation.

1 Introduction

An effort to develop alternative models for estimating two-way stop controlled (TWSC) intersection queue lengths in the state of Oregon took place in the year 2010. Significant improvements in terms of predicting queue lengths over other analytical procedures were achieved. However, the 2010 report recommended continuation of the study. In specific, sample coverage and size were envisioned to improve the model performance. Also, model predictability, which hovers between 60 to 70 percent, has the potential to be improved by increasing the variability of the explanatory variables. In this continuation study, queue length models are re-estimated and validated with a broader range of traffic, queue lengths, geometry and other conditions.

2 Data Collection and Analysis

Data collection sites by region, shown in Appendix A, were screened using Traffic Count Management (TCM) software and aerial imagery. Queue Length (QL) was defined as the maximum number of vehicles in the queue during the study period (one hour) as collected from traffic count videos1. Hourly traffic volumes from TCM and geometry from aerial images were collected for the screened sites. Unlike the 2010 dataset, the number of heavy vehicles and their percentage in traffic, and conflicting lanes for the subject lane groups, were also collected as an expansion to the set of explanatory variables (listed in Table 1).

[1] Intersection videos, stored in Blu-ray disk format, show queues which are manually recorded for the study time period

Table 1. List of Explanatory Variables Considered for 2013 Data Analysis

Category	List of Explanatory Variables
Geometry	Number of approaches (Number of legs) Lane groups Number of lanes on both major and minor approaches Conflicting lanes for the subject lane group Lane configuration (shared/separate) Channelization / Flared approaches Median Type Intersection Skewness Presence of Two Way Left Turn (TWLT) / exclusive left lanes
Operations	Approach speed Upstream Signal within 1/4 mile
Traffic Flow	Approach volume Conflicting volume Turning volume Number and Percent of heavy vehicles by turning movement

Intersections were chosen to cover a range of lane configurations, geographic regions, functional classifications, and traffic conditions. In total, the study collected data from 38 intersections. Unlike the 2010 study, this study focused more on collecting data from regions other than Region 1. As illustrated in Figure 1, slightly more than half of the intersections are from Region 3. All of the data collected were from counts taken over the past three years.

The maximum number of vehicles in the stopped queue was collected for every one hour interval covering both peak and off-peak periods for each lane group. Traffic volumes for the same period were obtained from TCM. The next step was to calculate the conflicting traffic flow rate according to the procedure documented in the 2010 Highway Capacity Manual (HCM). Conflicting volume from individual movements in a lane group are added algebraically to obtain the lane group conflicting movements. Similarly, lane group volumes are obtained by adding individual lane volumes in that lane group. In addition, the study estimated queue lengths by the two-minute rule, HCM methodology, and Gard's equation for comparative purpose.

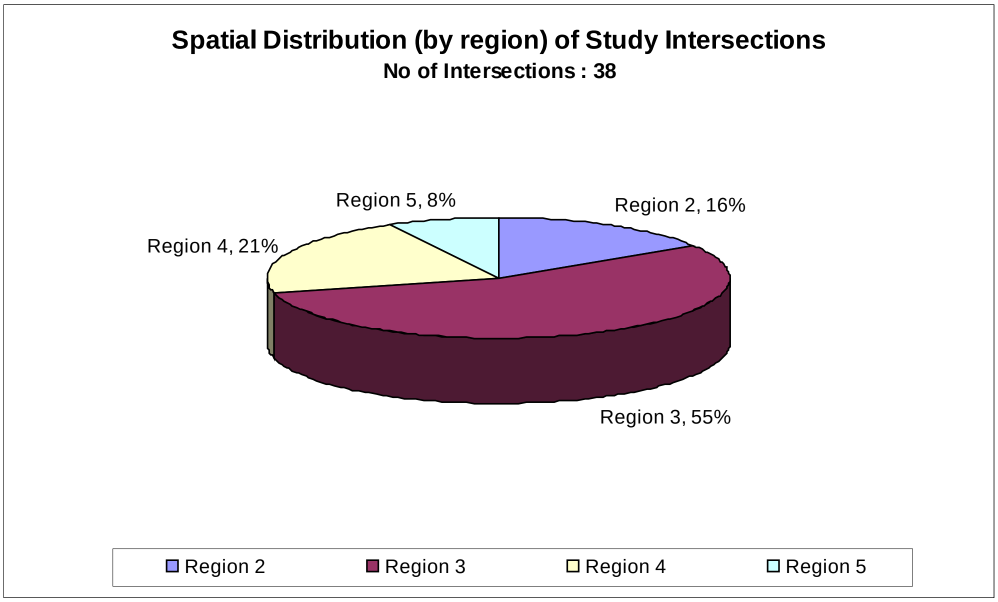


Figure 1. Distribution of Study Intersections by ODOT Region

3 Methodology

The 2010 Highway Capacity Manual (2010 HCM) ranks/ prioritizes the movements at TWSC intersections for capacity, delay and queue length estimation. Like the 2010 model, this study estimated queue lengths by lane groups. Apart from MJL (major left), MNLTR (minor shared left, through and right), and MNLR (minor shared left and right), the study also considered MNL (minor left), MNR (minor right), and MNTR (minor shared through and right) lane configurations. Statistical modeling using the QL as the dependent variable and a combination of explanatory variables (listed in Table 1) as independent variables was performed for each lane group. A summary of models for the given dataset was tabulated for each lane group. The best performing models were selected based on the significance of the model and its parameters. Once the models were selected, they were validated using a part of the 2013 data. The difference in queue lengths between the models and observed were calculated. This study used a difference of ± 1 vehicle as the acceptable range, a difference greater than + 1 vehicle labeled as over-estimated, and less than – 1 vehicle treated as under estimated. Data on percent of acceptable, under estimated and over-estimated queue lengths was used for models comparison. The next few sections outline the model estimates and comparisons.

4 Major Left (MJL) Lane Group Analysis

First, the analysis dealt with an assessment of the 2010 MJL model for the data collected in the year 2013. The assessment included a comparison of the 2010 MJL model estimates with the observed queue lengths, and a cross comparison with the estimates from the two-minute rule, HCM methodology, and Gard's equation. The difference between the model(s) and observed QLs was used to compare the models. Approximately 77 percent of the 2010 model estimates were within the acceptable range. The 2010 model estimates are comparable to the HCM and two-minute rule. However, the spread of over and under estimation is even for the 2010 model compared to over estimation from the two-minute rule and under estimation from the HCM method (Figure 2).

Next, a separate model was developed for the MJL lane configuration using 2013 data, to improve over the 2010 model estimates. A scatter diagram (Figure 3) between the major left volume and the queue length shows a wide spread of QLs and no definite trend observed visually.

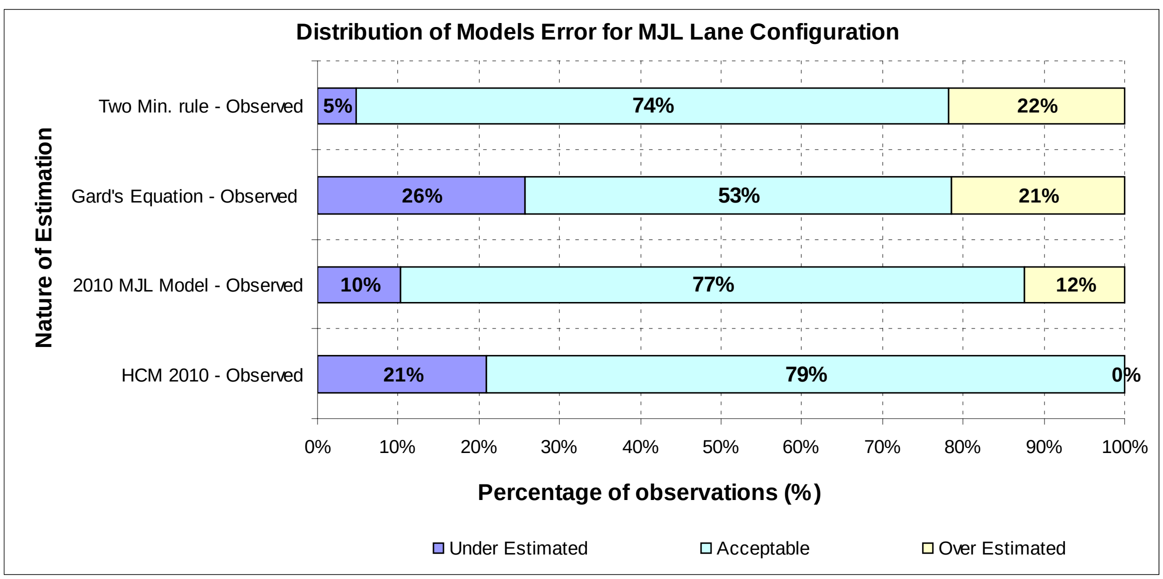


Figure 2. Comparison of 2010 MJL Model with Observed Queue Lengths

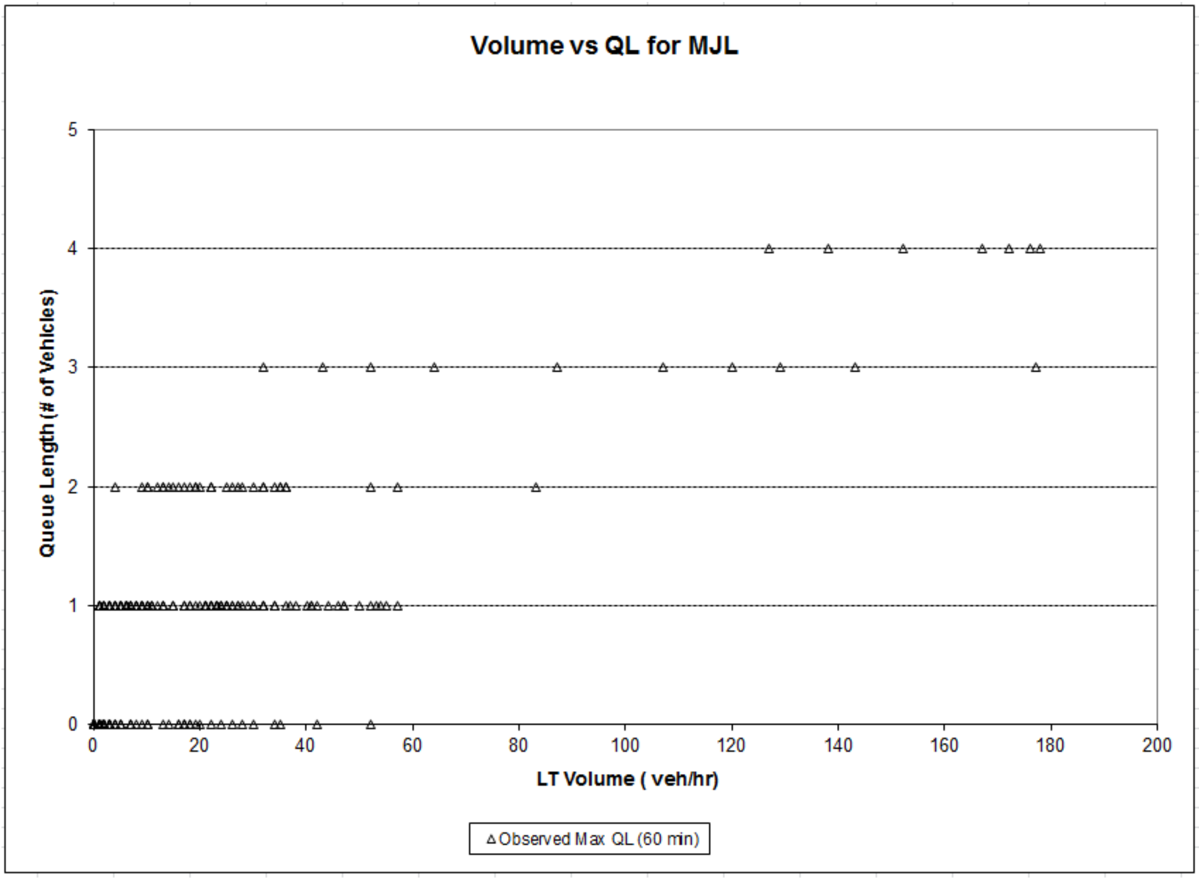


Figure 3. Observed Queue Lengths versus Observed Major Left Turn Volume

The observed queue length varied from a minimum of 1 vehicle to a maximum of 4 vehicles. Detailed statistical analysis is beyond the scope of this report. However, users may refer to the 2010 model development report for detailed model development analysis. After many iterations with the collected explanatory variables, only left turn volume (VOL), left turn conflicting volume (CONVOL), and combination of volumes explained the variability in QLs. However, the model only shows an R2 value of 0.55, which is less than the acceptable range of 0.60 to 0.80, generally considered for a good model. The lower value of R2 is due to less variability of QL with variation in left turn volume and conflicting volume. Next, the study made an attempt to see whether combining the 2010 and 2013 datasets brings more variability to the models. In fact, combination improves the model variability (R2 value) to 0.62. Table 2 shows the three models developed using data from both years.

Table 2. MJL Model Forms

Year Data Collected	MJL Model Form	R^2
2010	$QL = e^{(0.3925+0.0059*VOL+0.00104*CONVOL+0.49*Signal-0.81*LT)}$	67.0 (Percent of Deviation)
2013	QL = 0.69 + 0.007*VOL + 0.000071*CONVOL	0.55
2013 & 2010	QL = 0.70 + 0.004*VOL + 0.000078*CONVOL	0.62

Data validation used a part of 2013 data having variation in traffic and geometrical conditions. Comparison between the estimated and observed queue lengths showed that 80 percent of the 2010 model estimates are within the acceptable range. The model that uses the combined 2010 and 2013 data produces nearly 90 percent acceptable queue lengths. Figure 4 also shows the performance of other models. Both the two-minute rule and HCM 2010 methodologies are below 70 percent acceptable. In addition, the two-minute rule and Gard's equation over estimate QLs, whereas HCM 2010 is underestimating. The developed models other than the 2010 model over predict QLs but to a lesser extent than the two-minute rule and Gard's equation.

Though the 2013 model estimates are improved over the 2010 model, based on R2 value and nature of estimation (distribution of acceptable, under and over estimation) the 2010 model may best explain the MJL lane group QL.

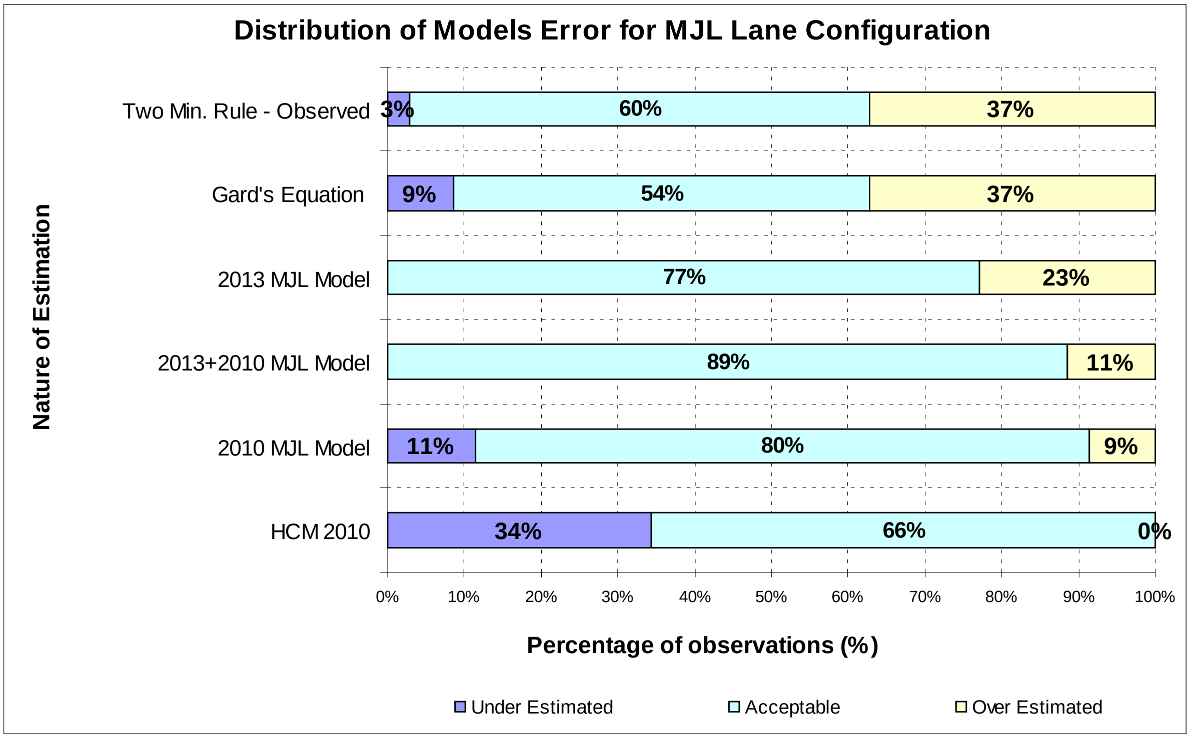


Figure 4. MJL Models Comparison

5 MNLTR Lane Group Analysis

When comparing the 2010 MNLTR model estimates with the observed QLs, around 78 percent of data (sample size of 357) is predicting acceptable QLs, which is 3 percent more than the two-minute rule estimates. However, the 2010 model underestimates more than it overestimates, as compared to the two-minute rule (Figure 5).

The best model form using 2013 data and combined years data does not yield good model fitness as the R2 value is less than 0.60 (Table 3). However, the validation data (sample size of 37) shows both the 2013 and combined year models predict nearly 90 percent acceptable QLs with even distribution of under and over estimation. The two-minute rule performs a little better than the 2010 model, but overestimates the QLs more than the 2010 model. The 2010 model evenly distributes the error in QL estimation (Figure 6).

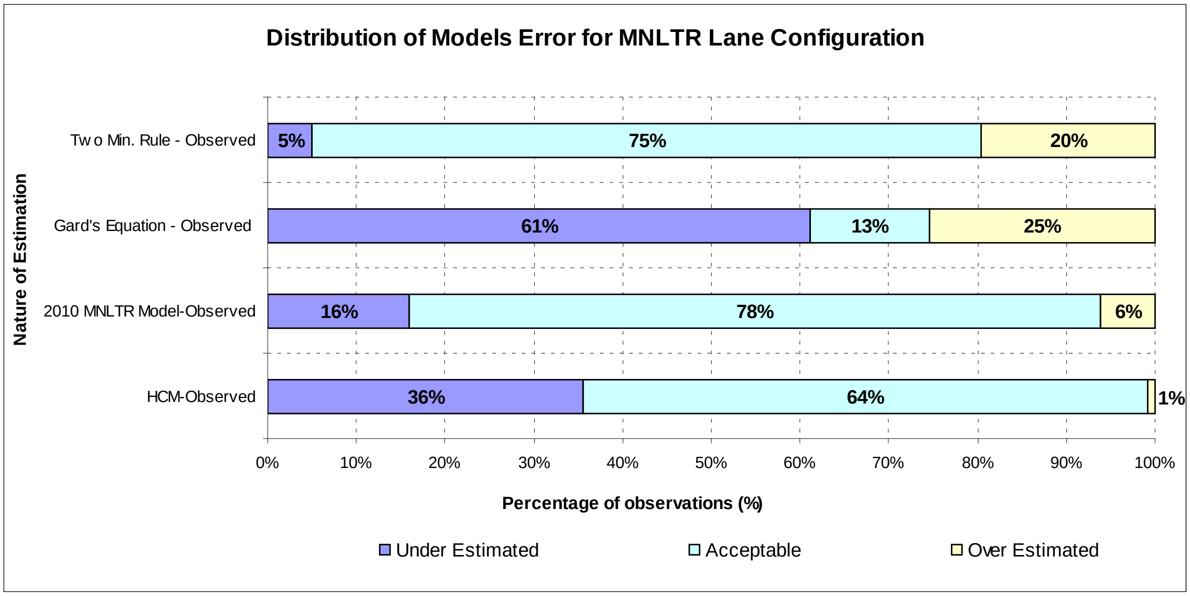


Figure 5. Comparison of 2010 MNLTR Model with Observed Queue Lengths

Though the 2013 model validation shows good results, based on the nature of estimation and model strength, the 2010 model seems the best for predicting minor shared left, through, and right lane configuration.

Table 3. MNLTR Model Forms

Year	MNLTR Model Form	R^2
2010	$QL = e^{(-0.7844+0.01636*VOL+0.0006*CONVOL-0.0000043*VOL*CONVOL)}$	71.6 (Percent of Deviation)
2013	QL = 0.88 + 0.0253*VOL - 1.2225*(VOL / CONVOL)	0.57
2013 & 2010	QL = 0.65 + 0.0246*VOL + 0.000383*CONVOL - 0.00000414* VOL* CONVOL	0.54

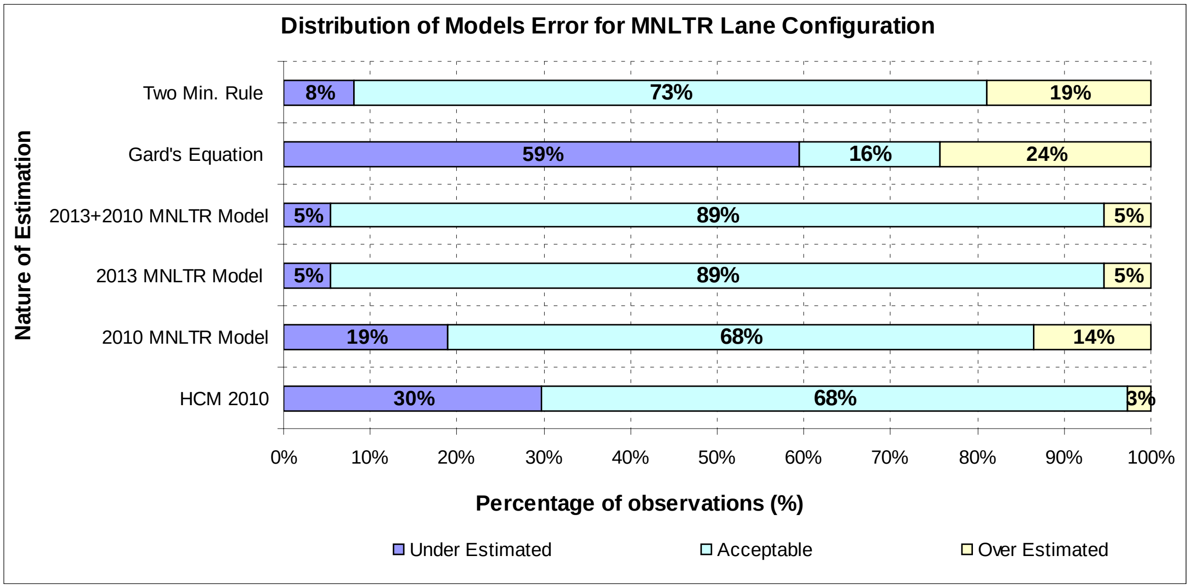


Figure 6. MNLTR Models Comparison

6 MNLR Lane Group Analysis

With an acceptable difference of one vehicle in queue length, 66 percent match with the 2010 model and 77 percent match with the two-minute rule. The HCM method matches 65 percent of the time. The results are shown in Figure 7. The 2010 model underestimates, but less when compared to the HCM methodology. However, the two-minute rule overestimates the QLs. The comparison shows some need to improve the 2010 model. The model form for 2013 and combined years data is shown in Table 4. The combined data model has a dummy variable “flared”, which is zero if the minor street does not have a flared approach. However, the combined data model has a lower R2 value, less than 0.60.

Table 4. MNLR Model Forms

Year Data Collected	MNLR Model Form	R^2
2010	QL= $e^{(-0.6319+0.0173*VOL+0.00066*CONVOL-0.000007913*VOL*CONVOL)}$	69.3 (Percent of Deviation)
2013	QL = 0.8641+ 0.0133*VOL - 0.00038*CONVOL+0.0000179*(VOL* CONVOL)	0.64
2013 & 2010	QL = 1.274 + 0.0189*VOL -0.1610*(VOL / CONVOL) -0.4006* Flared	0.52

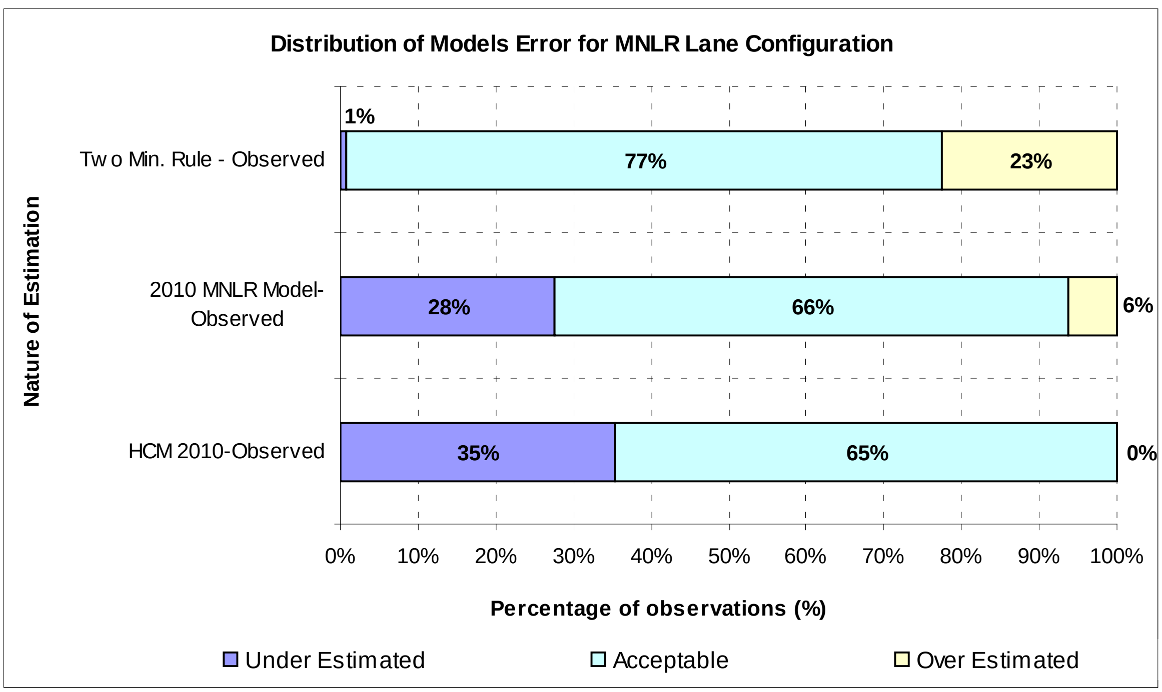


Figure 7. Comparison of 2010 MNLR Model with Observed Queue Lengths

When the models are validated using a sample size of 18, surprisingly the combined data model results in 90 percent acceptable QLs. Moreover, the 2010 and 2013 models have a 78 percent match, which is still 6 percent more than the two-minute rule. As with the other lane groups, the two-minute rule overestimates and the HCM method underestimates the QLs (Figure 8). Although the 2010 and 2013 models equally predict acceptable QLs, the 2013 model performs better in distributing the error difference; the

2010 model underestimates the QLs. Based on model strength, the 2010 model best describes the queue lengths for the three legged minor shared left and right turn lane group.

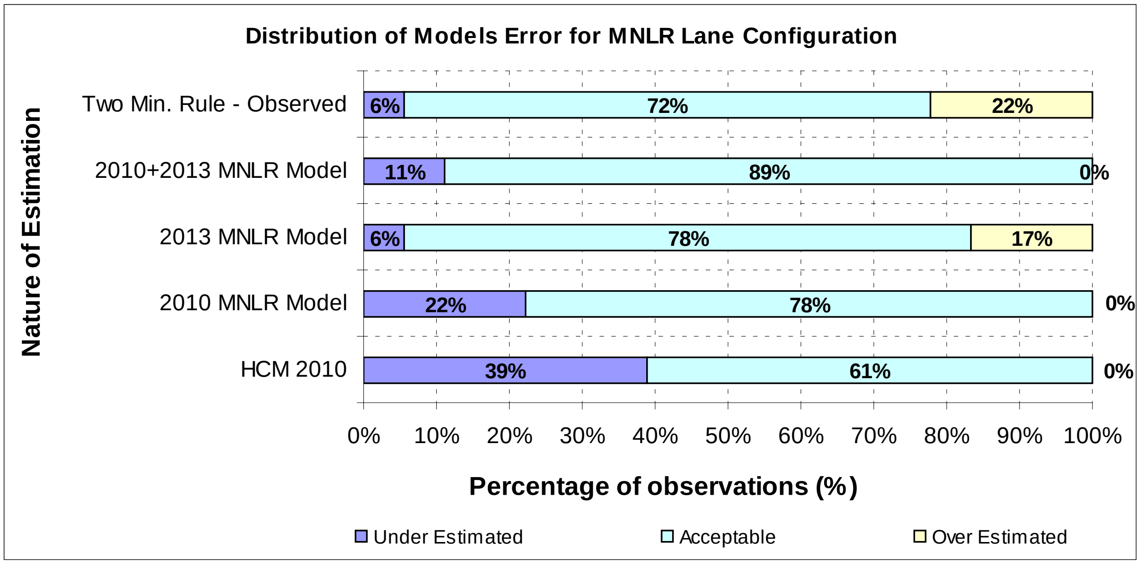


Figure 8. MNLR Models Comparison

7 MNL Lane Group Analysis

Both the two-minute rule and Gard's equation perform better than the 2010 MNL model. In addition, the 2010 model overestimates and the HCM methodology underestimates the QLs. Figure 9 clearly shows the 2010 model needs improvement. The model using 2013 and combined year data is listed in Table 5. Both 2013 and combined data models use volume and conflicting volume data to model QLs.

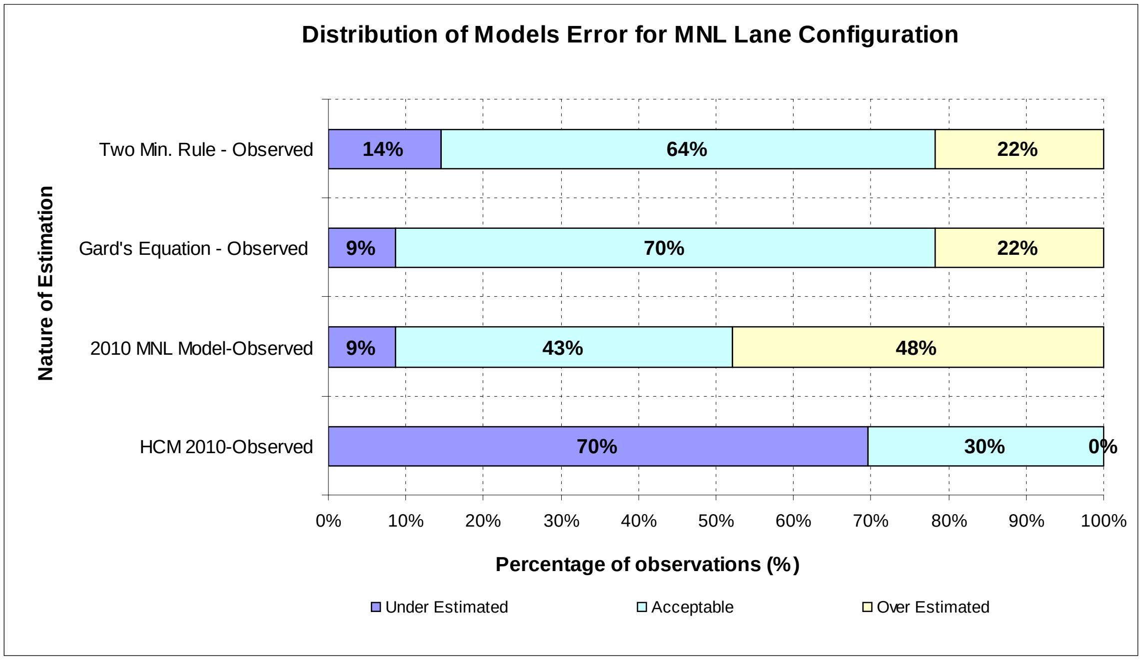


Figure 9. Comparison of 2010 MNL Model with Observed Queue Lengths

Figure 10 shows that the HCM methodology underestimates for most of the cases, and Gard's equation overestimates. Among the developed models, only the combined data model produces 53 percent acceptable QLs. Visually, the 2013 model produces slightly more acceptable queue lengths than the 2010 model. Error distribution is more even for the combined data model. The two-minute rule underestimates 61 percent of cases. Based on the model strength and distribution of errors compared to other models, the 2013 model best fits the MNL lane group QL estimation.

Table 5. MNL Model Forms

Year Data Collected	MNL Model Form	R^2
2010	QL= $e^{(1.7934-0.025*(CONVOL/VOL))}$	69.4 (Percent of Deviation)
2013	QL = 0.95+ 0.014*VOL +0.00074*CONVOL+3.01*(VOL/ CONVOL)	0.82
2013 & 2010	QL = 1.452+ 0.0217*VOL + 0.00126*CONVOL - 0.0147*(CONVOL / VOL)	0.53

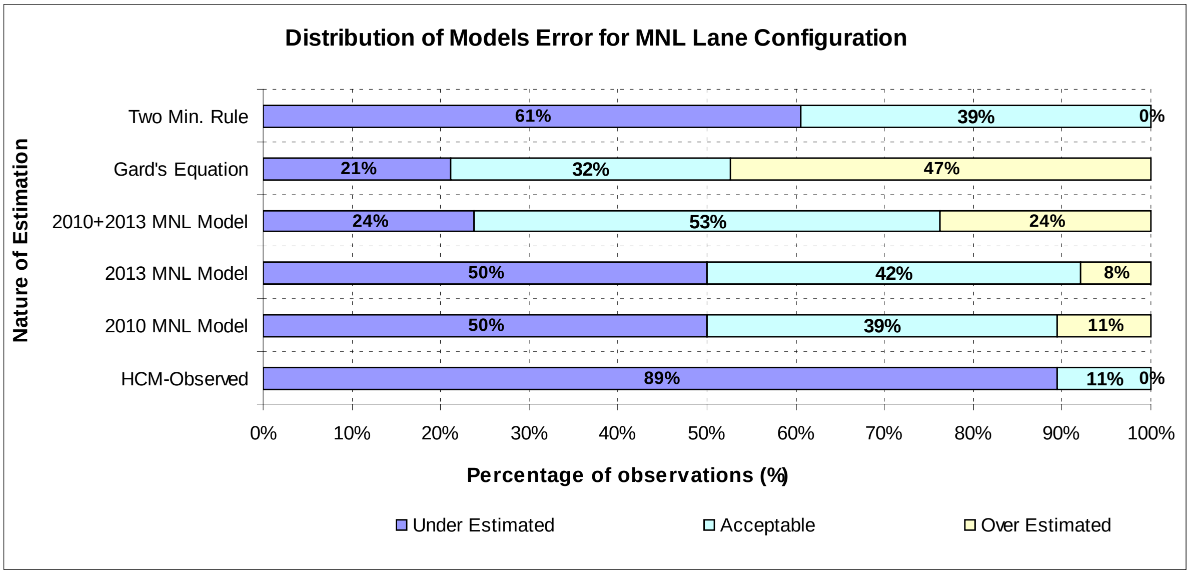


Figure 10. MNL Models Comparison

8 MNR Lane Group Analysis

The 2010 MNR model used only 18 data points and was not validated because of the very small sample size. Hence, the performance check for the 2010 model was not performed. Instead, a part of 2013 data (sample size of 44) was used to develop the model as shown in Table 6.

Table 6. MNR Model Forms

Year Data Collected	MNR Model Form	R^2
2010	$QL = e^{(0.2251+0.00005316*(VOL*CONVOL))}$	64.8 (Percent of Deviation)
2013	QL = 0.917+0.000047* VOL*CONVOL	0.37
2013 & 2010	QL = 0.865+ 0.0000534*VOL*CONVOL +0.2372*(VOL/CONVOL)	0.71

The differences between model and observed QLs are shown in Figure 11. Both the 2013 and combined year data models perform well compared to the 2010 model. The two-minute rule outperforms the 2010 model, but overestimates as compared to the 2013 model. Although the HCM methodology underestimates, it out-performs Gard's equation. Based on validation and model strength, the model based on combined year data may be used for MNR lane group queue length estimation.

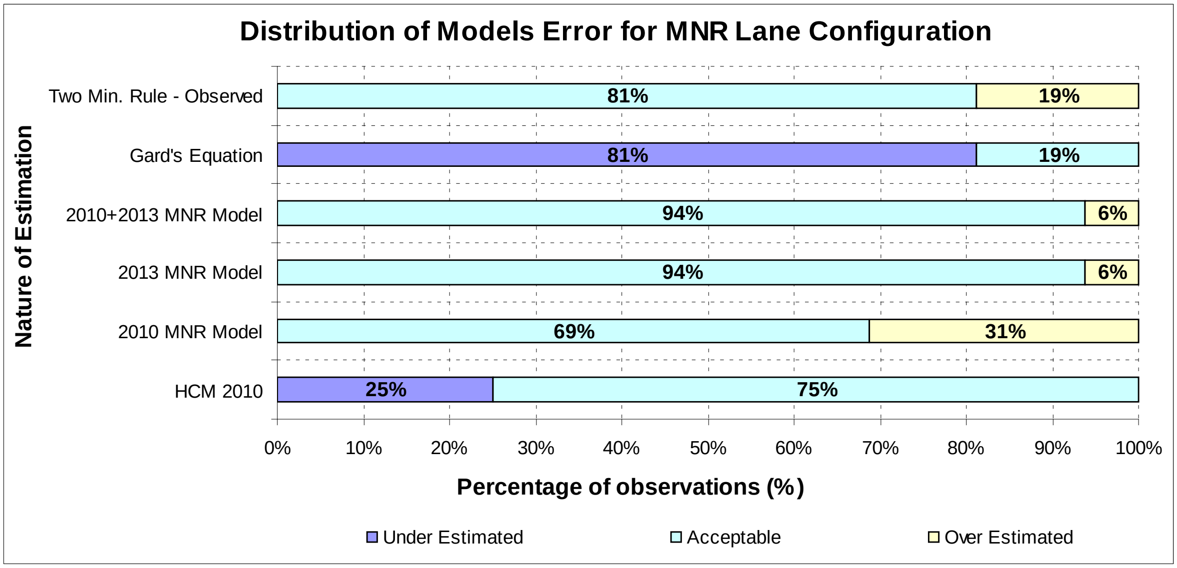


Figure 11. MNR Models Comparison

9 MNTR Lane Group Analysis

The minor shared through and right lane configuration is evaluated using a small sample size of 23. Only traffic volume on the lane group explains the variability in queue lengths. The best model, QL = 2.28 + 0.011 * VOL, has an R2 value of 0.61. The 2013 MNTR model was validated using 2010 data with a sample size of 13. The 2013 model produces 77 percent acceptable QLs compare to 69 percent from the two-minute rule (Figure 12). Both models are over predicting QLs, more so with the two-minute rule.

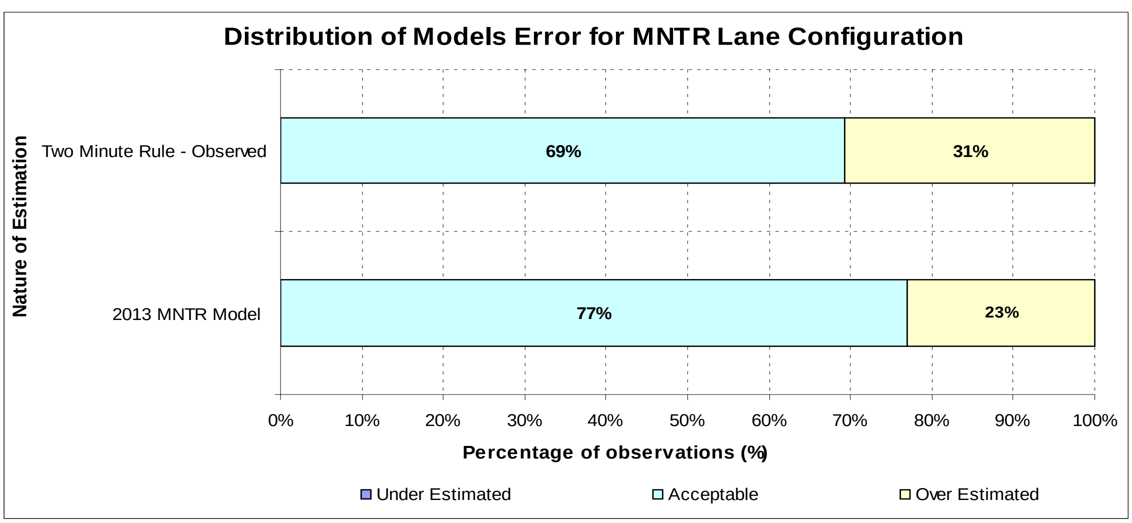


Figure 12. MNTR Models Comparison

10 Summary and Conclusions

The following table summarizes the developed models and applicable ranges for input data for each model:

Table 7. Summary of Queue Length Models

Lane Group	Model Equation & Ranges	R^2
MJL	$QL= e^{(0.3925+0.0059*VOL+0.00104*CONVOL+0.49*Signal-0.81*LT)}$ VOL = (0, 300] ; CONVOL = (0,2000] SIGNAL = 0 or 1 ; LT = 0 or 1	67 (Percent of Deviation)
MNLTR	$QL= e^{(-0.7844+0.01636*VOL+0.0006*CONVOL-0.0000043* VOL* CONVOL)}$ VOL = (0, 300] ; CONVOL = (0,3000]	72 (Percent of Deviation)
MNLR	$QL= e^{(-0.6319+0.0173*VOL+0.00066*CONVOL-0.000007913* VOL* CONVOL)}$ VOL = (0, 300] ; CONVOL = (0,3000]	69 (Percent of Deviation)
MNL	QL = 0.95+ 0.014*VOL +0.00074*CONVOL+3.01*(VOL/ CONVOL) VOL = (0, 300] ; CONVOL = (0,2000]	0.82
MNR	QL = 0.865+ 0.0000534*VOL*CONVOL +0.2372*(VOL/CONVOL) VOL = (0, 250] ; CONVOL = (0,1500]	0.71

VOL = Traffic volume on the subject approach in vehicles per hour;
CONVOL = Conflicting traffic volume as per the 2010 HCM methodology in vehicles per hour;
SIGNAL = Presence of upstream signal within ¼ mile of an intersection, applicable for major left turn only, 1 if there is a signal, otherwise 0;
LT = Presence of a separate left turn lane, applicable for major left turn only (1 if there is an exclusive left turn lane/median left turn lane/ two-way left turn lane, otherwise 0)

The developed models perform better than other models under different geography, traffic, and geometric characteristics. The 2013 data improves the predictability of the models. Although the study considered more explanatory variables, only volume and conflicting volume explain the variability in queue lengths. Based on the percentage variability in QL explained by the model, and the distribution of error differences between the predicted and observed QL, appropriate models were recommended for each lane group. Consistently, the two-minute rule overestimates and the HCM methodology underestimates queue lengths. Gard's equation estimates deviate from acceptable ranges for all lane group configurations. Moreover, the developed models distribute the error uniformly on both sides of the acceptable range. As always, data expansion has the potential to improve model predictability, especially for the minor shared through and left (MNTL), and minor shared through and right (MNTR) lane groups.

Appendix A - List of Study Intersections

TCM Site Number	No. of Legs	Region	County	Street Description	Mile Point	Location Description
19082	4	4	Crook	Main St/McKay Rd	2.14	Barnes Butte Rd @ Main St/McKay Rd
19084	4	4	Crook	S Fairview St	1.14	S Fairview St @ SE 5th St
19780	3	3	Jackson	PACIFIC HIGHWAY NO. 1 ROCK POINT FRONTAGE RD.	43.85	PACIFIC HIGHWAY NO. 1 ROCK POINT FRONTAGE RD. at Main Street (001CC, MP 44.23)
19791	3	3	Douglas	W Harvard Ave.		W Harvard Ave. @ W Maple St. vol only
19793	3	3	Douglas	NORTH UMPQUA HIGHWAY NO. 138	-0.75	OR138(W Harvard Ave.) @ W Corey Ct. vol only
19794	4	3	Douglas	W Harvard Ave.		W Harvard Ave. @ Harrison St. - vol only 6-9A & 3-6P
19842	4	3	Jackson	Talent Ave	1	Talent Ave (Rd 523, MP1.00) @ Creel Rd (Rd 8381, MP 0.23)
19844	3	3	Jackson	ROGUE VALLEY HIGHWAY NO. 63	10.86	Rogue Valley Hwy No. 63 (OR99) at N Rose Street (Rd 3816, MP 0.00)
38422	4	3	Douglas	COOS BAY-ROSEBURG HIGHWAY NO. 35	71.73	COOS BAY-ROSEBURG HIGHWAY NO. 35 @ Brockway Rd
38488	3	3	Curry	OREGON COAST HIGHWAY NO. 9	356.11	OREGON COAST HIGHWAY NO. 9 (US101) at Ransom Ave
4142011	3	2	Clatsop	LOWER COLUMBIA RIVER HIGHWAY NO. 92	94.6	US30 @ Tongue Point Rd.(old US30)
8032012	3	3	Curry	OREGON COAST HIGHWAY NO. 9	358.94	US101 @ Raymond Lane 4hr count 2-6P
8072012	3	3	Curry	OREGON COAST HIGHWAY NO. 9	358.45	US101 @ Court St. 4 hr count 2-6P Volume only
8092012	3	3	Curry	OREGON COAST HIGHWAY NO. 9	358.97	US101 @ Kings Way 4hr count 2-6P volume ony
9142011	4	4	Deschutes	MCKENZIE HIGHWAY NO. 15	110.15	OR126 (SW Highland Ave) @ 35th St
9172011	3	4	Deschutes	O'NEIL HIGHWAY NO. 370	2.11	OR370 @ NE 25th St.
9192011	3	4	Deschutes	O'NEIL HIGHWAY NO. 370	3.29	OR370 @ NE 41st St.
10032012	3	3	Douglas	COOS BAY-ROSEBURG HIGHWAY NO. 35	75.42	Coos Bay Roseburg Hwy(OR99) @ SW Landers Ave.
10112012	3	3	Douglas	NW Edenbower Blvd.		NW Edenbower Blvd. @ NW Broad St.
10282011	4	3	Douglas	UMPQUA HIGHWAY NO. 45	0.64	OR38 @ Winchester Ave. & River Front Way
15012012	3	3	Jackson	E Main St.		E Main St. @ Tolman Creek Rd.
15062012	3	3	Jackson	Wagner Creek Rd.		Wagner Creek Rd. @ Foss Rd. 3 hr cout 3-6p
15072012	3	3	Jackson	Wagner Creek Rd.		Wagner Creek Rd. @ W Wagner St. 3 hr count 3-6P

TCM Site Number	No. of Legs	Region	County	Street Description	Mile Point	Location Description
15082011	4	3	Jackson	Ashland St.		Ashland St. @ Normal Ave
15092011	3	3	Jackson	JACKSONVILLE HIGHWAY NO. 272	34.87	OR238(Hanley Rd.) @ W Main St.)
15112011	4	3	Jackson	GREEN SPRINGS HIGHWAY NO. 21	0.88	OR66(Ashland St. @ Clay St. & driveway(south leg)
15132012	4	3	Jackson	Front St.		Front St. @ Main st. 3 hr count 3-6P
16012012	4	4	Jefferson	MADRAS-PRINEVILLE HIGHWAY NO. 360	1.15	US26 @ SW Dover Lane
16022012	3	4	Jefferson	THE DALLES-CALIFORNIA HIGHWAY NO. 4	96.48	US97 @ SW Fairgrounds Rd.
20132011	3	2	Lane	N Delta Hwy		North Delta Rd. @ N Stapp Dr.
20172011	3	2	Lane	River Rd.	10.33	River Rd. @ Corliss lane
21062012	3	2	Lincoln	Toledo Frontage Rd (US20 Bus)	7.46	Toledo Frontage Rd (US20 Bus) @ East Slope Rd
21072012	3	2	Lincoln	Toledo Frontage Rd (US20 Bus0	8.21	Toledo Frontage Rd (US20 Bus) @ NE Sturdevant Rd
23012012	4	5	Malheur	OLDS FERRY-ONTARIO HIGHWAY NO. 455	25.75	Yturri Beltline @ NW Washington Ave. site 4801 - south leg
25012010	4	5	Morrow	COLUMBIA RIVER HIGHWAY NO. 2 PORT OF MORROW CONN. NO. 3	165.54	I-84 e/b ramps @ Laurel Lane
25022010	4	5	Morrow	COLUMBIA RIVER HIGHWAY NO. 2 PORT OF MORROW CONN. NO. 5	166.1	I-84 w/b on/off ramps @ Laurel Lane
27122011	3	2	Polk	N Main St.		N Main St. @ Ellis St.
7012011	3	4	Crook	OR370	4.99	OR370 @ Lone PIne Rd.

Queue Lengths at Single Lane Roundabouts in Oregon

Transportation Planning Analysis Unit (TPAU)
Transportation Development Division
Oregon Department of Transportation

November 2014

Table of Contents

Table of Exhibits

Table of Tables

Executive Summary

This study observed and analyzed single lane roundabout queue lengths. Cities and counties have actively utilized the benefits of roundabouts. Consequently, roundabouts have been and continue to be located all across the state. A manual data collection procedure was developed for recording queue lengths as video was taken for traffic counts. Equipment, contracts, and help were obtained. Miovision processed video recordings into counts.

Sites were scoped for consideration. Some examples were not considered for different reasons, such as only having two legs (not enough conflict to create queues), or due to resource limitations. A site might be dropped for not having a required element, yield control or splitter islands. Sites were then scoped for placement of equipment and personnel. Multi-lane roundabouts were dropped from study due to operational differences and lack of existing sites.

Roundabout data was collected, such as number of legs and splitter island widths.
Factors investigating for potential to influence roundabout operation and queues: number of legs, presence of a school, inscribed diameter, splitter island width, entry flow, and circular flow.

This study finds the Two-Minute Rule greatly overestimates queues at Oregon roundabouts. An empirically estimated equation was developed but found to be less accurate than the HCM 2010 methodology. The HCM 2010 roundabout queuing methodology is recommended to replace the Two-Minute Rule to estimate 95th percentile queue lengths for conditions that are applicable as per the HCM (isolated roundabouts, few pedestrians, undersaturated, etc.). For other situations alternative tools should be used, such as microsimulation.

1 Introduction

Considerable work has been invested in observing roundabout operation to study and explore predictive queue methodologies.

1.1 Methodologies

1.1.1 Two-Minute Rule

The Two-Minute Rule methodology estimates queue lengths for major street left turns and minor street movements during a two-minute stoppage of the turning movement. This method does not consider impacts of conflicting flows on a queue. Currently in the Analysis Procedures Manual (APM) the Two-Minute Rule is used to estimate queues at roundabouts except where simulation is appropriate. The two-minute rule calculation of the 95th percentile queue:

$$S = vtL$$

Where:

S = 95th percentile queue (feet)

v = average left-turn volume arriving in a 2-minute interval

t = storage ability; usually 1.75 to 2.0 (Table 1-1)

L = average stored vehicle length based on truck percentage (Table 1-2)

Table 1-1 Selection of "t" Values (source: APM)

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

The L variable starts with a value of 25-feet in the equation until the truck percentage of the turning volume equals five percent, as per Table 1-2.

Table 1-2 "L" Storage Length Adjustments for Trucks (source: APM)

Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

For dual left turn lanes, the results can be divided by 1.8. This follows an assumption that queued vehicles will not evenly distribute between turn lanes.

1.1.2 Highway Capacity Manual

The HCM 2010 procedures are founded on National Cooperative Highway Research Program 3-65(1) recommendations based on a database (31 sites) of U.S. roundabout operation.

The HCM 2010 states that roundabouts share the same basic control delay formation as two-way and all-way stop controlled intersections. There is an adjustment for the effect of yield control, rather than stop control. In the absence of research on traveler perception of quality of service at roundabouts, HCM 2010 roundabout service measures and thresholds follow those of unsignalized intersections.

The general procedure for automobile analysis of roundabouts is summarized in HCM 2010 Exhibit 21-9. There are 12 steps of analysis (please see HCM 2010 for full procedure):

Step 1: Flow rates from demand volumes

Step 2: Passenger car equivalents (bicycles and trucks)

Step 3: Circulating and exiting flow rates, addition of movements

Step 4: Entry flow rates by lane

Step 5: Capacity of entry lanes

Step 6: Pedestrian impedance to vehicles

Step 7: Vehicles /hour /lane from capacities and factors

Step 8: Volume/capacity ratio for each lane

Step 9: Average control delay

Step 10: LOS for each lane on each approach

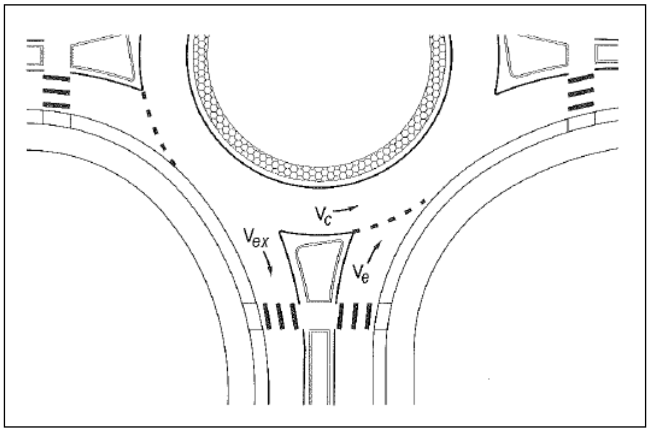
Step 11: Average Control Delay and LOS for entire roundabout

Step 12: 95th percentile queues

For a single lane roundabout automobile analysis, the following steps are applicable (excludes steps 3B, 4, 5, and 6B).

2010 HCM Exhibit 21-2 (Exhibit 1) shows a single lane roundabout with an entry flow conflicting with a circulatory flow. Please note the subscripts: “c” is for circulatory, “e” is for entry, and “ex” is for exiting flow. Entry vehicles yield to circulatory vehicles.

Exhibit 1-1 Flow Rate Nomenclature



Step 1: Flow rates from demand volumes

Volumes should be gathered from an intersection count. Bicyclists using the crosswalks are counted as pedestrians. Bicyclists using the roundabout as vehicles are added to the intersection volumes for each movement (including U-turns). The count should also provide a PHF for each movement. HCM 2010 Equation 21-8 finds the demand flow rate for each movement.

$$v_i = \frac{V_i}{PHF}$$

Where:

v_i = demand flow rate for movement (veh/h)
V_i = demand volume per movement, bicycle = vehicle (veh/h)
PHF = peak hour factor

Step 2: Passenger car equivalents (bicycle and trucks)

Flow rates in vehicles per hour (vph) are converted to equivalent passenger cars per hour (pce/h) using vehicle factors.

Exhibit 1-2 HCM 2010 Exhibit 21-10

Vehicle Type	Passenger Car Equivalent, E_T
Passenger car	1.0
Heavy vehicle	2.0

Demand volumes (vph) are converted to passenger car equivalents (pce/h), using a heavy vehicle factor equation. E_T and E_B are the equivalent factors for trucks and bicycles. The proportion that these vehicle types occur in a count is designated as P_T and P_B.

A possible variation of the heavy vehicle adjustment factor equation:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1)}$$

Where:

f_{HV} = heavy vehicle adjustment factor
P_T = proportion of demand volume that consists of heavy vehicles (decimal)
E_T = passenger car equivalent for heavy vehicles (table)
P_B = proportion of demand volume that consists of bicycles (decimal)
E_B = passenger car equivalent bicycles (0.5, page 21-21)

This f_{HV} is then used in HCM 2010, equation 21-9.

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Where:

$v_{i,pce}$ = demand flow rate for movement (veh/h)
v_i = demand flow rate for movement (veh/h)
f_{HV} = heavy vehicle adjustment factor

Step 3: Circulating and exiting flow rates; addition of movements

The circulating flow rates in front of each entry are summed in terms of passenger car equivalents. See HCM 2010 equation 21-11 below.

$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Where:

$v_c =$ Circulating flow rates in front of specified entry; in passenger car equivalents

$v_{WBU,pce} =$ Flow rates of a specified movement

Step 3B: If considering a bypass lane, calculate the conflicting flow rates

The conflicting flow rates for where the bypass lane merges into the exiting lane can be calculated with HCM 2010 Equation 21-12, similar to Equation 21-11.

Step 4: Entry flow rates by lane, if more than one lane

This step is for a multi-lane roundabout/more than one entry lane. For more than one entry lane, it is important to identify current lane utilization and nearby attractions. Future developments should be considered as well. This may be a good opportunity to apply a travel demand model. If this is not available, see the HCM 2010 exhibits in chapter 21.

Step 5: Capacity of entry lanes; uses value from Step 3

HCM 2010 Equation 21-1 finds capacity for movements using circulatory flow rate.

$$c_{e,pce} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce}}$$

Where:

C = lane capacity (passenger cars per hour; pc/h)
V_c = Conflicting flow (pc/h)

If considering more than one entry lane, see the HCM 2010 Exhibits in Chapter 21.

Step 6: Pedestrian impedance to vehicles

This is the pedestrian impedance for single lane roundabouts; for two entry lanes, consult the HCM 2010, Exhibit 21-19. For one entry lane, use HCM 2010 Exhibit 21-17, to find the entry capacity adjustment factor for pedestrians.

$$IF \qquad v_{c,pce} > 881 \qquad f_{ped} = 1$$

$$Else \quad IF \qquad n_{ped} \leq 101 \qquad f_{ped} = 1 - 0.000137n_{ped}$$

$$Else \qquad f_{ped} = \frac{1119.5 - 0.715v_{c,ped} + 0.00073v_{c,pce}n_{ped}}{1068.6 - 0.654v_{c,pce}}$$

Where:

f_{ped} = entry capacity pedestrian adjustment factor
v_c = conflicting flow (pc/h)
n_{ped} = conflicting pedestrians (p/h)

Fewer than 40 pedestrian crossings of a leg in one hour do not have a significant effect on roundabout operation. If following the HCM 2010 procedure, an adjustment factor for pedestrians of 1.0 is recommended if there are fewer than 40 pedestrian crossings of a leg. Following the HCM, if the number of passenger car equivalent vehicles circulating in front of an entrance is over 881, then the adjustment factor for pedestrians is a factor of 1.0. If that is not the case and the number of pedestrians crossing at a crosswalk is less than or equal to 101, then the second equation determines the adjustment factor for pedestrians. Otherwise, see the final equation represented from HCM 2010 Exhibit 21-17.

Step 6B: If considering more than one entry lane, see HCM 2010 Exhibit 21-19.

Step 7: Vehicles /hour /lane from capacities and factors

A weighted vehicle adjustment factor is created with HCM 2010, Equation 21-15.

$$f_{HVe} = \frac{f_{HV,U} v_{U,PCE} + f_{HV,L} v_{L,PCE} + f_{HV,T} v_{T,PCE} + f_{HV,R,e} v_{R,e,PCE}}{v_{U,PCE} + v_{L,PCE} + v_{T,PCE} + v_{R,e,PCE}}$$

Where:

f_{HVe} = averaged heavy vehicle adjustment factor for entry lane
f_{HVi} = heavy vehicle adjustment factor for movement i
$v_{i,PCE}$ = demand flow for movement i (pc/h)

The flow rate is converted back to vehicles per hour with HCM 2010, Equation 21-13, which is a rearrangement of Equation 21-9.

$$v_i = v_{i,PCE} f_{HV,e}$$

Where:

$v_{i,pce}$ = demand flow rate for movement (veh/h)
v_i = demand flow rate for movement (veh/h)
f_{HVe} = heavy vehicle adjustment factor

Step 7.5: The capacity of a lane is converted back to vehicles per hour in Equation 21-14.

$$c_i = c_{i,PCE} f_{HVe} f_{ped}$$

Where:

$c_{i,pce}$ = demand flow rate for movement (Epc/hr)
c_i = demand flow rate for movement (veh/hr)
f_{HVe} = heavy vehicle adjustment factor
f_{ped} = entry capacity pedestrian adjustment factor

Step 8: Volume/capacity ratio for each lane

The volume/capacity ratio of a lane is calculated in Equation 21-16.

$$x_i = \frac{v_i}{c_i}$$

Where:

x_i = volume-to-capacity ratio of the subject lane i (only looking at one lane here)
v_i = demand flow rate of the subject lane i (veh/h)
c_i = capacity of the subject lane i (veh/h)

Step 9: Average control delay, similar to unsignalized intersections

Signal timers aiding in the study stated that a signal would likely not get such small queues or delays as the roundabouts studied. The HCM 2010 states the delay to be similar to unsignalized intersections, per United Sates roundabout data. The 2010 HCM makes a good point about delay at the peak hour or design hour:

> “At higher volume-to-capacity ratios, the likelihood of coming to a complete stop increases, thus causing behavior to resemble STOP control more closely.”

At higher volumes, it is likely that motorists may stop before the crosswalk as well as the yield/stop line. The 2010 HCM describes this as resembling STOP control.

The average control delay of a lane is calculated in 2010 HCM Equation 21-17. The adding of the third term, the lesser of the v/c or 1.0, is new for the 2010 HCM.

$$d = \frac{3600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3600}{c}\right)x}{450T}}\right] + 5X\ \min[x,1]$$

Where:

d = average control delay (s/veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/h)
T = time period (h) (T = 0.25 for a 15-min analysis)

Step 10: LOS for each lane on each approach

The delay from Step 9 determines LOS of each lane via 2010 HCM Exhibit 21-1.

Exhibit 1-3 HCM 2010 Exhibit 21-1

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a] v/c ≤ 1.0	v/c >1.0
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: [a] For approaches and intersectionwide assessment, LOS is defined solely by control delay.

Step 11: Average Control Delay and LOS for entire roundabout

The average control delay of a roundabout is calculated in 2010 HCM Equations 21-18 and 21-19. As this process is only considering single lane roundabouts, these equations will boil down to an average of approach (2010 HCM Equation 21-19):

$$d_{intersection} = \frac{\sum d_i v_i}{\sum v_i}$$

Where:

$d_{intersection}$ = average control delay for entire intersection (s/veh)
d_i = control delay for approach i (s/veh)
v_i = flow rate for approach I (veh/h)

With the intersection control delay, look up the LOS via the 2010 HCM Exhibit 21-1 (as shown in Step 10).

Step 12: 95th percentile queues for each lane

The 95th percentile queue for a given approach lane is calculated using Equation 21-20.

$$Q_{95} = 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3600}{c}\right)x}{150T}}\right]\left(\frac{c}{3600}\right)$$

Where:

Q_{95} = 95th percentile queue (veh)
x = volume-to-capacity ratio of the subject lane
c = capacity of the subject lane (veh/h)
T = time period (h) (T = 0.25hr for a 15-min analysis)

1.2 Challenge

The current APM methodology of using the Two-Minute rule has been observed to overestimate queue lengths at roundabouts. The challenge was to observe and collect data in regard to roundabout observations. The goal was to find a better way to estimate single lane roundabout queue lengths for planning level analysis. Geometric dimensional factors were included in the study to assess the importance and impact of physical design elements. Inscribed diameter and splitter island width appeared to be factors based on visual observations.

1.3 Purpose

This study documents roundabout observations, study locations, data collection, development of an empirical roundabout maximum queue predictive equation, and compares other queue predictive methods.

1.4 Data Collection and Use

Data collection required prior effort: identifying potential parameters influencing queue behavior, location, selection, and data to record. After collection, data was processed to get the calculation inputs. Methodologies were compared to the developed empirical equation. Equation validation was conducted to check and compare accuracy.

2 Data Collection & Analysis

Data was used in equation development, validation, and comparing methods for accuracy.

2.1 Potential Data

Table 2-1 shows the locations of the 69 single lane roundabouts in Oregon at the data collection time.

Table 2-1 Locations of Single Lane Roundabouts in Oregon

City	#
Albany	2
Beaverton	1
Bend	28
Central Point	1
Clackamas	3
The Dalles	1
Eugene	3
Hillsboro	2
Lake Oswego	2
Madras	2
Medford	1
Newberg	3
Oregon City	1
Portland	3
Redmond	1
Sherwood	3
Springfield	5
Sunriver	1
Tigard	1
unincorporated	2
Wilsonville	3

County	#
Clackamas	9
Deschutes	30
Jackson	2
Jefferson	2
Wasco	1
Lane	8
Linn	2
Multnomah	2
Washington	10
Yamhill	3

Region	#
Region 1	21
Region 2	13
Region 3	2
Region 4	33
Region 5	0

2.2 *Data Collection*

Studied roundabouts were chosen to cover a range of geographic regions, physical features, traffic volumes, and traffic conditions. Multi-lane roundabouts were dropped primarily due to the low number existing in Oregon. Some data did not prove to be useful, such as city's various roadway classifications.

There are 69 single lane roundabouts in Oregon at this time. During the data collection period 53 roundabouts were visited. Unfortunately some were not visited due to distance and funding limitations (Central Point, Medford, and The Dalles). The amount studied in Bend is very small in proportion to the number of roundabouts in Bend. Two-leg roundabouts did not have enough conflicting flow for this study. Conflicting flow and driver behavior affected roundabouts that were at a parking lot entrance or turn around. Some roundabouts were scouted, but could not be videoed due to obstacles (usually trees) in the center island. Some video recorded roundabouts were dropped from the study due to technical counting difficulties. Other videos came back with pictures that did not seem to match the roundabout. Of those, 23 were video recorded. With data cleaning and removal of outliers, 13 roundabouts were used for the equation set and 15 roundabouts were used in the validation set. Some roundabouts were represented in both data sets, but that no roundabout approaches were duplicated in the two data sets. These roundabouts are listed in Appendices B and C. Some roundabouts were visited, but not studied. Data collection procedures that were followed are listed in Appendix D.

Three, four, and five leg roundabouts were initially considered. Roundabouts with five operating legs were dropped from analysis. There weren't many five leg roundabouts and they operated differently. The equation was created for only three and four leg roundabouts. Three leg roundabouts were considered if the volumes and conflicts were observed or predicted to be abundant. Two leg roundabouts were not considered for the study due to a lack of conflict points. They slowed traffic with their physical presence, as roundabouts are designed to do, acting as a speed bump.

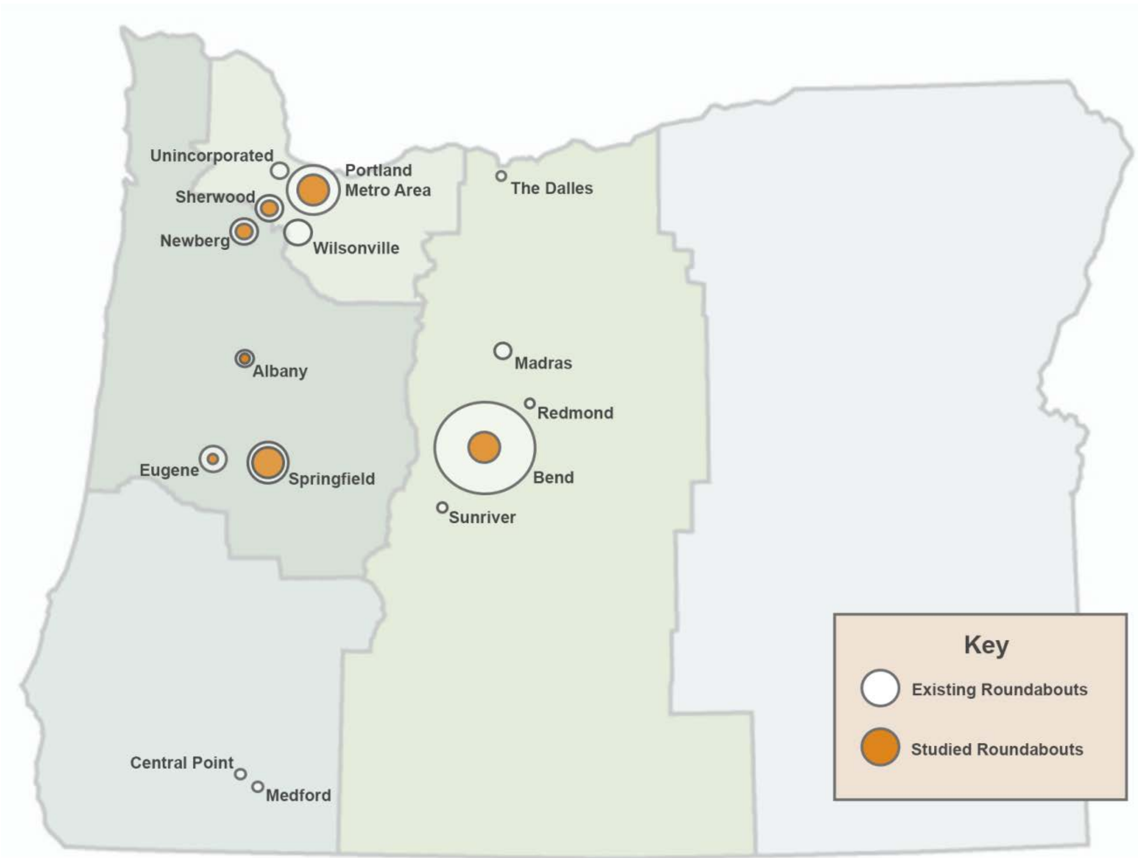
In total, 15 different roundabouts were used for data, shown in Appendices B and C. Out of 15 roundabouts:

- One roundabout was studied in: Albany, Hillsboro, Lake Oswego, Eugene, Sherwood/Newberg, Portland, and Tigard
- Two roundabouts were studied in: Happy Valley
- Three roundabouts were studied in: Bend, Springfield
- Three roundabouts were not near a school
- Three roundabouts had three legs

Two of the better performing roundabouts have inscribed diameters greater than 160 feet. Roundabouts with larger inscribed diameters and splitter island widths appeared to have improved performance among those observed.

Exhibit 2-4 shows the data collection in respect to the 69 existing single lane roundabouts in the state of Oregon.

Exhibit 2-1 Data Collection Map



The development and validation roundabouts are in Appendices B and C respectively. All data was collected in 2011. Where the data is available, both AM and PM peak periods were recorded.

Queue data was collected through observations during video recordings of traffic volumes. The maximum queue length and number of vehicles in the stopped queue was noted for every 15 minute interval.

Generally, an hour of traffic was recorded or selected. Of that hour, there were four 15 minute intervals (for four legs). An hourly traffic volume was created by multiplying 15 minute intervals by four.

It was recorded if the intersection was within a half mile of a school. School names, start times, and release times were sought out. Several cities have decided to build roundabouts near schools. This changed the peak hour of the roundabouts and the times they were studied. The proximity of the schools also infused a larger number of buses and pedestrians as this intersection may be a point all would have to pass to approach the school from one side of the city.

General items recorded were the date, city, intersection, and street classification. Geographic information was recorded including: number of legs, inscribed diameter, and splitter island width, see Exhibit 2-5. If it was a multi-lane roundabout, then that was also noted.

During data collection, it was observed that three leg roundabouts performed better than observed four leg roundabouts. The queue lengths were observed to be shorter. With fewer inputs into an intersection, operation had fewer conflicts.

In a cost savings measure, bicycle and pedestrian traffic was manually recorded. Video counts were developed with vehicle classifications of auto, medium truck and heavy truck.

The movements recorded were left, through, right, and U-turn. These movements were recorded for several modes: autos, medium trucks, heavy trucks, bicycles, and pedestrians. Pedestrians were recorded for which approach they crossed. If a bicycle used a pedestrian crossing, then the crossing was recorded as a pedestrian occurrence. If a bicycle moved into the approach lane and navigated through the roundabout, then they were counted as a bicycle, but also part of the vehicle group (much like a truck).

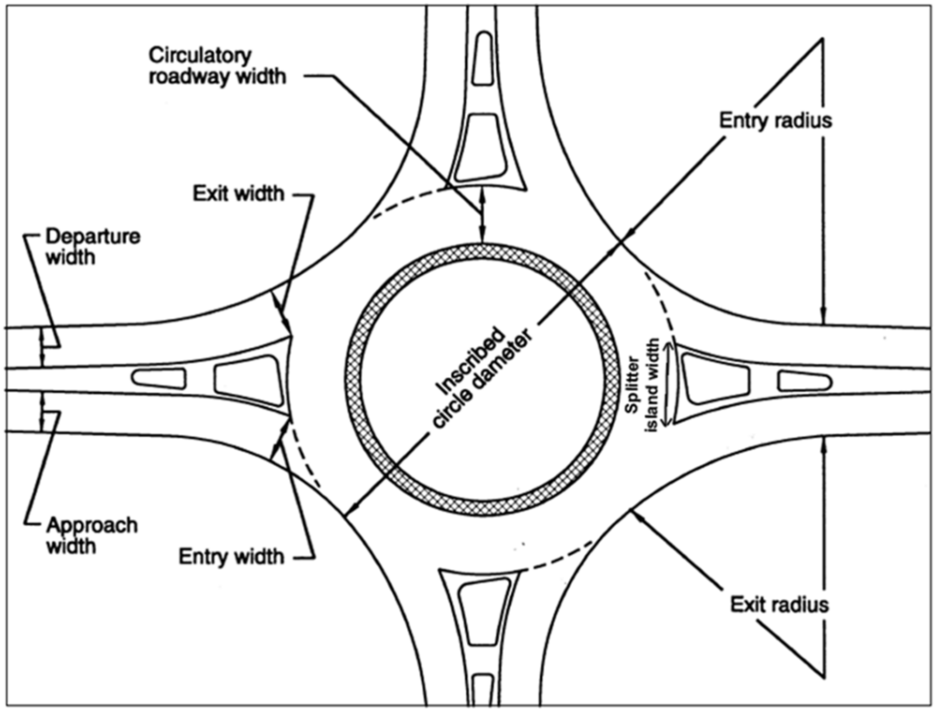
Roundabout information:

- number of legs
- if within ½ a mile of a school (“yes” or “no”)
- portion of an hour studied
- inscribed diameter

Inscribed diameter and splitter island dimensions are shown in Exhibit 2-5.

Exhibit 2-2 Roundabout Dimensions

Information per leg



- number of pedestrian crossings of each approach in the studied hour
- splitter island width adjacent to the circular roadway (larger is better)

The entry flow rate, circular flow or conflicting flow, and the exiting flow rates were calculated after observation and recordings. This information was computed with the aid of a spreadsheet.

2.3 Factors Observed to Influence Queue/Operation Behavior

While at the roundabout sites, it was observed that certain factors appeared to influence queue behavior and overall operation.

- Splitter island width
- Circulatory and entering volumes
- Within ½ a mile of a school

Circulating (interrupting) and entering (queue creating) volumes were the primary factor on operations.

Roundabouts with wide splitter islands appeared to improve operation and lower queue lengths. The wider a splitter island, the more time a waiting vehicle has to move from the approach leg into the circulatory roadway.

Roundabouts slowing traffic near a school (near a school zone) were observed to have an increase in school buses and young pedestrian crossings. The slower buses and crossing children appeared to increase queue lengths. School proximity may be an environmental type of variable similar to CBD, as an aggregate indicator of area characteristics such as parking, speeds, driver behavior, school zones, signs, markings, etc. Proximity of a school may indicate conditions where drivers tend to be more alert to the potential for school age children to be in the area and may tend to exercise a bit more caution than in non-school areas. School age children may not actually have to be present for the effect to exist.

3 Equation Data

3.1 Data Analysis

The data was split into two data sets, one for development of the predictive queue equation and another for validation (See Appendix A). Due to differences observed in the field and in calculations as well as lack of available sites, the multi-lane roundabouts were dropped and the study narrowed to single lane roundabouts.

3.2 Equation Data

Data for creating the equation and comparing methodologies included 243-15 minute sample sets. These 15 minute data sets were expanded to represent an hour. These data sets were from Region 1, Region 2, and Region 4. The data sets include the cities of Albany, Bend, Happy Valley, Hillsboro, Lake Oswego, Springfield, Portland, and Tigard.

Heavy vehicle percentages were calculated prior to equation development. The data influencing queues were identified and their significance considered.

3.3 *Volume to Flow Rates*

The intersection counts were converted into flow rates, making adjustments for bicycles, medium trucks, and heavy trucks for each movement.

The hour movement volumes of all vehicles, bicycles, medium trucks, and heavy trucks will be required. The highest 15 minute movement volume of the roundabout should also be recorded to determine the PHF for each movement.

Use the HCM 2010 Equation 21-8 to find the demand flow rate for each movement.

$$v_i = \frac{V_i}{PHF}$$

Where:

v_i = demand flow rate for movement (veh/h)
V_i = demand volume for movement, include bicycles as a vehicle (veh/h)
PHF = peak hour factor

Bicycles were part of the intersection volumes equaling a car for each movement (including U-turns). This does not involve the passenger car equivalent at this step. This is also the case for trucks; they are all counted as one vehicle entering the roundabout.

Roundabout data needed include the number of legs, if within ½ a mile of a school, decimal portion of an hour studied (1.0 for an hour), and the inscribed diameter.

The number of pedestrian crossings of each leg (including bicycles using pedestrian crossings) and splitter island width adjacent to the circular roadway (larger improves operation) were recorded. Inputs for the entire roundabout would include: number of legs (3 or 4), located within ½ of a mile of a school, portion of an hour studied (recommend 1.0), inscribed diameter, and passenger car equivalents.

Recommended Passenger Car Equivalents for bicycle, medium, and heavy truck are as shown in Table 3-1.

Table 3-1 Recommended Passenger Car Equivalents

Vehicle Type	Passenger Car Equivalents (E)
Passenger Car	1.0
Bicycle	1.0
Medium truck (two axles, UPS truck)	1.5
Heavy vehicle	2.0

Demand volumes (vph) were converted to passenger car equivalents (PCE/h), using a heavy vehicle factor equation similar to that found in the 2010 HCM. E_m and E_h are the equivalent factors for medium and heavy vehicles, 1.5 and 2 respectively. The proportion of these vehicle types occurring was calculated and designated as P_m and $P_{h.}$

Note that the recommended value of 1.0 PCE for bicycles (E_b) cancels out one term in the HCM 2010 equation below. The proportion of these vehicle occurrences was calculated. The heavy vehicle adjustment factors were calculated using the following equation.

$$f_{HV} = \frac{1}{1+P_m(E_m-1)+P_h(E_h-1)}$$

Where:

f_{HV} = heavy vehicle adjustment factor
P_m = proportion of demand volume that consists of medium trucks (decimal)
P_h = proportion of demand volume that consists of heavy vehicles (decimal)
E_m = passenger car equivalent for medium trucks (Passenger Car Equivalents given)
E_h = passenger car equivalent for heavy vehicles (PCE s given)

This f_{HV} is then used by the Single Lane Roundabout Calculator in the form of HCM 2010, Equation 21-9.

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Where:

$v_{i,pce}$ = demand flow rate for movement (PCE/hr)
v_i = demand flow rate for movement (veh/hr)
f_{HV} = heavy vehicle adjustment factor

Circulating and exiting flow rates were then calculated. The circulating flow rates in front of each entry are summed in terms of passenger car equivalents. See HCM 2010 Equation 21-11 below.

$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Where:

$v_c =$ Circulating flow rates in front of specified entry; in passenger car equivalents

$v_{WBU,pce} =$ Flow rates of a specified movement

3.4 Empirical Queue Length Equation

From the equation and validation data sets mentioned, an equation was developed to estimate queue lengths at a roundabout. Outliers were taken out of the data set.

The empirically developed equation to predict queue for an approach:

$$Q = 25 \text{ X } \exp(-2.071 + 0.6829L + 0.4673S - 0.003466D - 0.03644I + 0.002454v_e + 0.000004307 \text{ X } v_e \text{ X } v_c + 0.0201P)$$

Where:

- Q = max queue (ft)
- L = number of legs (3 or 4)
- S = School within ½ mile of a roundabout, then 1 (0 otherwise)
- D = inscribed Diameter (ft)
- I = splitter Island width (ft)
- v_e = entry flow adjusted for PHF and vehicle type (pc/h)
- v_c = adjusted circular flow conflicts with approach (pc/h)
- P = total pedestrians or bicyclists in crosswalk (#/h)

As an additional test, only data points with queues 50 feet or greater were considered. Often accurate prediction of less than two cars is not significant to the roundabout operation. The empirical $Q \geq 50$ equation to predict queues for an approach greater than one car:

$$Q_{50} = 25 \text{ X } \exp(-0.02165 + 0.1445L + 0.2809S + 0.001321v_e + 0.000003877 \text{ X } v_e \text{ X } v_c + 0.009111P)$$

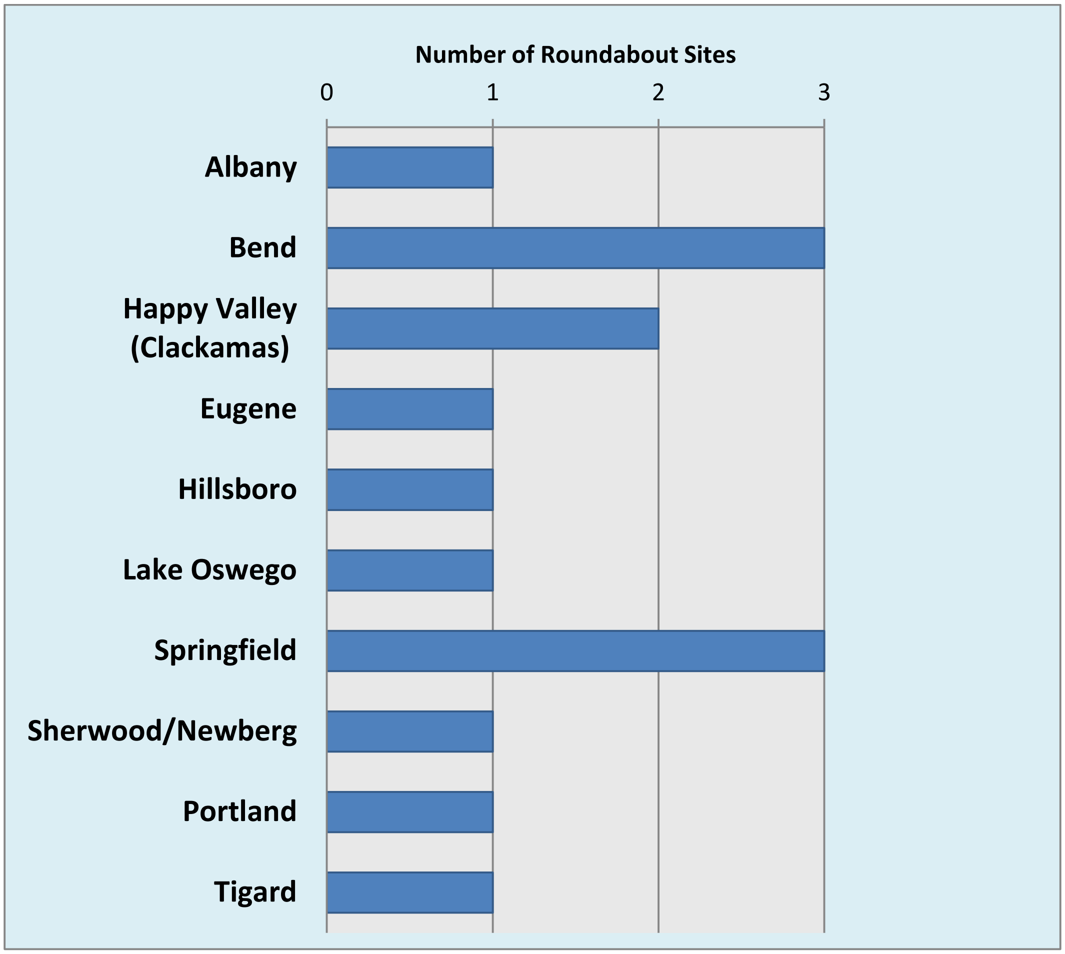
4 Validation Data

Validation tests the accuracy of equations in predicting queue lengths. Validation was processed with a subset of data previously set aside. The validation set was randomly created. The validation data, while smaller, was collected and treated just the same as the original data set.

4.1 Comparison

Data for validating the equation and comparing methodologies included 113 15 minute sample sets. The validation data set is shown in Exhibit 4-1.

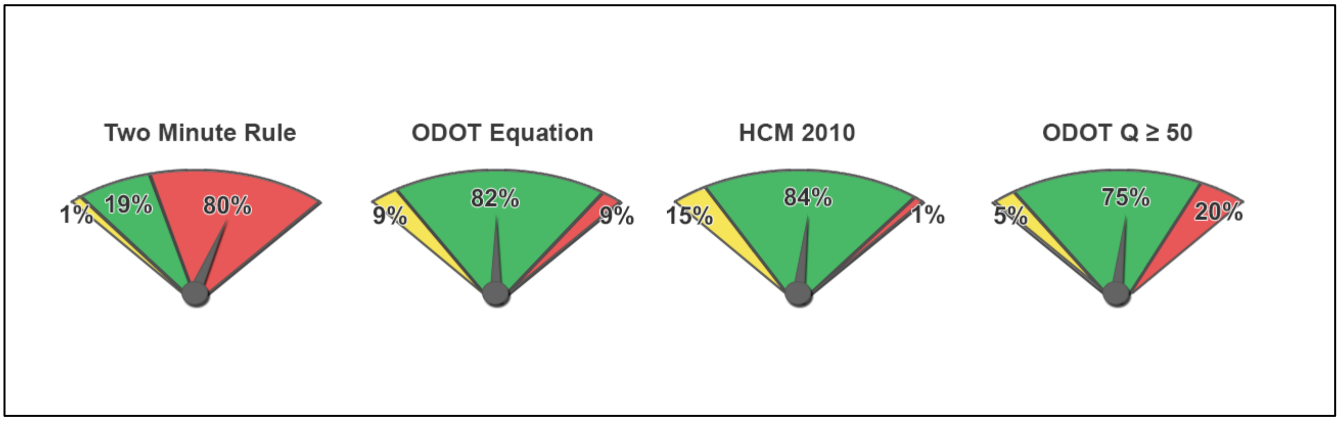
Exhibit 4-1 Validation Set: City



The validation data set included sites in Region 1, Region 2, and Region 4.

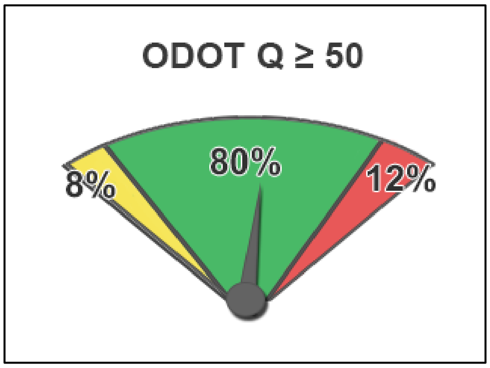
Predictive queue methods were compared using an accuracy window of being within two vehicles or 50 feet (+ or – two vehicles). In the validation set, the accuracy of the Two Minute Rule is 19%, with 80% of queue lengths overestimated. The accuracy of the HCM predictive queue methodology is 84%. The empirical equation predicted queues with an accuracy of 82%. Overall, the HCM 2010 model provided the highest accuracy level (Exhibit 4-2).

Exhibit 4-2 Results of Validation Set Comparison of Methods



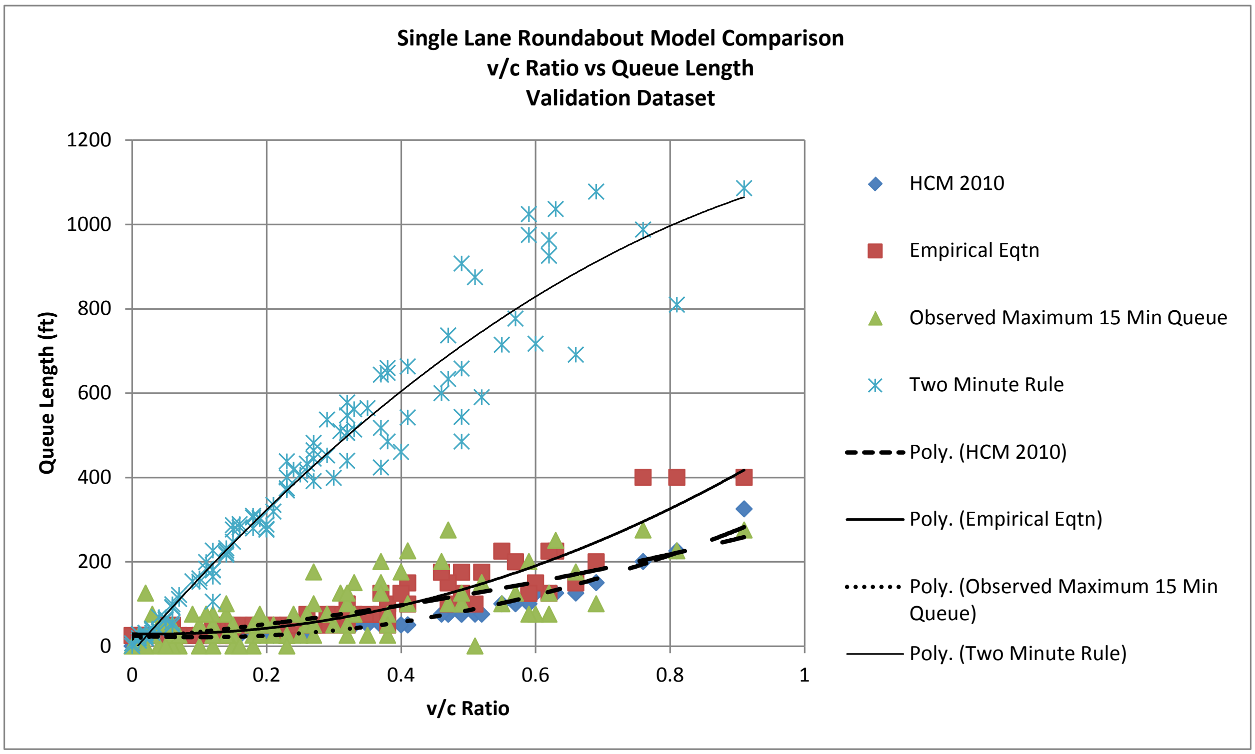
Since queue prediction may not be necessary for one or zero (1 - 0) cars at an approach, a potential alternative empirical equation was developed for queues 50 feet (two car lengths) or greater. Data points with zero or one vehicle in queue were excluded. When applied to the same data set, the accuracy was 75%. The accuracy was 80% when applied to the 65 samples that had queues of two cars or greater (Exhibit 4-3). This alternative empirical equation did not perform as well as the empirical equation which included all data points.

Exhibit 4-3 Using only 65 of 113 Samples (Queues ≥ 50)



A comparison of methodologies was also made in terms queue length versus HCM 2010-computed v/c ratio using the validation dataset. The results are shown in Exhibit 4-4. Queue lengths generally increase with v/c ratio, although other factors may affect the queue length. As shown, the Two Minute Rule significantly overestimates most of the observed data points. The HCM model and the empirically estimated equation are generally within the range of the observed data. However, the empirical equation overestimates queue lengths where v/c ratios exceed about 0.75. This may be attributable to the lack of high volume roundabouts in the estimation dataset. The HCM 2010 equation has better accuracy at the higher v/c ratios. The HCM model may also be a better estimate of 95th percentile queues, which may be slightly less than the observed maximum 15-minute queues.

Exhibit 4-4 V/C Ratio Versus Queue Length Comparison



5 Conclusions & Scope for Future Study

5.1 Conclusions

This study finds the Two-Minute Rule greatly overestimates queues at Oregon roundabouts. The estimated empirical equation has better accuracy, but overestimates queues at higher v/c ratios. The HCM 2010 equation provides the highest accuracy level, and is likely to better represent 95th percentile queue lengths which may be somewhat less than the maximum observed 15-minute queue lengths collected in this study.

The HCM 2010 roundabout queuing methodology is recommended to replace the Two-Minute Rule to estimate 95th percentile queue lengths for conditions that are applicable as per the HCM (isolated roundabouts, few pedestrians, undersaturated, etc.). For other situations alternative tools should be used, such as microsimulation.

5.2 Potential Future Research

The HCM 2010 methodology is applicable to typical isolated roundabouts. There are several limitations of the methodology as discussed in the HCM, which advises the use of alternative tools to produce more accurate results in those circumstances. Further study and development of guidance on the use of alternative methods/tools is desirable. Both deterministic software as well as microsimulation could be evaluated in order to develop guidance, settings and parameters for use.

Appendix A – Empirical Queue Length Equation

Empirical Queue Length Equation for Oregon Single-Circular-Lane Roundabouts

1. Equation Development

The roundabout data were divided into two data sets for equation development and validation respectively. The estimation data set has 244 records, and the validation data set has 112 records. The dependent variable is the maximum queue length at a roundabout leg in 15 minutes. Poisson regression is a regular method to model count data, and could be used to estimate the number of vehicles in the queue.

The following equation was developed from the estimation dataset to predict maximum 15-minute queue length. Statistical methods and engineering judgment was used to select variables.

$$Q = 25 \times exp\big(-2.071 + 0.6829\mathrm{L} + 0.4673S_c - 0.03644S_p + 0.002454v_e + 0.000004307v_e v_c + 0.0201\mathrm{P} - 0.003466\mathrm{I}\big)$$

Where:
Q = queue length (ft)
L = number of legs (3 or 4)
S_c = School within ½ mile of a roundabout, then 1 (0 otherwise)
I = Inscribed diameter (ft)
S_p = Splitter island width (ft)
v_e = entry flow adjusted for PHF and vehicle type (pc/h)
v_c = adjusted circular flow conflicts with approach (pc/h)
P = total pedestrians or bicyclists in crosswalk (#/h)

In Figure 1, observed queue lengths from data are compared with predicted queue lengths from the equation. Queue lengths less than or equal to 50 ft are slightly over-estimated but it is not an issue because queue lengths more than 50 ft are more concerned.

Figure 2 shows diagnostic plots of the model. These plots show good fit of the model. The model explains 58.2% of the variance.

Figure 1 Comparison of observed and predicted queue lengths

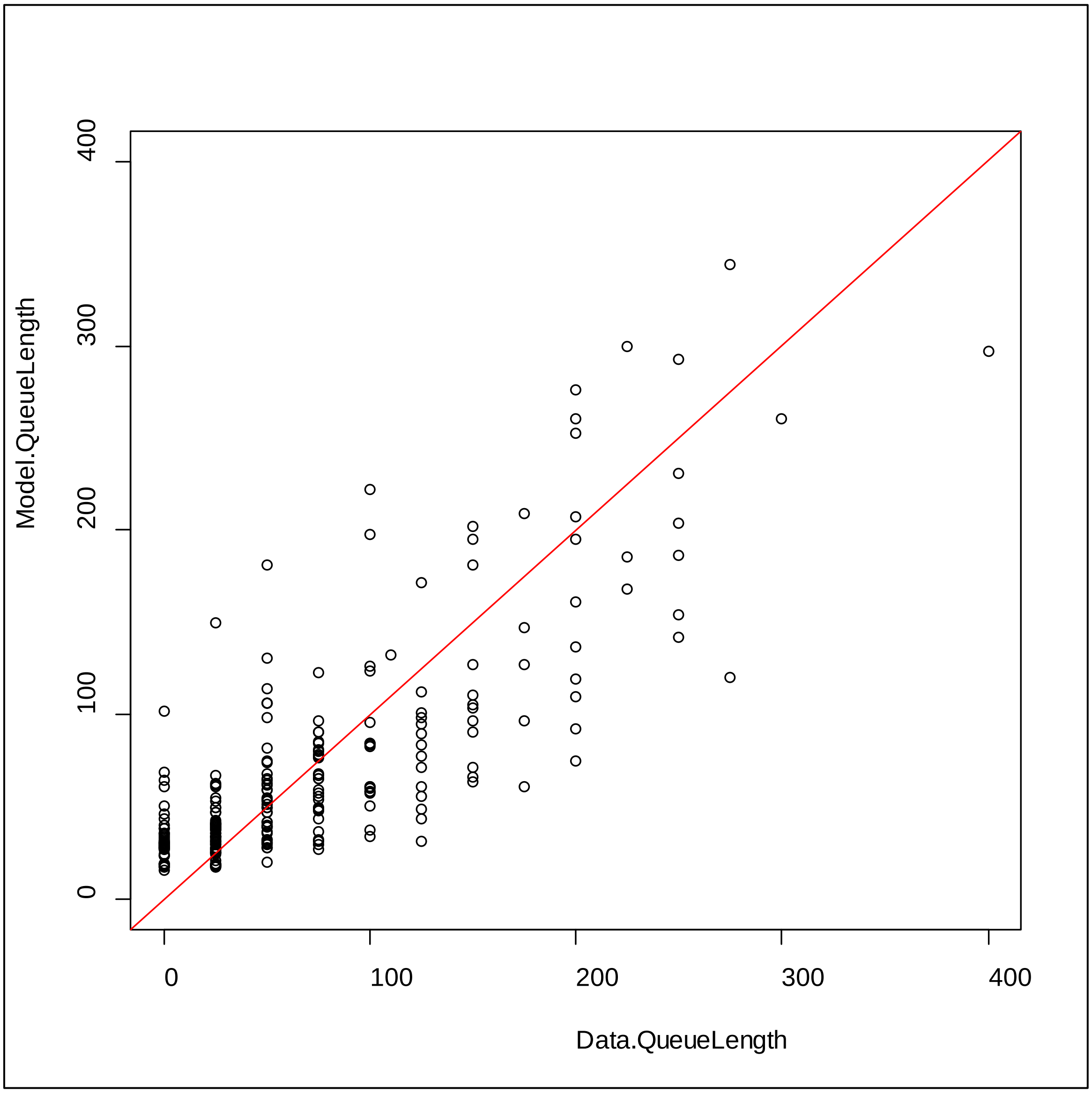
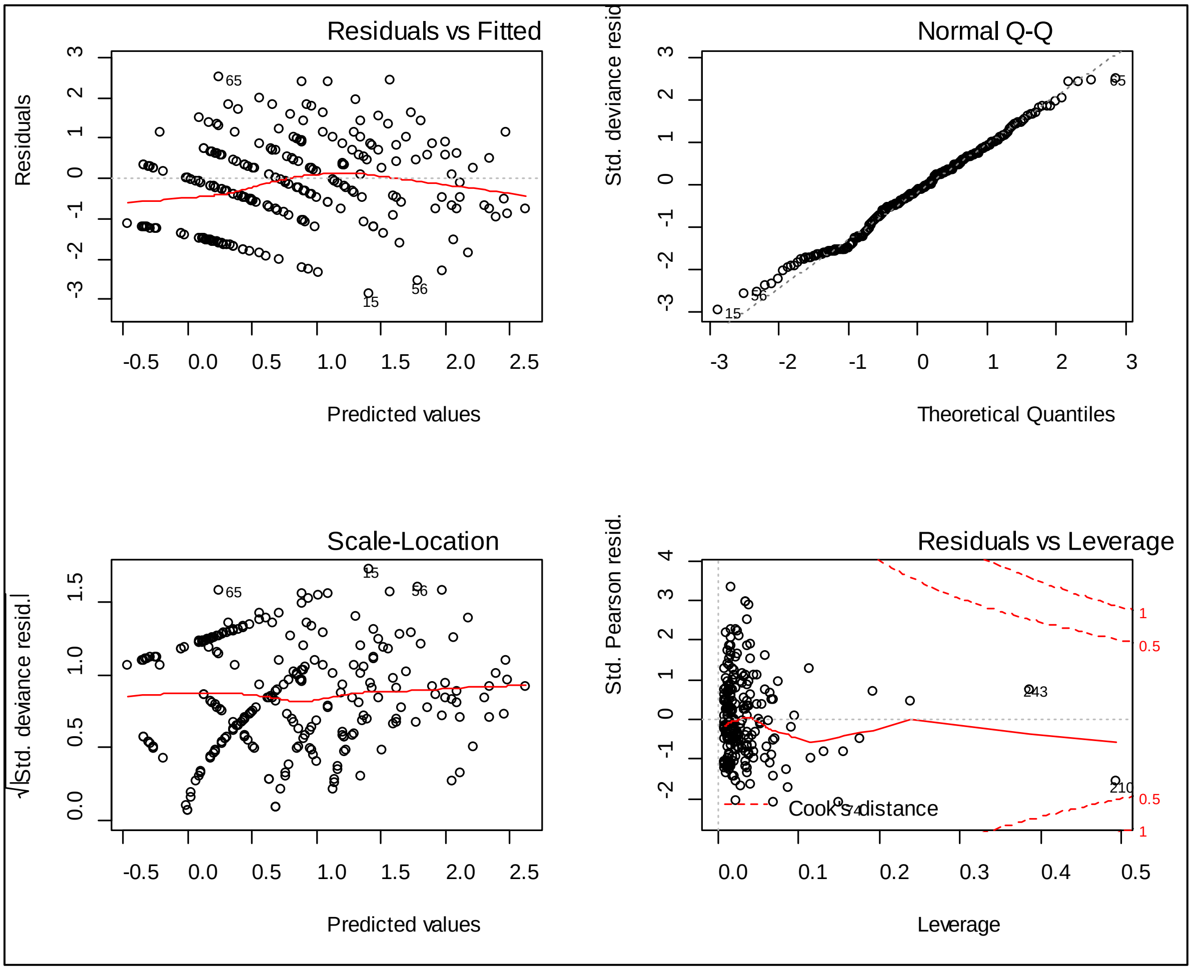


Figure 2 Diagnostic Plots of the Model



2. Sensitivity Analysis

R software provides effect display (Fox, 2003) that plots sensitivity of variables. In the estimated model, the entry flow rate and circular-lane flow rate have interactions. In Figure 3, the vertical axis is the queue length (in vehicles) generated from the model. The horizontal axis is circular-lane flow rate. The "l" in red color above each graph shows the value of the entry flow rate. The low-left graph shows that the modeled queue length is close to zero when the entry flow rate is zero. The low-right graph shows that for the entry flow rate of 233, the queue length increases when the circular-lane flow rate increases. In the up-left graph, the entry flow rate is 466 and the queue length increases with a higher rate. In the up-right graph, the entry flow rate is 700 and the queue length increases quickly when the circular-lane flow rate increases.

Figure 3 Effort Display of EntryFlowRate*CircFlowRate

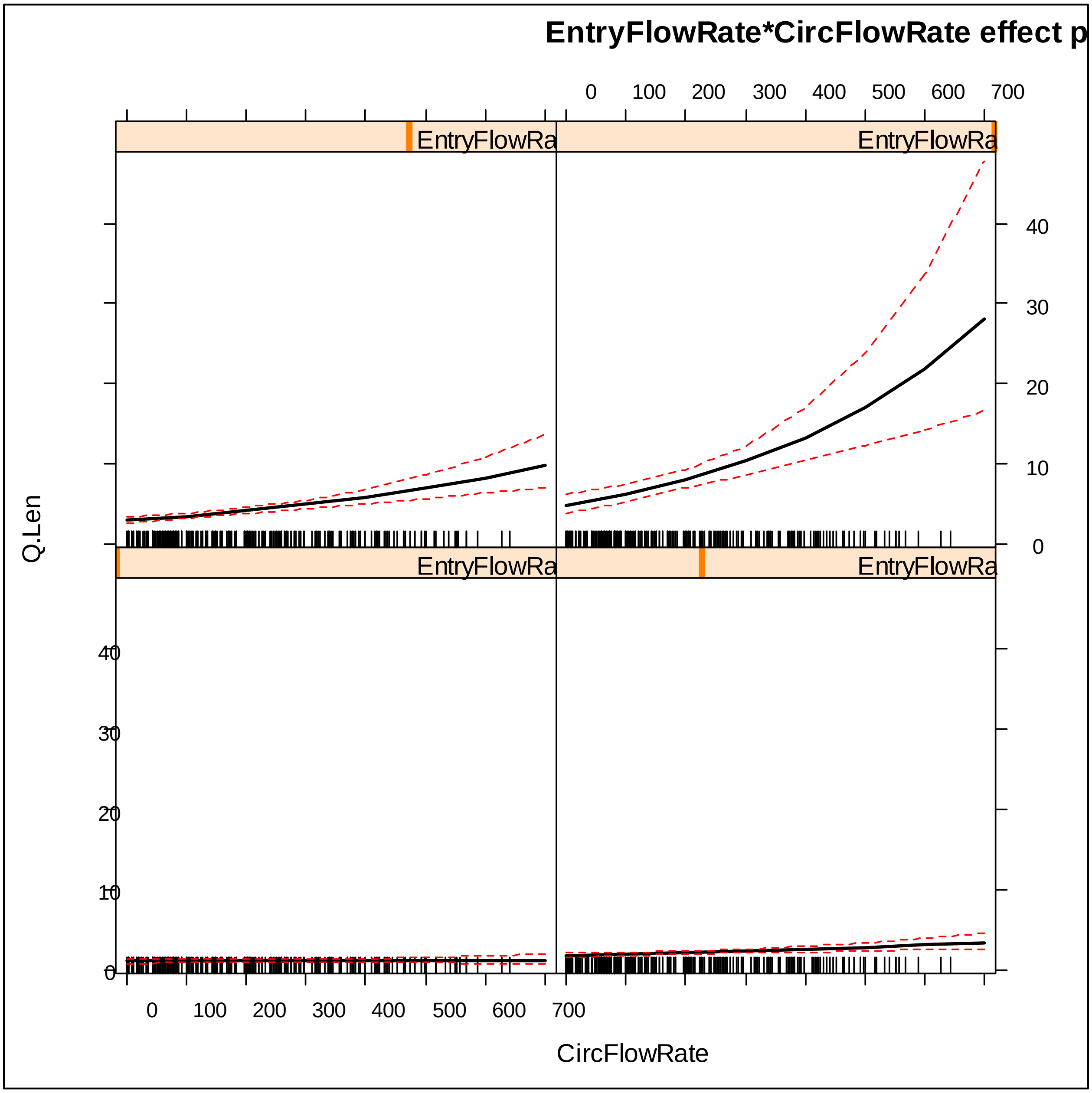


Figure 4 shows the sensitivity of other model variables. The observations are as follows. The three-leg roundabouts have less queue lengths than the four-leg roundabouts do. The queue length is larger when a school is nearby. The queue length increases when the pedestrian-crossing occurrence increases. The queue length decreases when the inscribed diameter or splitter island width increases. These observations of model sensitivity are consistent with the field observations.

Figure 4 Effect Displays of Model Variables

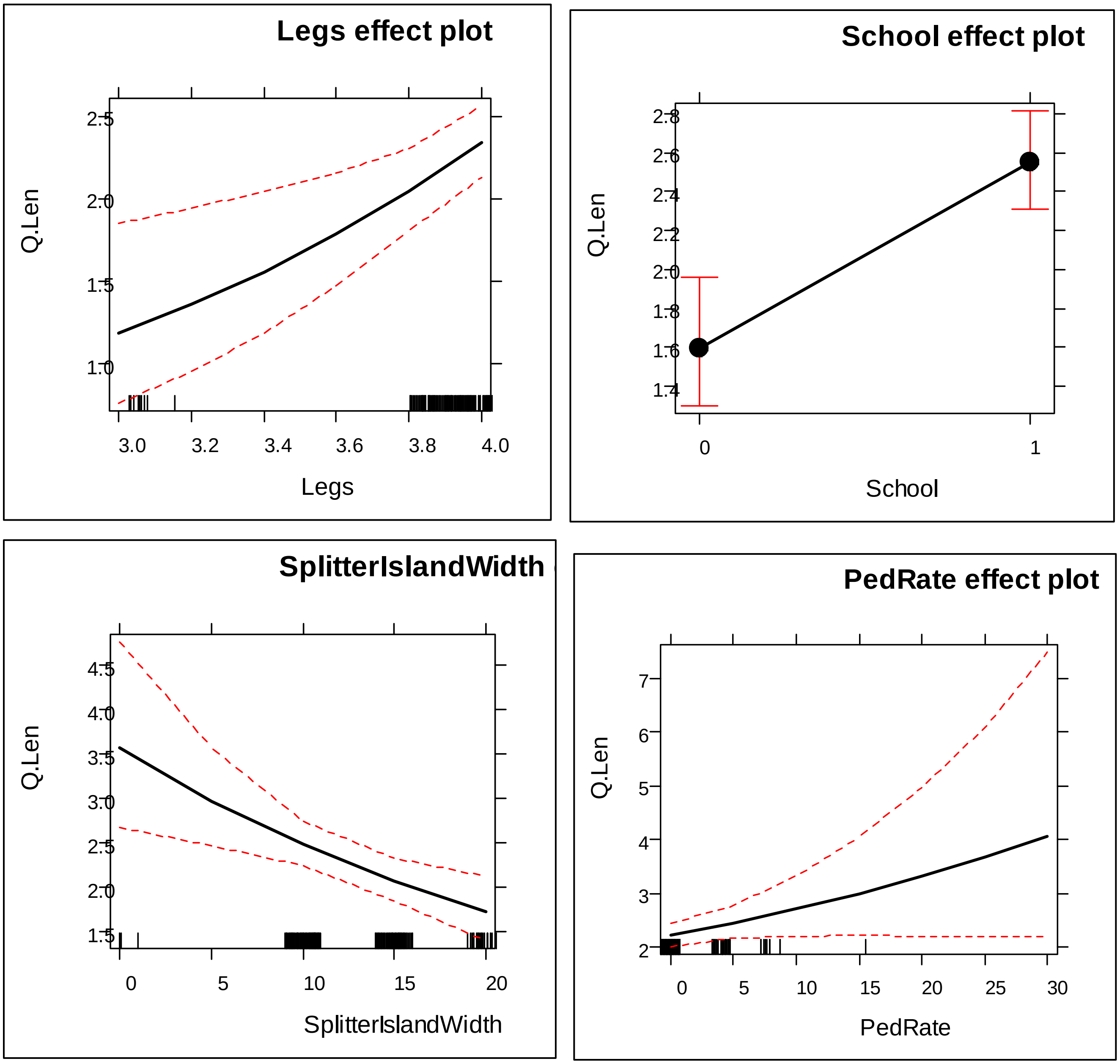
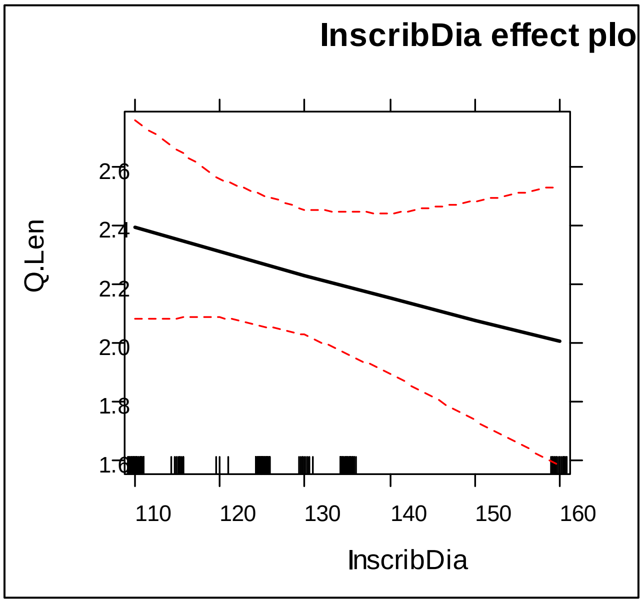


Figure 4 Effect Displays of Model Variables (continued)



References

Fox, J. (2003) Effect displays in R for generalised linear models. Journal of Statistical Software8:15, 1–27, <http://www.jstatsoft.org/counter.php?id=75&url=v08/i15/effect-displays-revised.pdf&ct=1>.

Appendix B – Data Set Sites

Table B- 1 Development Intersections

City	Intersection	# Legs	School	Inscribed Dia (ft)	Splitter Island Width (ft) (N, E, S, W)
Albany	NW Gibson Hill Rd and NW North Albany Rd	4	Yes	125	15, 15, 20, 20
Bend	Butler Market Rd and NE 8th Street	3	Yes	120	NA, 10, 10, 0
Bend	Franklin Ave and NE 8th Street	4	Yes	125	10, 10, 10, 10
Bend	NW Shevlin Park Rd/Newport Ave and NW College Way	4	No	135	10, 10, 15, 15
Happy Valley	Monteray Ave and Stevens Rd	4	Yes	135	20, 20, 15, 15
Happy Valley	Monteray Ave and Causey Ave	3	Yes	130	15, 15, 10, 15
Hillsboro	SE Alexander St and SE Brookwood Ave	4	Yes	160	15, 20, 10, 10
Lake Oswego	SW Stafford Rd and Rosemont Rd/Atherton Dr	4	Yes	135	15, 15, 15, 15
Springfield	Thurston Road and 58th Street	4	Yes	110	10, 10, 10, 10
Springfield	Jasper Road and 42nd Street	4	Yes	125	20, 20, 15, 15
Springfield	Corporate Way (Maple Island Farm Rd) and International Way	4	No	110	0, 10, 10, 10
Portland	SW Terwilliger and SW Palater Rd	4	Yes	125	20, 10, 15, 10
Tigard	Barrows Rd and Roshack Rd	4	No	115	15, 15, 15, 15

Table B- 2 Validation Intersections

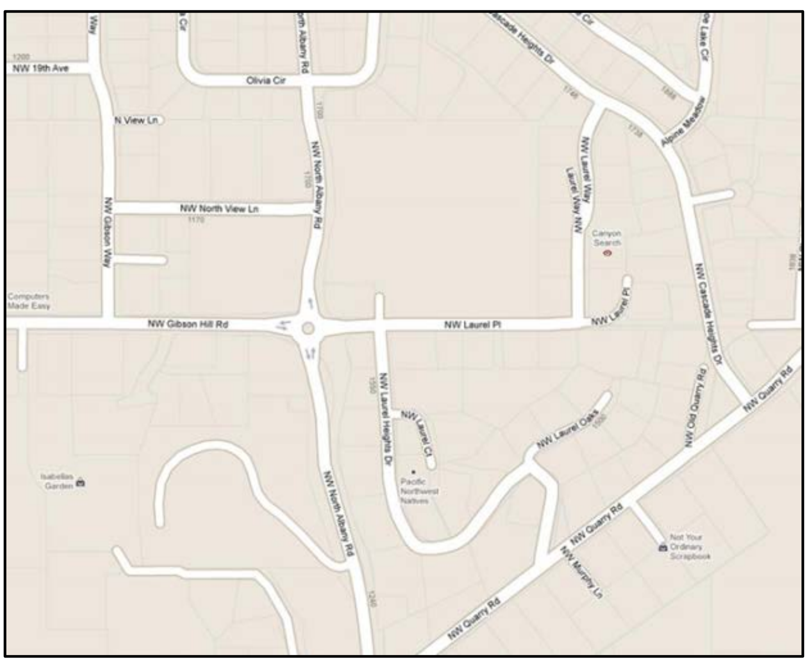
City	Intersection	# Legs	School	Inscribed Dia (ft)	Splitter Island Width (ft) (N, E, S, W)
Albany	NW Gibson Hill Rd and NW North Albany Rd	4	Yes	125	15, 15, 20, 20
Bend	Butler Market Rd and NE 8th Street	3	Yes	120	NA, 10, 10, 0
Bend	Franklin Ave and NE 8th Street	4	Yes	125	10, 10, 10, 10
Bend	NW Shevlin Park Rd/Newport Ave and NW College Way	4	No	135	10, 10, 15, 15
Happy Valley	Monteray Ave and Stevens Rd	4	Yes	135	20, 20, 15, 15
Happy Valley	Monteray Ave and Causey Ave	3	Yes	130	15, 15, 10, 15
Hillsboro	SE Alexander St and SE Brookwood Ave	4	Yes	160	15, 20, 10, 10
Eugene	Barger Dr and Green Hill Rd	3	Yes	125	10, 10, 10, NA
Lake Oswego	SW Stafford Rd and Rosemont Rd/Atherton Dr	4	Yes	135	15, 15, 15, 15
Sherwood/ Newberg	Crestview and Springbrook	4	Yes	200	25, 20, 25, 30
Springfield	Thurston Road and 58th Street	4	Yes	110	10, 10, 10, 10
Springfield	Jasper Road and 42nd Street	4	Yes	125	20, 20, 15, 15
Springfield	Corporate Way (Maple Island Farm Rd) and International Way	4	No	110	0, 10, 10, 10
Portland	SW Terwilliger and SW Palater Rd	4	Yes	125	20, 10, 15, 10
Tigard	Barrows Rd and Roshack Rd	4	No	115	15, 15, 15, 15

Appendix C – Site Descriptions

Albany, NW Albany Rd and NW Gibson Hill Rd

	Study Periods	Bikes	Peds
Bicyclists & Pedestrians	6:30 – 7:30 AM	0	0
	4:45 – 5:45 PM	0	0
# legs	4		
Heavy Veh.	Few		
Schools	North Albany Elementary North Albany Middle School Oak Grove Elementary Fairmont Elementary		

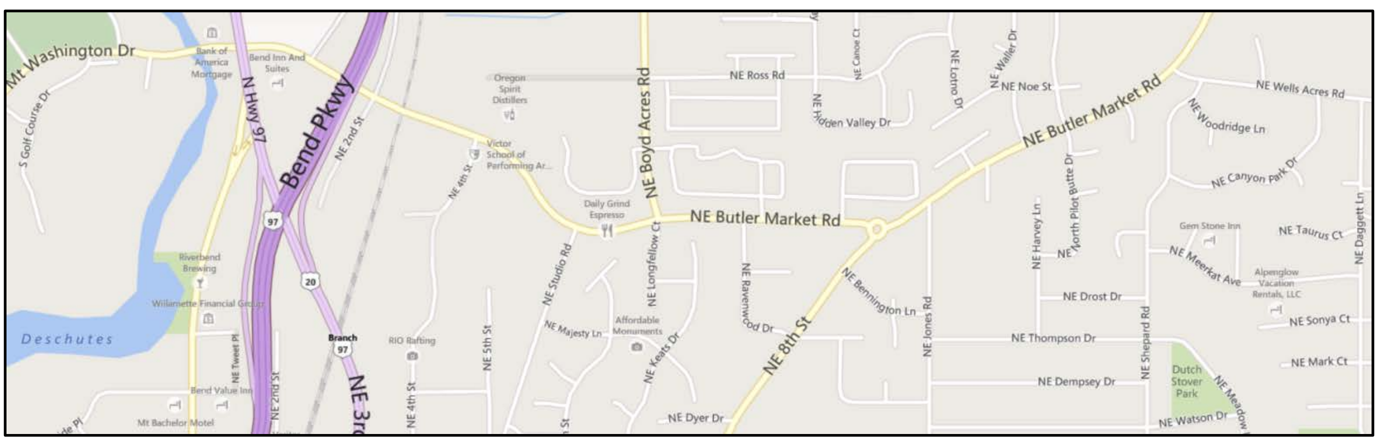
There are two heavy movements that are overlapping. There are no serious issues with the grade on approaches. Good restriction of plants in center.



Bend, Butler Market Rd and NE 8th Street

	Study Periods	Bikes	Peds
Bicyclists & Pedestrians	7:30 – 8:30 AM	4	0
	2:30 – 3:30 PM	4	0
	4:30 – 5:30 PM	4	0
# legs	3		
Heavy Veh.	Few		
Schools	Pilot Butte Middle School Juniper Elementary		

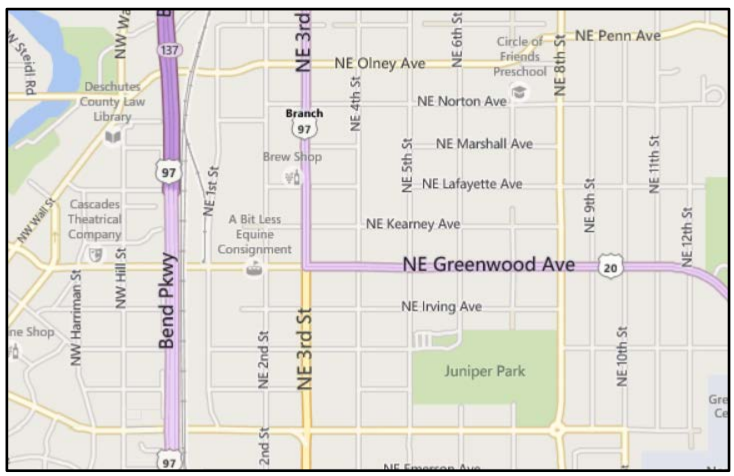
Somewhat transparent art object in the center. To the west, Butler Market Road connects with Mt Washington Drive, Bend Parkway, and US97. Continuing east, Butler Market Road is a major connector that intersects with 27th Street, Eagle Road, and Hamby Road. NE 8th Street to the South is a significant connector that parallels the Bend Parkway.



Bend, Franklin Ave and NE 8th Street

Bicyclists & Pedestrians	Study Periods	Bikes	Peds
	7:15 – 8:15 AM	3	0
	2:30 – 3:30 PM	5	6
	4:00 – 5:00 PM	9	12
# legs	4		
Heavy Veh.	Several		
Schools	Bend High School Marshal High School		

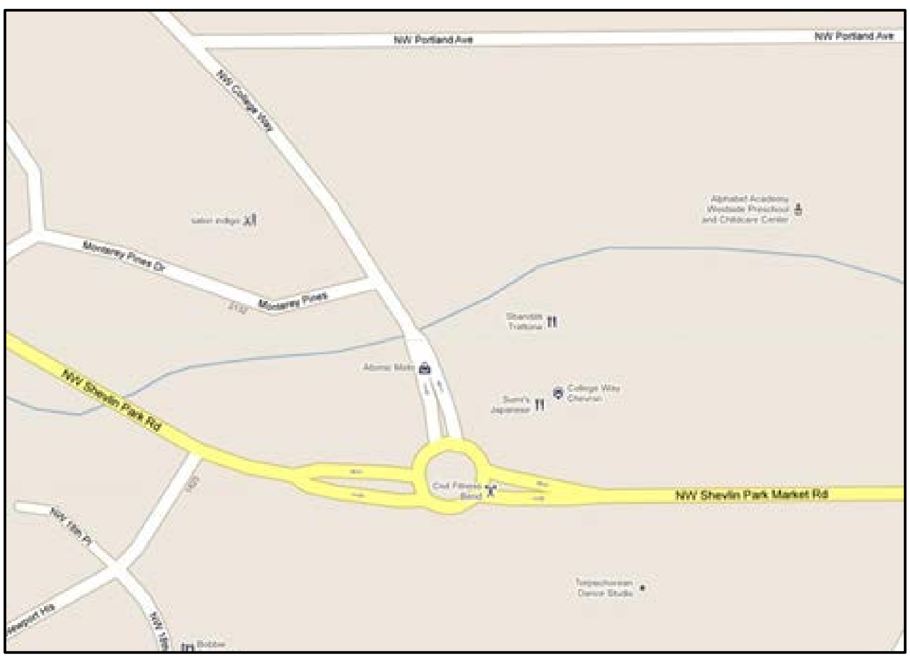
Placing drainage grates in the wheel path cases an undesirable additional stop by drivers. This is a nice place to slow traffic at the corner of a park.



Bend, NW Shevlin Park Rd/Newport Ave and NW College Way

Bicyclists & Pedestrians	Study Periods	Bikes	Peds
	7:15 – 8:15 AM	6	4
	5:00 – 6:00 PM	4	3
# legs	4		
Heavy Veh.	Several		
Schools	N/A		

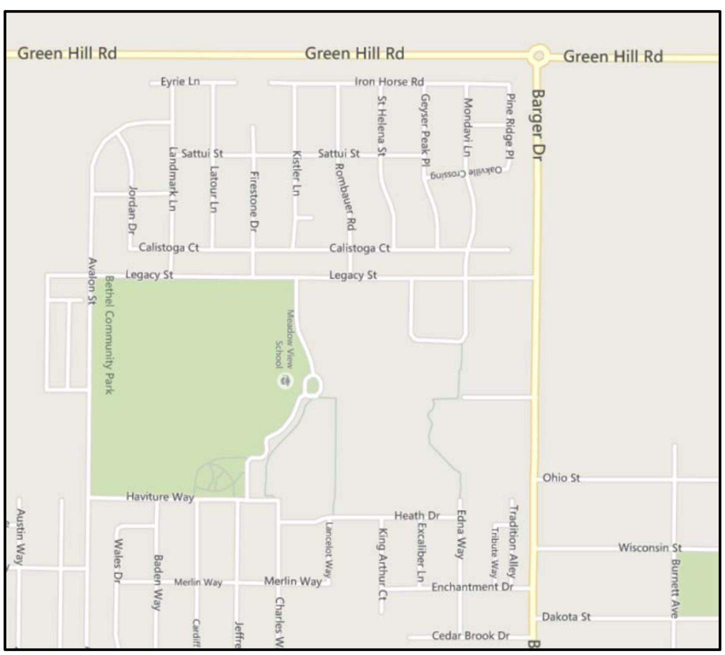
This is now a four leg roundabout. Large WB67 vehicles were seen competently crossing the median into the gas station. College Way has a very significant grade to it. Note how the city was able to easily add a fourth approach to this intersection.



Eugene, Barger Dr and Green Hill Rd

Bicyclists & Pedestrians	Study Period	Bikes	Peds
	4:45 – 5:45 PM	0	5
# legs	4		
Heavy Veh.	Moderate		
Schools	Witch Hazel Elementary Southern Meadows Middle School		

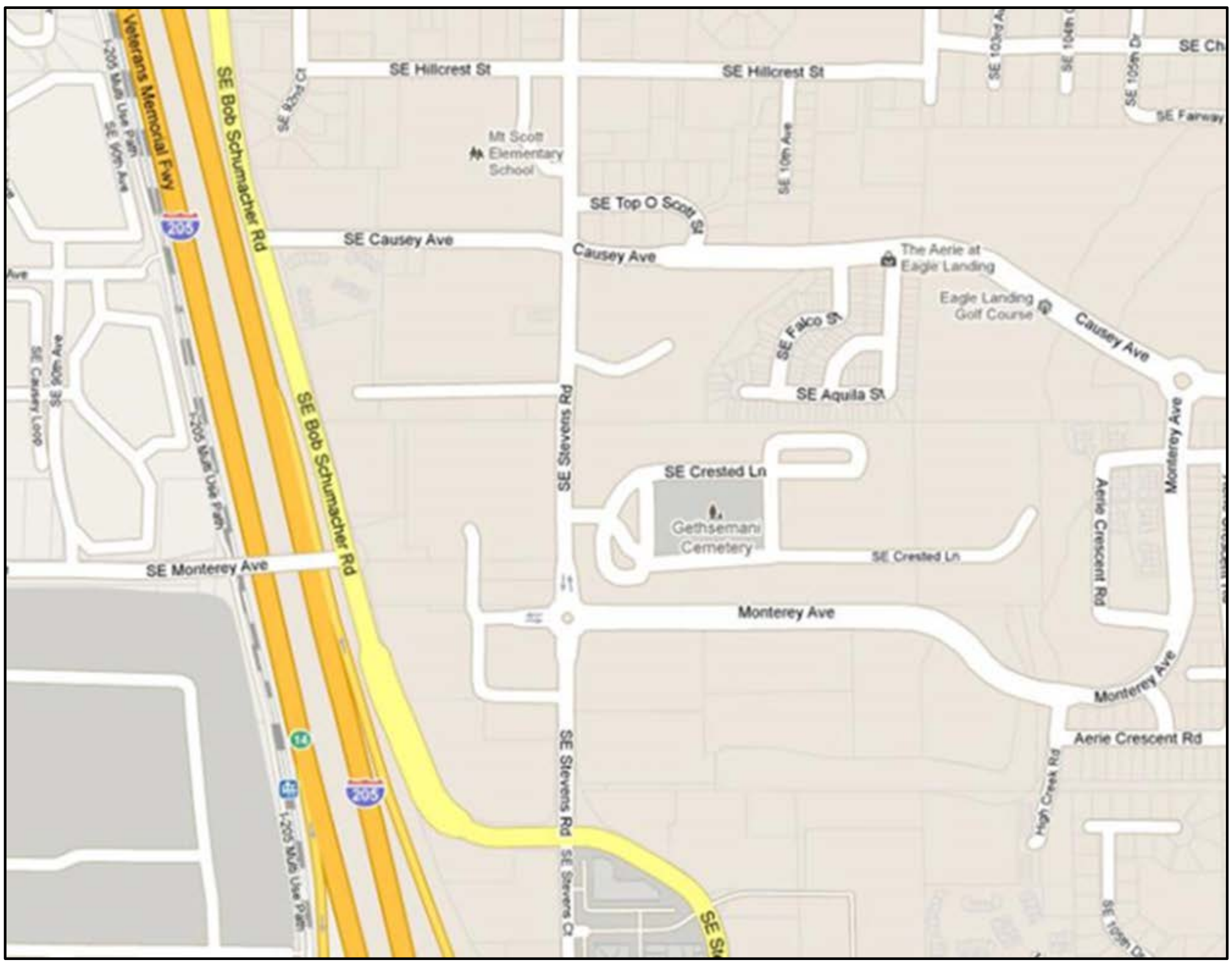
Note double yield signs. This roundabout functioned very well with a modest amount of items in the circular island. There are some utilities that seem close, but apparently has not been an issue to date. This roundabout is located on the east side of Eugene. Clear Lake Road is to the north, Royal Avenue is to the south.



Happy Valley, Monterey Ave and Stevens Rd

Bicyclists & Pedestrians	Study Period	Bikes	Peds
	6:15 – 9:00 AM	2	3
# legs	4		
Heavy Veh.	Few		
Schools	Mt Scott Elementary, Little Explorers Kindergarten		

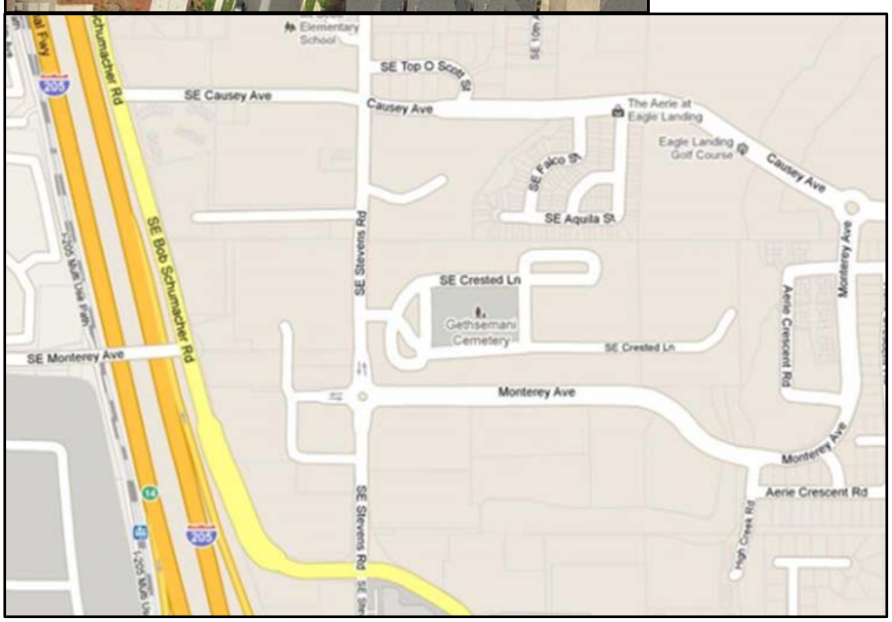
No queues beyond 2 cars. The 4th leg is Hope Community Church entrance. There is a significant grade difference to parking lot. Vehicle observed stopping and taking a picture from the circulatory roadway. One vehicle cut off another as they were staring at the eagle.



Happy Valley, Monterey Ave and Causey Ave

Bicyclists & Pedestrians	Study Period	Bikes	Peds
	7:00 – 7:45 AM	2	3
# legs	3		
Heavy Veh.	None		
Schools	Mt Scott Elementary Little Explorers Kindergarten		

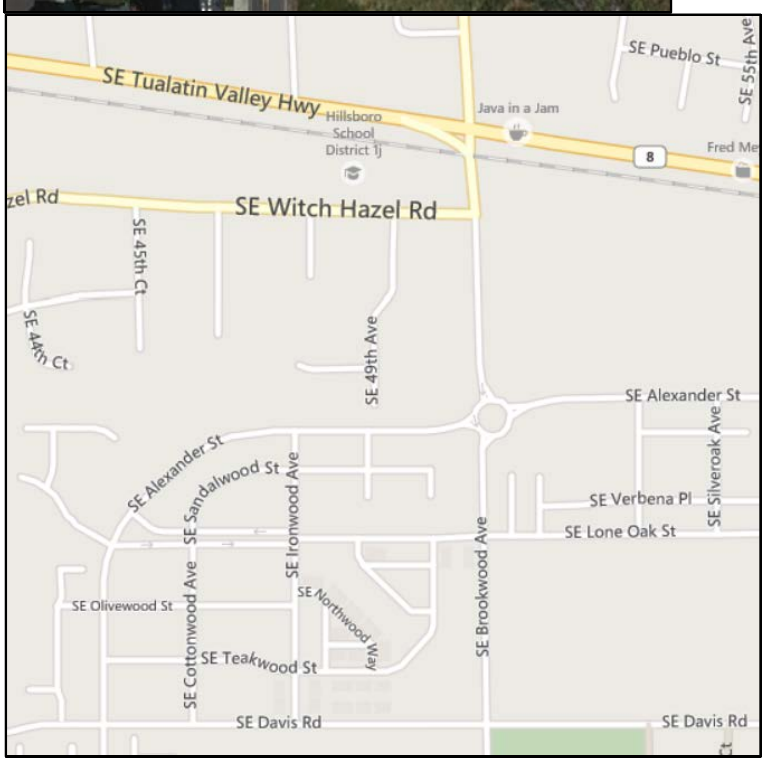
Golf course access is located nearby. Low volume, longest queue was one vehicle. No bicyclists or pedestrians in observed hours. Advertising sandwich signs were placed in the truck apron.



Hillsboro, SE Alexander St and SE Brookwood Ave

	Study Periods	Bikes	Peds
Bicyclists & Pedestrians	7:45 – 8:45 AM	0	2
	4:15 – 5:15 PM	0	5
# legs	4		
Heavy Veh.	Moderate		
Schools	Witch Hazel Elementary Southern Meadows Middle School		

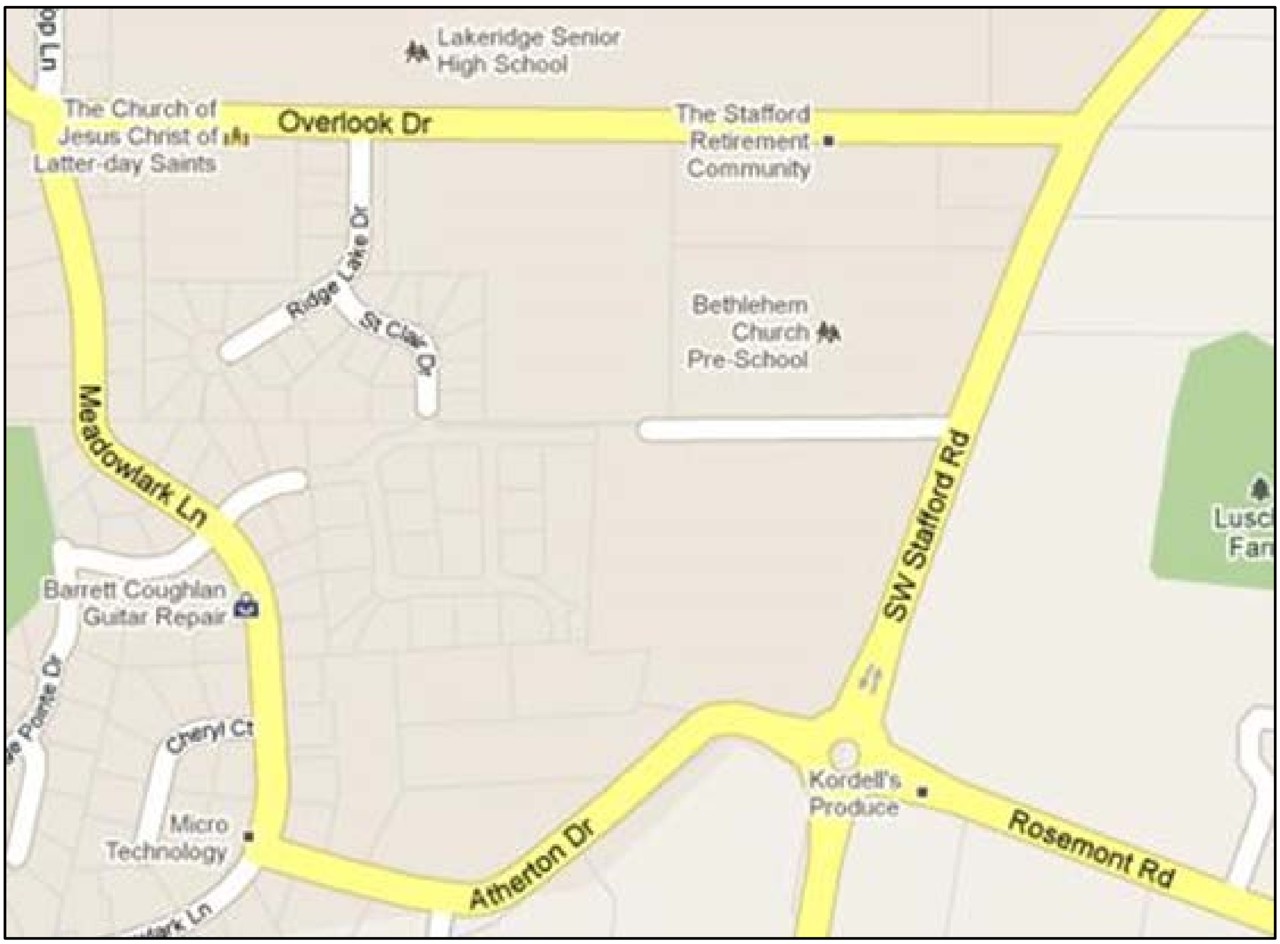
There is a railroad crossing just south of the Tualatin Valley Highway, OR8 to the north. With no obstructive feature, this roundabout operates very well. The bicycle ramps seem appropriate, where there are bicycle lanes.



Lake Oswego, SW Stafford Rd & Rosemont Rd/Atherton Dr

Bicyclists & Pedestrians	Study Periods	Bikes	Peds
	7:00 – 8:00 AM	0	0
	2:15 – 3:15 PM	0	1
# legs	4		
Heavy Veh.	Moderate		
Schools	Lake Ridge High School		

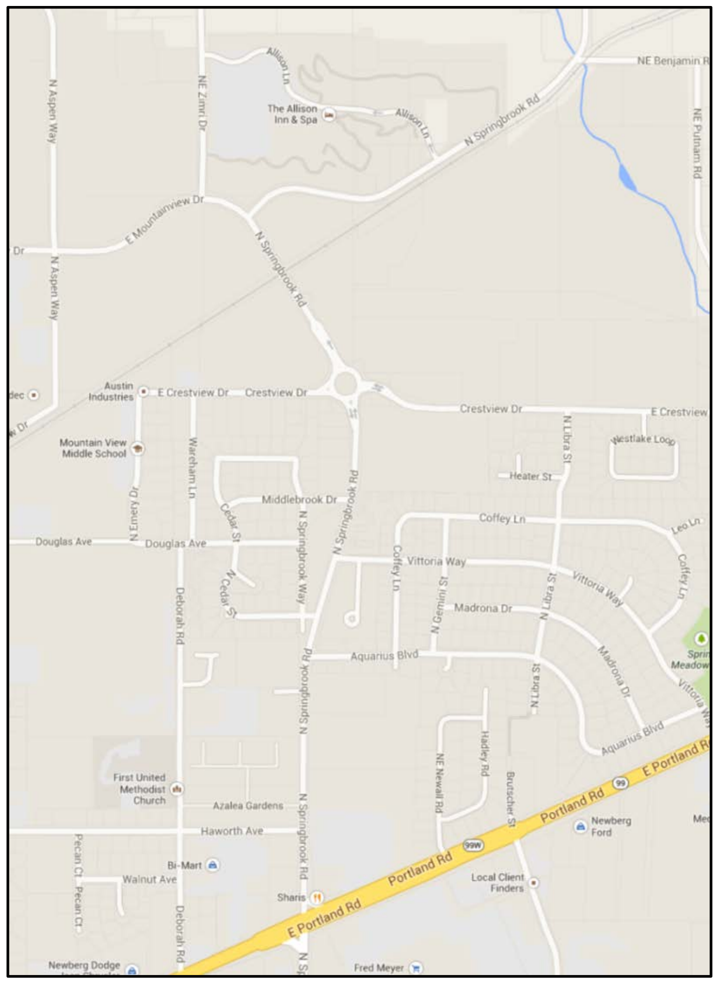
To the north Stafford Road leads to Lake Ridge High School, Lake Oswego Golf Course, and Lake Oswego. Stafford Road and Rosemont both eventually lead to I205. Where "sidewalk" exists, it is in the form of an asphalt multi-use path.



Newberg, Crestview Dr and Springbrook Rd

	Study Periods	Bikes	Peds
Bicyclists & Pedestrians	7:00 – 8:00 AM	0	0
	2:15 – 3:15 PM	0	1
# legs	4		
Heavy Veh.	Moderate		
Schools	Lake Ridge High School		

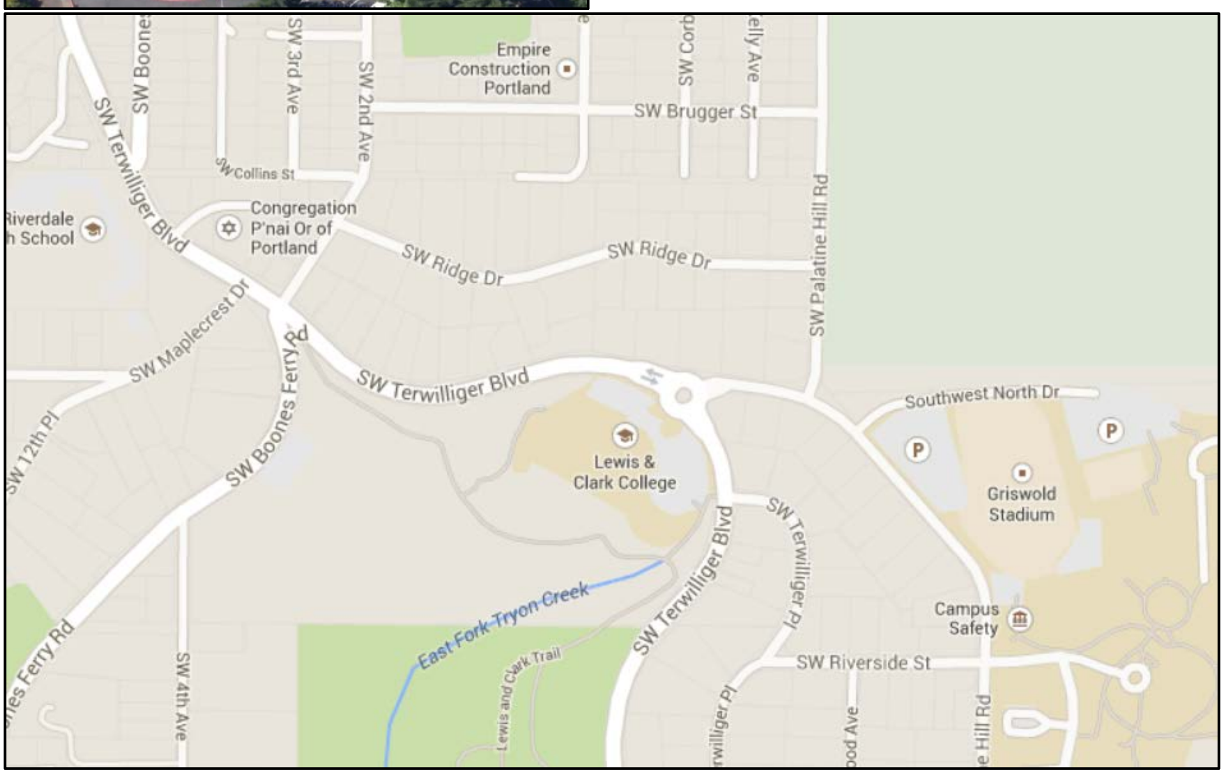
To the north Stafford Road leads to Lake Ridge High School, Lake Oswego Golf Course, and Lake Oswego. Stafford Road and Rosemont both eventually lead to I205. Where "sidewalk" exists, it is in the form of an asphalt multi-use path.



Portland, SW Terwilliger Blvd and Palater Rd

Bicyclists & Pedestrians	Study Periods	Bikes	Peds
	9:00 – 10:00 AM	9	16
	2:45 – 3:45 PM	10	4
# legs	3/4		
Heavy Veh.	Few		
Schools	Riverdale High School Lewis & Clark University		

This was a three leg intersection, with a fourth leg serving a law school. One leg serves as access to a park. The roundabout also serves two house driveway accesses. There were some heavy vehicles/buses. The truck apron is ineffective and not discernable by the travelling vehicles and is driven over regularly. This roundabout operates well with grades.

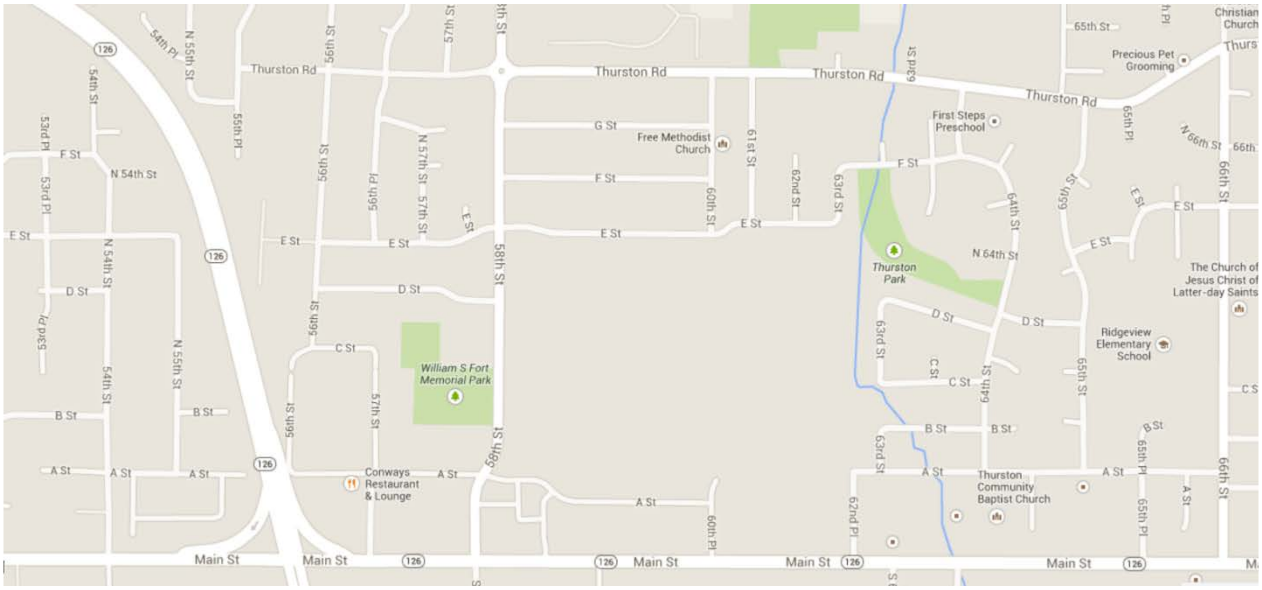


Springfield, Thurston Road & 58th Street

Bicyclists & Pedestrians	Study Periods	Bikes	Peds
	7:30 – 8:30 AM	1	6
	2:30 – 3:30 PM	0	5
# legs	4		
Heavy Veh.	Moderate (buses)		
Schools	Thurston High School		

This roundabout, built in 2001, is retrofit of a two-way stop. This roundabout helps prepare northbound drivers for the 10 mph curve beyond the roundabout. Bike lane ends sign is not common around roundabouts.

Thurston Road is one of few routes that parallels OR126/Main Street.

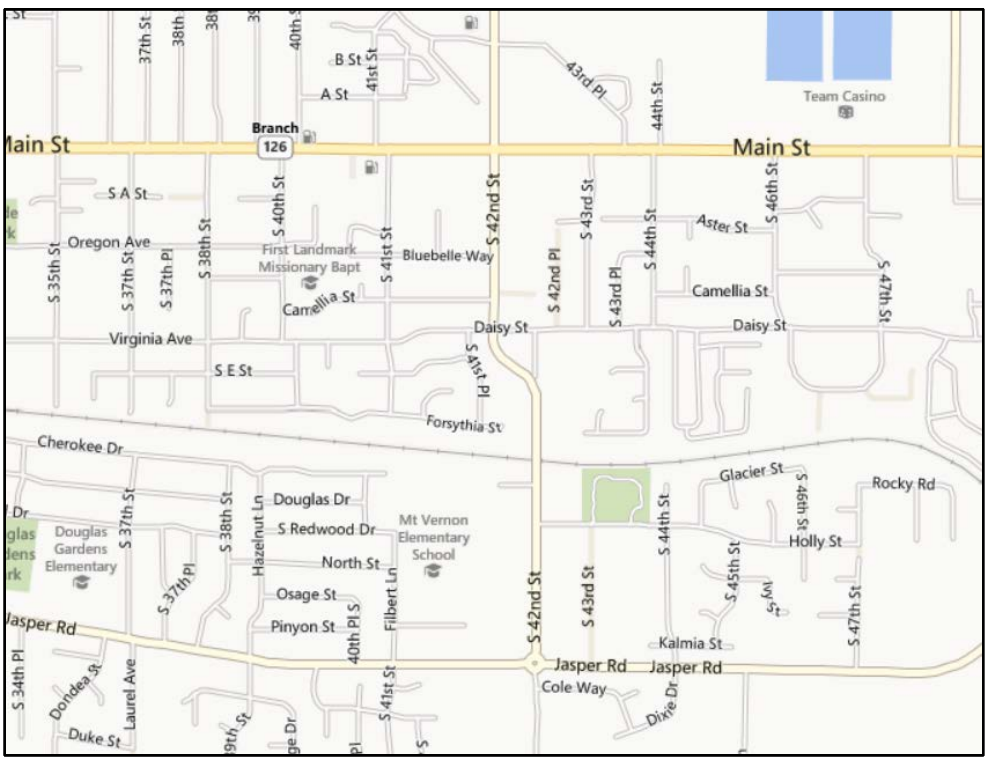


Springfield, Jasper Rd and 42nd

	Study Periods	Bikes	Peds
Bicyclists & Pedestrians	7:30 – 8:30 AM	0	0
	2:45 – 3:45 PM	0	0
# legs	4		
Heavy Veh.	Moderate (buses)		
Schools	Mt Vernon Elementary		

This is a roundabout on what is known as on OR222, the Springfield-Creswell Highway. The OR222 is marked on the north and east legs of this intersection. The customers used the convenience store in the northeast corner with ease.

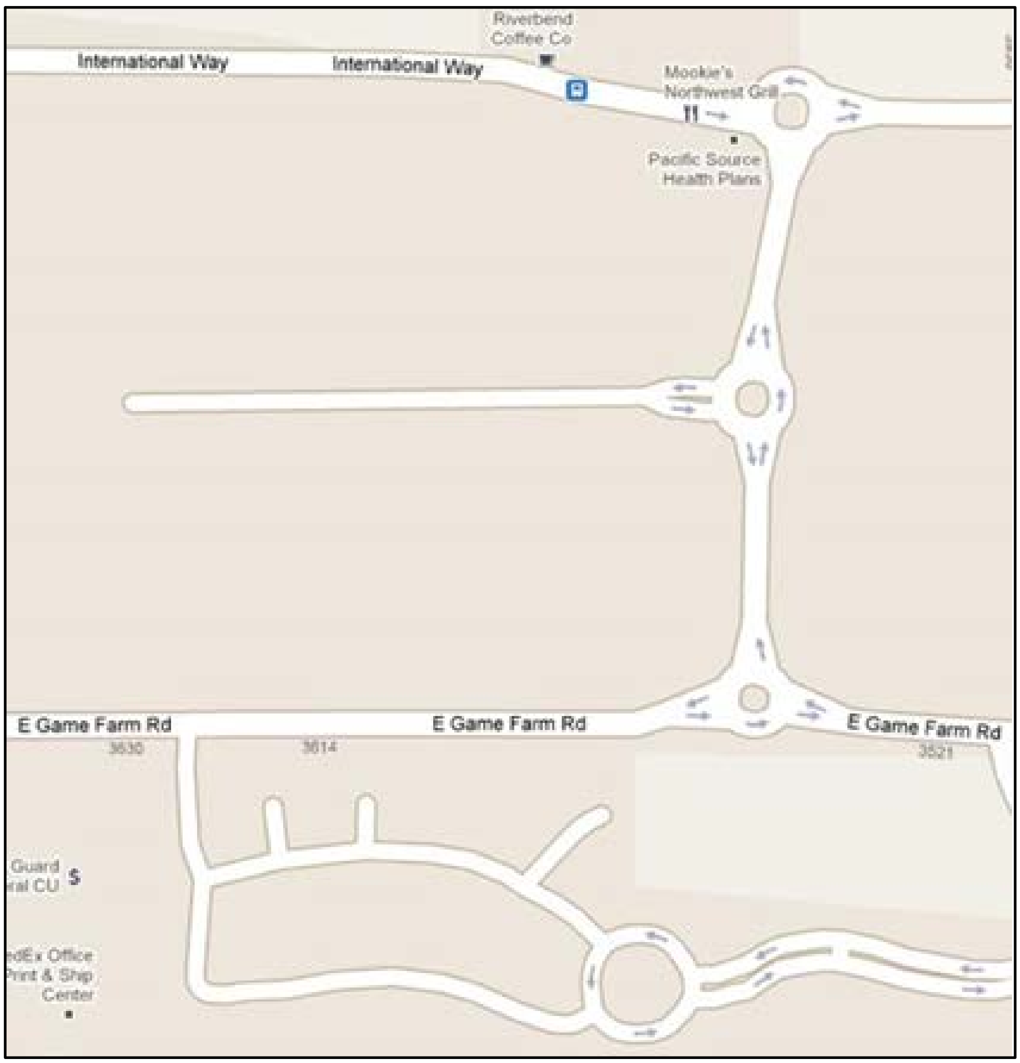
There is a neighborhood to the south. The northwest corner is a field that may develop at some point in the future.



Springfield, Maple Island Rd and International Way

	Study Periods	Bikes	Peds
Bicyclists & Pedestrians	11:30 AM – 12:30 PM	0	0
	4:45 – 5:45 PM	0	0
# legs	4, 3 splitter islands		
Heavy Veh.	Few (2 buses)		
Schools	N/A		

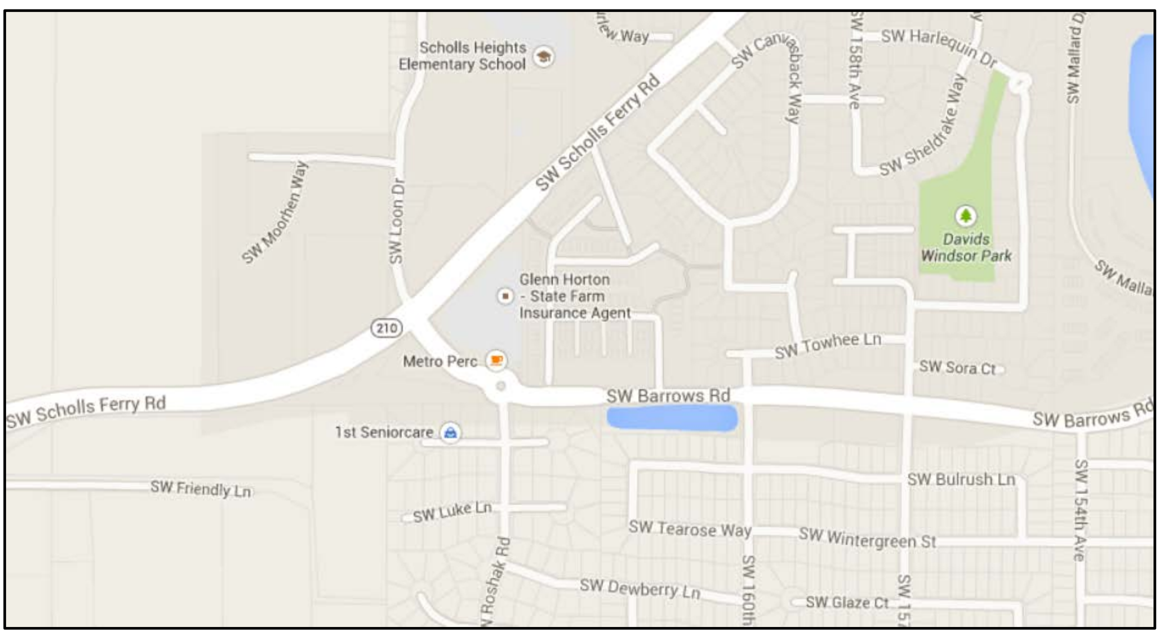
Note the double yield signs. The fourth leg is a shared access to a couple of businesses and a café. This is one of three roundabouts on Maple Island Road.



Tigard, Barrows Rd and Roshak Rd

Bicyclists & Pedestrians	Study Period	Bikes	Peds
	5:15 – 6:15 PM	15	2
# legs	4		
Heavy Veh.	Few		
Schools	N/A		

Note the double yield signs. SW Scholls Ferry Road, OR210, one block north, a business development to the east, a neighborhood to the west, and a gated fire access with mountable splitter island to the south. This roundabout operates well with grade. This roundabout has truck aprons like bulb outs on the corners between legs.



Appendix D – Data Collection Procedures

Procedures

Roundabouts were scoped for camera and observation locations. These were the procedures:

- Ensured all equipment from the materials list is packed in the car
- Double checked that PPE was packed for each person; class two vests and caps
- Scoped area for good location for tripod camera and station to count queues
- Located parking spot for state vehicle
- Set up tripod and telescoping pole/camera (sun glare)
- Ensured station was a safe location to measure queues
- Planned escape route from location if needed
- Measured distances of 50 feet, 100 feet, and 150 feet (50ft = 15.24m), placed metal/plastic flags at each distance
- With clip board, pen, and paper recorded queues

Materials List

These were the materials used.

- Procedures
- Roller Wheel
- Clip Boards
- Writing devices
- Paper/work sheets
- Plastic Flags, chalk for backup
- Red Hats
- Class 2 Vests
- Sunglasses
- Vehicle

Camera items

- Pole = 6.1 ft
- Tripod
- Battery Unit
- Charger (every night)
- Laptop and software
- Security cord, lock, and chain
- 9 - 20 lb. weights
- Ratchet strap

STATE OF OREGON | WHITE PAPER

Department of Transportation
Transportation Planning Analysis Unit
Mill Creek Office Park
555 13th Street NE Suite 2
Salem, Oregon 97301-4178
(503) 986-4112 FAX (503) 986-4174

File Code:

Date: July 1, 2008

AUTHORS: Alex Bettinardi, P.E., Transportation Analyst
Doug Norval, P.E., Transportation Analyst Engineer
Peter Schuytema, P.E., Senior Transportation Analyst
Dorothy Upton, P.E., Senior Transportation Analyst

SUBJECT: Simulation Guidelines Project (February 2006 – May 2008)

Executive Summary

The purpose of this project was to advance ODOT's simulation procedures and guidelines for planning and project analysis. The findings from this project are to be used to update the Analysis Procedures Manual, Chapter 8. The project used Trafficware's SimTraffic/SYNCHRO software, however it was developed to be as independent of specific software as possible.

The project set out to determine the different calibration needs of study areas by area type; small urban, small-medium Metropolitan Planning Organization (MPO), Large MPO (METRO), and recreational areas. Two representative sites for each area type were selected by the team creating eight locations to analyze. For each location, field data was collected, the data was post processed and evaluated, and a series of calibration tests were run for determining the best calibration procedure by area type. The data collected and the calibration tests performed were designed after conducting a thorough literature review of the latest research and calibration methods. From that literature review, speed, headway, and driver reaction time were found to be the calibration parameters that best matched the project objectives.

After all of the data was collected and the calibration testing was complete the analysis showed that a consistent calibration procedure could not be found that was applicable to all locations or any subgroup of locations. The conclusion found by this work was similar to the findings from the literature review; calibration can be improved by collecting additional field data and incorporating it into one's model, however there is no ideal combination of calibration parameters and data across all models / projects, so engineering judgment needs to be applied during the calibration process to determine what/when/why/where/how adjustments to the model and/or additional data is needed to achieve an acceptable level of calibration.

This study did develop a series of guidelines, checklists, and thresholds to help aid analysts in the calibration process. Also, calibration requirements using SimTraffic's "vehicles exited" measure of effectiveness (MOE) was established from this study.

Introduction

The Transportation Analysis Planning Unit (TPAU) of the Oregon Department of Transportation (ODOT) is constantly working to improve the traffic analysis procedures that it recommends for use on Oregon highways. The Analysis Procedures Manual (APM) is a comprehensive guidebook of all the procedures that TPAU has developed. One of the sections of the APM that needed attention and refinement was the methodology suggested for micro-simulation analysis of projects (Chapter 8).

Prior to this work, the APM explained the inputs that went into a micro-simulation, but did not provide guidance on how to adjust these parameters and how to measure the performance of an analyst's micro-simulation, beyond citing the FWHA toolbox for additional information on calibration. To improve guidance on the proper way to apply micro-simulation analysis for the projects in the State of Oregon, TPAU formed this study to develop a set of procedures and criteria that would produce higher level of accuracy and precision from the micro-simulation analysis being conducted in Oregon. The study began February 2006 and ended May 2008.

To begin and frame this study a literature review was conducted, which totaled 23 manuals, reports, and articles. The list for the literature review came from searches from TPAU, as well as the State of Oregon Library which conducted searches to help form a comprehensive list of all existing research. The literature review contained calibration and validation processes for varying types of microsimulation applications, across a wide variety of software and methodologies. The review did not produce a clear set of steps, parameters, or measures of effectiveness to base this study on. However, the review did reinforce SimTraffic's calibration help documents which stated that the headway, turning speeds, and driver reaction time may need to be adjusted to achieve calibration.

The goal of this study was to create a set of simulation guidelines that would be independent of the software being used. However, the literature reviewed indicated that different process would have to be employed depending on the software being used. As part of this study, a list of Microsimulation software to be evaluated was developed; SimTraffic, VISSIM, PARAMICS, and CORSIM. However, after initial tests with calibrating these four software, it was determined that ODOT-TPAU did not have the time or budget to complete a study that would provide guidance on how to calibrate a Microsimulation across all software platforms. SimTraffic was used for this study for three reasons:

1. The ODOT-TPAU staff had been using SimTraffic for many years and was very familiar with the software. For this reason SimTraffic did not have the learning curve that some of the software had.
2. VISSIM and PARAMICS both offered dynamic assignment. After the literature review and testing the software it was determined that this feature was above and beyond the goals of this study. The literature reviewed suggested that it was harder to calibrate a model using dynamic assignment. In addition, most of the Microsimulation analysis performed by ODOT-TPAU, is at the corridor or small network level. Dynamic assignment is not as beneficial for smaller networks or corridors. The real benefit with dynamic assignment, and the primary use found in the literature, is for large congested networks; multiple parallel corridors, freeways with multiple access points being modeled

and a surrounding grid, or large downtown grids. Currently, ODOT does not deal with this scale of Microsimulation very often, although dynamic assignment will likely be more important for Oregon in the future and should be reevaluated then.

3. Both the literature review and the experience of ODOT-TPAU agreed that SimTraffic is closer to calibration "out-of-the-box" then the other software available at the time of this study and would therefore greatly simplify this work.

For these reasons only SimTraffic was used and, consequently, the guidelines are primarily for calibrating a network coded in SimTraffic*[*]. However, it is the hope of ODOT-TPAU that the guidelines for SimTraffic will be transferable to other software, or at least a good starting point in achieving calibration under other software.

After refining the study based on the literature review and what was available using SimTraffic, the following calibration parameters (independent variables, x) and measures of effectiveness (dependent variables, y) were used to develop the simulation guidelines:

Calibration Parameters

- X_1) Headway Factor
- X_2) 85^{th} Percentile Speed
- X_3) Driver Reaction Time

Measures of Effectiveness

- Y_1) Maximum Queue Length
- Y_2) Average Queue Length
- Y_3) 95^{th} Percentile Queue Length
- Y_4) Travel Time (Average Speed)
- Y_5) Vehicles Exited
- Y_6) Total Stops
- Y_7) Average Cycle Length

Site Selection

This study was scoped to test areas that represented projects from all areas of Oregon. The team wanted to ensure that results from this study could be used for all areas and projects across Oregon, not just the Willamette Valley (the majority of the population). The team picked eight locations that provided even coverage of the following project/area characteristics (note, more than eight locations was desired, however budget and time limitations only allowed for eight):

- Population – small urban areas, small MPO, medium MPO, large MPO (METRO)
- Project type – Expressway, Major arterial, pre-timed downtown grid
- Access Issues – Little to no access restriction to full access restriction
- Trip/Area Type – Commuter route, recreational route, urban area

[*] This study began using SimTraffic 6 (Build 612). During the course of this work SimTraffic 7 was released by Trafficware. The work was checked and completed using build 761.

All eight locations represented typical locations where a project would be needed or studied. Although, to keep control on the project, all study areas were kept fairly small, the largest area encompassing seven intersections. Larger networks would require data collection resources beyond the capability of this study and could have introduced extra noise to the calibration work making more difficult to draw conclusions. Larger networks may be addressed in future studies that will build off of the work conducted for this study.

Summary points for the eight locations chosen to meet the above criteria are provided here:

Albany, Oregon – US20, collections made July 25th & September 19th, 2006, and July 24th, 2007

- Small urban area
- Five-lane arterial
- Access-controlled
- Urban area
- Two signalized intersections
    - Spring Hill Drive
    - North Albany Road

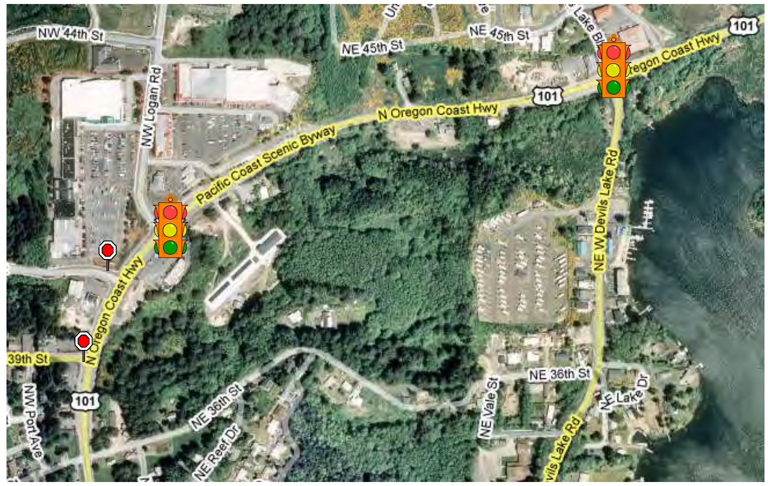
Notes:
Wide turns required turning speeds to be collected and input.

Bend, Oregon – US97, collections made July 19th, 2007 and November 11th, 2007

- Small MPO
- Five-lane-arterial – planned freeway
- Partial Access control
- Recreational area / Commuter route
- Two signalized intersections
    - Cooley Road
    - Robal Road
- Five unsignalized intersections/accesses
    - Clausen Drive
    - Lowe's Driveway
    - Chavre Way
    - Target Driveway
    - Nels Anderson Place

Notes:
This was a subsection of a much larger network for an active project for US97 in the northern end of Bend. There were many challenges (simulation run times, sparse field data for calibration, complexity incomparable to other locations) with using the full model area for the calibration testing.

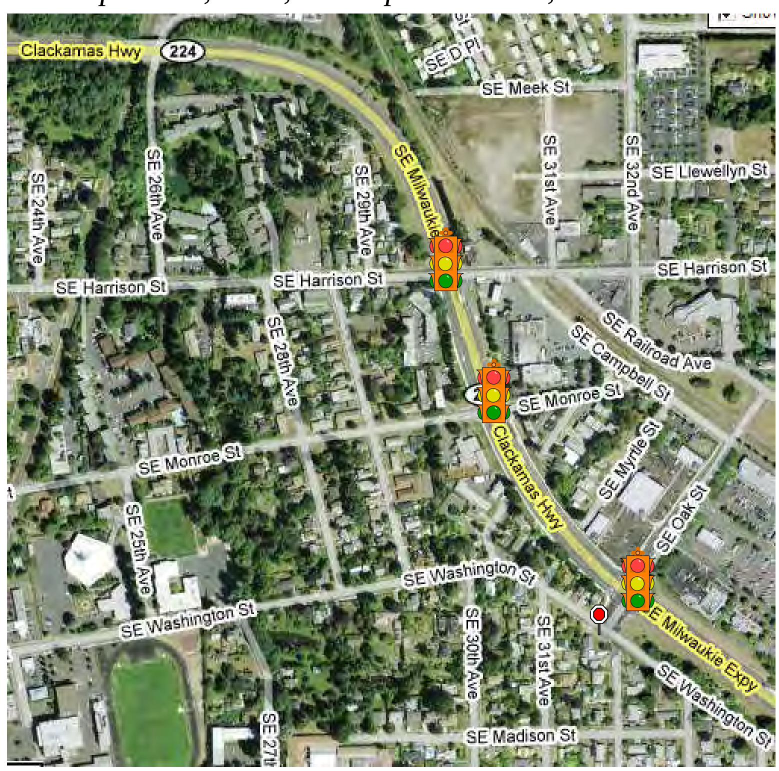
Lincoln City, Oregon – US101, collections made August 17th, September 11th & 28th, 2007



- Small urban area
- Two/Five-lane arterial
- Partial Access control
- Recreational area
- Two signalized intersections
  - West Devils Lake Road
  - Logan Road
- Two unsignalized intersections
  - 40th Street
  - 39th Street

Notes:
Wide turns required turning speeds to be collected and input. Also, short turn bays required special treatment with positioning distances.

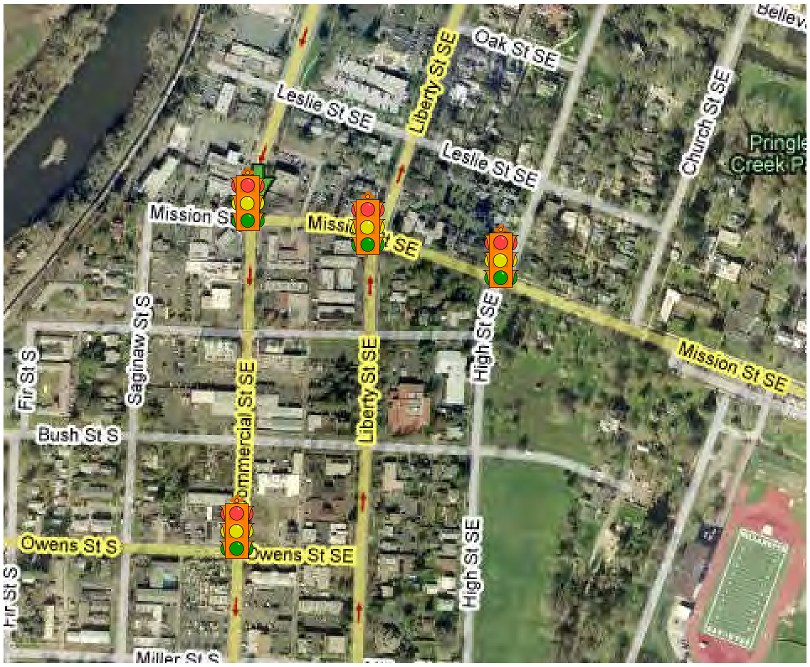
Milwaukie, Oregon – OR224, Clackamas Hwy, collections made October 12th, 2006, April 24th, 2007, and September 25th, 2007



- Large MPO (METRO)
- Five-lane expressway
- Access-controlled
- Urban commuter route
- Three signalized intersections
  - Harrison Street
  - Monroe Street
  - Oak Street
- One unsignalized intersection
  - Washington Street

Notes:
Due to the heavy turn moves during the peak hour, turn bay lengths were essential. With SimTraffic 7, taper lengths made a significant difference. Also, the <100 ft link length on Oak Street between OR224 and Washington Street caused problems with the turn moves on to Oak Street. Make note that signing near signals may cause improper behavior/operation at the signal. In this case, a yield sign on Oak Street in the simulation caused left turning vehicles to stop and check for clearance on their green, behavior not seen in the field. The yield sign was originally placed in the network when the network was in SYNCHRO 6 to remove improper long queues on Washington Street. However, in 7 the changes in behavior logic, removed the need for the yield sign and created a situation where its presence caused improper behavior at the intersection.

Salem, Oregon – Mission Street, collections made October 24th, 2006 and June 26th, 2007



- Medium MPO
- One-way grid
- Unrestricted access
- Urban commuter route
- Four signalized intersections
    - o Commercial Street
    - o Liberty Street
    - o High Street
    - o Owens Street

Notes:
Network performed better than witnessed in the field. Special attention needed to be given to the OD paths, specifically the heavy move from Mission Street to Commercial Street to Owens Street. This helped better model the congestion levels witnessed.

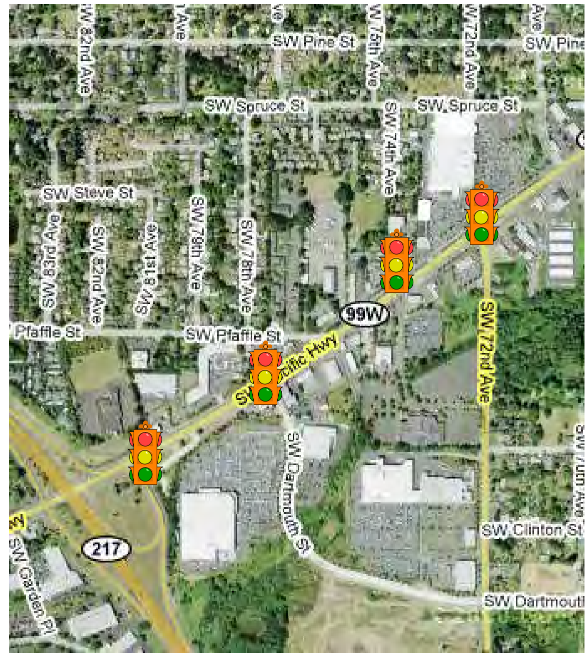
Salem, Oregon – OR22, collections made October 17th, 2006, May 15th and September 18th, 2007

- Medium MPO
- Expressway
- Access-controlled
- Urban commuter route
- Two signalized intersections
    - o Airport Road
    - o Hawthorne Avenue

Notes:
Driver expectance in this area did not follow the posted speed limit. The free flow speed was found to be less than the posted speed limit.

Tigard, Oregon – OR99W, collections made November 8th, 2006, May 8th and September 25th, 2007

- Large MPO (METRO)
- Five-lane arterial corridor
- Partial access control
- Urban Commuter Route
- Four signalized intersections
    - Northbound OR217 ramp terminal
    - Dartmouth Street
    - Theater access
    - 72nd Avenue



Notes:
~5% grade along OR99W at 72nd Street. Saturation flow measurements were taken for both the up and downgrade. After the ideal saturation flow was back-calculated both the up and downgrade rates were found to be very close (1683 and 1738 pcphpl, respectively). This finding helped to validate both the Highway Capacity Manual (HCM) saturation flow collection methodology and the field collection practices used for this study (the data was collected during the same time period by two different analysts).

Woodburn, Oregon – OR99E, collections made June 27th, 2006 and September 26th, 2006

- Small urban area
- Five-lane arterial
- Unrestricted access
- Urban area
- Two signalized intersections
    - Hardcastle Street
    - Lincoln Street

Notes:
This was the first collection site. The methodology was modified for the future sections based on this initial collection. Even with the change in methodology, there was not enough variation to warrant recollecting the data.

Data Collection

For this study, data that would be required for a project simulation analysis needed to be collected for each of the eight locations. Above and beyond that, new calibration data needed to be collected. Since the study would determine which calibration data would be important, many additional or to-be-determined unnecessary data also needed to be collected in order to get to the set of calibration data that ODOT would recommend or require be collected for simulation analysis. This created an extensive list of data to collect for each site, which is part of the reason why the test locations had to be limited to only a handful of intersections. The following is the list of data that was required at all eight locations:

- Roadway geometrics (Turn bay and taper lengths were critical)
- Classified vehicle counts
- Signal timing, phasing details, coordination
- Saturation flow measurements (following HCM methodology)
- Driver reaction time
- Queued vehicles (stopped counts)
- Average speed (using floating car measurements)
- Free-flow speed (85th percentile measurements)
- Turning speeds (using either a probe vehicle in traffic or using LIDAR measurements)
- Lane utilization

The data currently required for projects (geometries, vehicle counts, Free-flow speed, turning speeds, signal timing) have tested processes for collection and was fairly straight forward. The additional detail required for the calibration work (saturation flow, driver reaction time, queued counts, average speed) was not as familiar to TPAU and posed the threat of being beyond the collection resources available for this study. Many of these measurements would typically be carried out by a team of two, one person to view traffic and one person to record the measurements. Another option would be to set up video cameras with time stamps and go back after the fact and record the data. To compound the problem, this study involved visiting and collecting data at each location a minimum of two times.

> *Part of the study was scoped to determine the importance of the calibration data being collected on the count day(s), or if there was a window of time around count days where the calibration data would be acceptable. It is anticipated that project schedules and resources will likely not allow for all of the vehicle counts and the calibration counts to occur on the same day.*

The large data collection requirements for this study created concern on the feasibility of this project with the resources available. Fortunately, a process using JAMAR counters was agreed on and developed during the planning stages of this project. The more complex JAMAR counters offer time stamp functionality, where the time and button number is marked each time a button pressed. For these time stamp units, JAMAR offers a methodology to collect saturation flow rates using the JAMAR units and their software, PETRA. TPAU did not have a current license of PETRA, in addition, TPAU saw the ability to collect additional data with JAMAR units during saturation flow collection. During the planning stages of the project TPAU wrote customized software in R that read and interpreted the text files reported from the JAMAR units.

Using the JAMAR counters allowed a single analyst to collect the following data for an approach all at the same time:

- Saturation flow rates
- Queued vehicle counts by cycle and period
- Driver reaction time
- Lost time for the approach
- Heavy truck counts (percentages for ideal saturation flow rate)
- Turning vehicle counts (percentages for ideal saturation flow rate)
- Vehicle counts
- Lane utilization
- Phasing detail, signal operation by cycle
- Arrival type (rating) for the approach

The JAMAR units made it possible to collect all the necessary data with the resources and budget available. TPAU has written up the process and instructions for how to use the JAMAR units as they were used for this study. On request, TPAU will provide instructions and the software.

Calibration Testing Methodology

Step 1

After all of the data was collected, the first step was to build a SimTraffic model for each location following all the recommendations currently in APM. These SimTraffic models served as the reference or base case for the calibration testing. To-date the “visual calibration” from the APM served as an acceptable level of calibration, the goal was to improve the calibration above and beyond this minimum level of prep-work. The “visual calibration” included measuring, adjusting, and fine tuning the following inputs:

- Reviewing and fixing all Error and Warning Reports
- Setting up the Seeding and Recording
- Setting the Random Seed Number to zero
- Vehicle composition (lengths and percentages)
- Turning speeds for irregular turns
- Adding full signal detail, including detection and detector spacing
- Proper geometry including turn bay and taper lengths
- Observed driver behavior
    - Lane (turning) alignment
    - Blocking intersections
    - Improperly using the shoulder or median
    - Positioning lengths
    - OD paths for major moves

After all of these were coded and a visual calibration or "laugh test" was performed to make sure that the microsimulation resembled the traffic conditions witnessed in the field. Each location required varying amounts visual calibration time and data collection, which was dependent on size and issues that were unique for every location.

Part of this study included developing a "Simulation Field Inventory Worksheet" for microsimulation work. This worksheet can be requested from TPAU. TPAU recommends that field collection / observation be made as close to the count date, or if there are multiple count dates, as close to the 30th highest hour as possible. The analyst being on site at the time of the count (or a representative day) will help ensure that the volume coded into the model from the counts (or adjusted counts as the case will probably be) can physically make it through the network. Most areas under analysis are near congestion. When counts are adjusted up, it is possible to create an input volume that is greater than the capacity at the intersection. For this reason, it is important to witness the driver behavior/movement during the count and noting where problems do and do not occur and to verify that volumes to be input into the model are within the capacity of all given intersections. If the analysts visits the site off of the count day, it is advised that a short (peak hour) count be performed at an important (or group of important) intersection or major approach. This will help ensure that any adjusted counts are in line with what's actually occurring on the day the site is visited.

Step 2

The next step was to review, clean, tabulate, and analyze all of the calibration data collected. For each location the measures of effectiveness (independent variables listed in the introduction) were tabulated by 15 minute collection periods for each location, and put into a text format identical to the text report created by SimTraffic. This process allowed for easy comparison between field conditions observed and model conditions. To help automate this comparison, custom software written in R was used to quickly compare, summarize, report, and plot the comparison of the thousands of measurements.

The use and application of the calibration parameters (dependent variables) was not as straight forward; each of the three variables had a different collection, tabulation, and application process applied. The three are described individually here:

Headway Factor (X_1) – TPAU had four questions about headway (saturation flow) to be answered by this study:

1. If the collection day had to be on the count day.
2. If the collection had to be within the peak hour or if it could shoulder on the surrounding hours, if 15 cycles (as required by the HCM) could not be collected within the peak.
3. In some small urban areas it may not be possible to get 15 cycles with eight or more queued vehicles (as required by the HCM) within the peak hour for a complete sat flow count. TPAU wanted to know how using cycles with less than eight queued vehicles affected the calibration. Headway factors were calculated using five or more queued vehicles, in addition to the eight or more required by HCM.

4. The HCM states to take an average of the 15 or more cycles to determine the field saturation rate. TPAU also calculated this measure using the median instead of the average, which removed the lower outliers, raising the saturation flow. TPAU wanted to know how using the headway factor calculated from the median sat flow would affect calibration.

These four "on-off" type questions created a factor of 16 extra tests to the existing four required tests for a full factorial design (discussed in *Step 3* of calibration testing), equaling a total of 64 possible calibration tests to be performed for each location. This type of testing was not possible for study.

In an attempt to still get at some of these questions and keep the study within reasonable bounds, it was decided to test the minimum headway factor and maximum headway factor found from these 16 cases, in addition to testing with and without the "standard or ideal" headway factor, which was collected on the count day, during the peak hour, using the average of 15 or more cycles which had eight or more queued vehicles. This created a total of 16 (8 x 2) calibration cases to test for each of the eight locations, which was a reasonable number to test.

85th Percentile Speed (X_2) – Unlike headway factor which was desired to be collected during the peak hour on the count day, the 85th percentile speed needed to be free flow, without congestion. That meant that it would not be collected during the peak and would ideally be the average over multiple days to ensure a true desired free flow speed. This measure did not have the same variability and questions regarding collection that headway factor posed. Therefore, the 85th Percentile Speed was simple on or off test during the calibration testing.

Driver Reaction Time (X_3) – Initially TPAU planned to collect and test driver reaction time for each location (meaning that driver reaction time would be area dependent). However, after the data was collected and tabulated, it was clear that there was far too much variability in driver reaction time from analyst and intersection even in the same area and collection period to recommend this as practice. However, what TPAU did find, was that over all locations, population sizes, and area types, the driver reaction time for Oregon drivers was fairly constant, and, more importantly, the collected times were greatly different than SimTraffic's defaults. In addition, SimTraffic recommended that their defaults be changed to match those collected. For these reasons, TPAU tested the benefit of using a statewide average not a location based driver reaction time.

SimTraffic Driver Reaction	State Wide Average Driver Reaction	State Wide Standard Dev. Driver Reaction
0.8	2.4	0.7
0.7	1.6	0.3
0.6	1.3	0.3
0.6	1.1	0.3
0.5	1.0	0.3
0.5	0.9	0.2
0.5	0.8	0.2
0.4	0.8	0.2
0.3	0.7	0.2
0.2	0.5	0.2

Step 3

The final step was to run a series of 16 tests on each of the eight locations. The tests represented the 16 unique combinations of applying the three calibration parameters to the base or reference, "visually calibrated" network. As touched on in the discussion of the development of the calibration parameters (*Step 2*), there were four possible settings for headway factor and two possible settings (on or off) for 85th percentile speed and driver reaction time, creating a 4 x 2^2 factorial design (16 cases). The factorial design was applied as follows:

	X_1	X_2	X_3
Case 1 (Base)	-1	-1	-1
Case 2	1	-1	-1
Case 3	-1	1	-1
Case 4	1	1	-1
Case 5	-1	-1	1
Case 6	1	-1	1
Case 7	-1	1	1
Case 8	1	1	1
Case 9	Min	-1	-1
Case 10	Min	1	-1
Case 11	Min	-1	1
Case 12	Min	1	1
Case 13	Max	-1	-1
Case 14	Max	1	-1
Case 15	Max	-1	1
Case 16	Max	1	1

In this table (-1) and (1) represent off (default) and on (field measurement used), respectively. Each case has a unique set of inputs and by testing all cases the goal was to determine the importance of each input. For cases (models) where the headway factor was to be applied the headway factor was collected and calculated at the entrance approaches for the major arterial being studied, creating two points. The headway factors at the entrance points were assumed to carry through the study area. Example, if the major route was East-West, a WB headway factor would be collected at the intersection farthest east and a EB headway factor would be collected at the intersection farthest west. The EB and WB headway factor inputs would be adjusted to the calculated value at every intersection along the major route for any case that required that a measured headway factor be used (note, there were 16 different ways that the headway factors were calculated, three measurements were applied in the testing; the ideal (1), min, and max).

For cases where the 85th percentile speed was applied, the measured speed was used in place of the posted speed. The speed was measured along the major route and as close the study area as possible. For most of the locations, measurements could not be made within the study area since the close intersections did not allow for free flow speeds to be reached, or the area was not safe enough to setup for speed collection. If the measurement could not be made in the study area, it was made immediately outside the area on a representative section. The side streets were always assumed at the posted speed.

For cases where the driver reaction time was applied the default SimTraffic driver reaction time (Green React) by drive type was overwritten by the driver reaction times averaged over the 31 distributions collected from this study for ODOT (note the highest reaction time, 2.4 – driver type 1, was limited to 2.0 by the SimTraffic software).

SimTraffic Parameters

Vehicles | Drivers | Intervals | Data Options

Driver Types	1	2	3	4	5	6	7	8	9	10
Yellow Decel (ft/s^2)	12.0	12.0	12.0	12.0	12.0	11.0	10.0	9.0	8.0	7.0
Speed Factor (%)	0.85	0.88	0.92	0.95	0.98	1.02	1.05	1.08	1.12	1.15
Courtesy Decel (ft/s^2)	10.0	9.0	8.0	7.0	6.0	5.0	4.0	4.0	3.0	3.0
Yellow React (s)	0.7	0.9	1.0	1.0	1.2	1.3	1.3	1.4	1.4	1.7
Green React (s)	2.0	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.5
Headway @ 0 mph (s)	0.65	0.63	0.60	0.58	0.55	0.45	0.42	0.40	0.37	0.35
Headway @ 20 mph (s)	1.80	1.70	1.60	1.50	1.40	1.20	1.10	1.00	0.90	0.80
Headway @ 50 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Headway @ 80 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Gap Acceptance Factor	1.15	1.12	1.10	1.05	1.00	1.00	0.95	0.90	0.88	0.85
Positioning Advantage (veh)	15.0	15.0	15.0	15.0	15.0	2.0	2.0	2.0	1.2	1.2
Optional Advantage (veh)	2.3	2.3	2.3	1.0	1.0	1.0	1.0	1.0	0.5	0.5
Mandatory Dist Adj (%)	200	170	150	135	110	90	80	70	60	50
Positioning Dist Adj (%)	150	140	130	120	110	95	90	80	70	60

OK | Cancel | Default | Vehicle and Driver Parameters

Reaction time at start of green (s)

Prior to this study, the APM required that every microsimulation be run a minimum of five times so that a "true" average could be analyzed, helping to remove the effect of uncommon random effects in traffic arrival or patterns. At the beginning of this study TPAU recognized that, there can be significant variation in MOEs between each SimTraffic run. Because of this each of the 16 cases was run to the point where the MOEs produced a "static" average value. The following equation was used to determine how many runs were required for each case,

$$\boxed{n = t^2\sigma^2 \big/ (error)^2}$$

For this study, a confidence interval of 90% was used. Each MOE's standard deviation and mean for the model area were measured and put into this equation to determine how many runs were required to achieve a model wide mean with 90% confidence. The study found that the number of runs required varied greatly depending on which measure was to be used. Queue lengths could require ~50 runs for some of the larger more congested networks (Bend, Milwaukie). For the smaller less congested networks (Woodburn, Albany), the standard 5 runs allowed all of the MOE's to be within a 90% confidence interval. After all the data had been tested an analyzed "Vehicles Exited" was the only MOE determined to be acceptable to recommend to be used to determine calibration. "Vehicles Exited" is a stable measure and never required more than five runs. It is for this reason that the APM will continue to recommend a minimum of five runs, however future work will have to fully investigate the sensitivity of queue lengths and how many runs is required to achieve a stable measure. Queue lengths are the primary reason to go to microsimulation so it is very important that they are reported correctly.

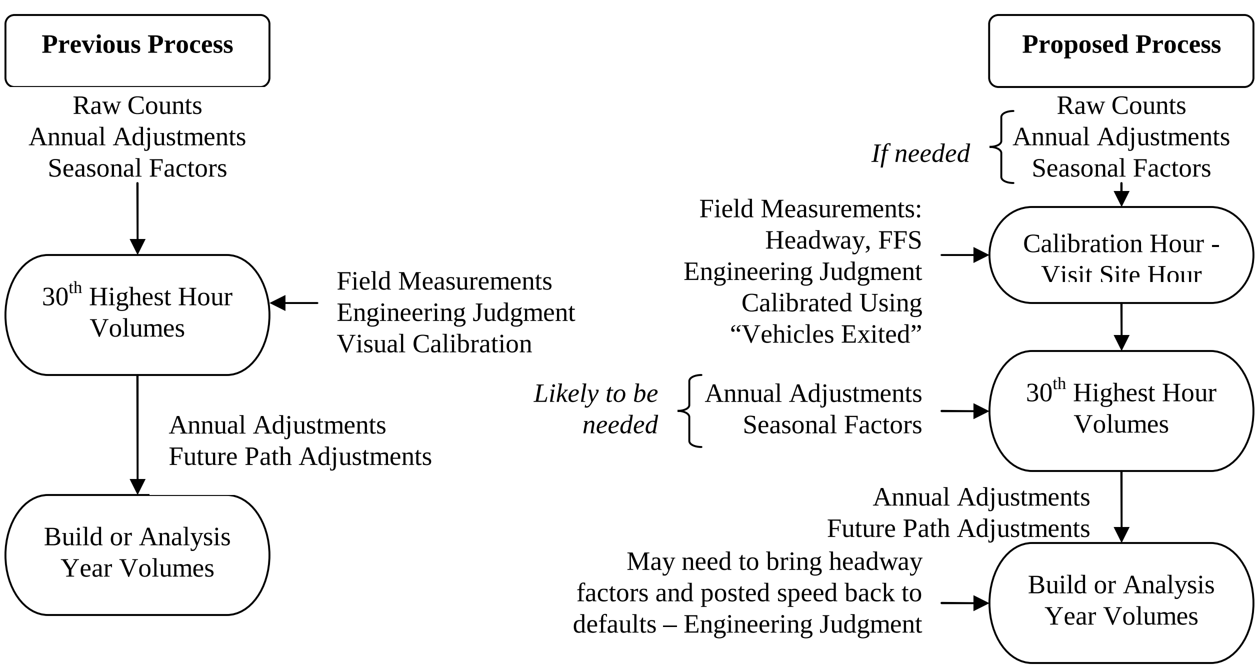
Conclusions / Proposed APM Changes

The work from this study improves on the current microsimulation process outlined by the APM. However, this work was not fully conclusive. From the comparison of the eight test sites, no clear combination of headway factors, 85th percentile speed, or driver reaction time, was found to improve the performance across all of the simulations and performance measures. This was also true for comparing test sites by their area characteristics; again no consistent combination of factors to achieve calibration could be found.

Consistent with the literature review conducted for this study, the APM should recommend that engineering judgment be used to correctly apply headway factors (HF) (saturation flow rates) and free-flow Speeds (85th percentile) to achieve an acceptable calibration. As indicated in Trafficware's Manual, free-flow speeds applies to turning speeds as well as corridor speeds.

That being the case, the ODOT driver reaction time should be used, as it was found that the SimTraffic default is grossly different than Oregon driver reaction times. In addition, Trafficware recommends that driver reaction times be adjusted as one of the three major parameters to achieve calibration. The ODOT driver reaction time will continue to be updated and refined as more data is collected and should be used for all projects on Oregon highways using microsimulation.

The major change to the prior APM methodology will be additional "network" or "build" that will need to be constructed for any given microsimulation project. Previously, the APM instructed users to build a "base year network" with 30th highest hour volumes. Then from that a future year (build year) network was saved with volumes grown from the 30th highest hour to the future year. The new process, which will include this calibration work, will be almost the same, except it will we require one more volume set for calibration. The following flowcharts illustrate the change:



The additional step in the "proposed process" would add an additional volume set during the base year development. To a point this work is already being done in the "previous process", however it is not formally documented and saved in a standard manner, as proposed. The proposed method, would adjust the raw counts up to the day that the area was visited by the analyst, who would then be able to verify the performance of the study area. From this "visit site hour", the visual calibration and "true" calibration, using "vehicles exited" could be completed. To reach "true" calibration, the analyst could use methods from the visual calibration or by incorporating field measured headway factors or 85th percentile speed, determined from the analyst's engineering judgment.

> *The study found that, although saturation flow rates can vary greatly from day-to-day (+200 vehicles per hour difference on different days), an area's saturation flow rates tend to average with a reasonable standard deviation over multiple days, under similar levels of congestion. If the saturation flow count can not be performed on the count / visit day, it is advised that the 15 or more cycles be collected during the peak hour over several days, to establish an average that the analyst has confidence in.*
>
> *Further, this study found that there was no benefit in deviating from the HCM's averaging the saturation flow counts (using median in place of average). In addition, the study findings agreed with the HCM when cycles with less than 8 queued vehicles are used. The findings showed that using cycles with less than 8 queued vehicles will decrease the saturation flow. Therefore, as the HCM states, only cycles with a queue of 8 or more vehicles may be used as part of the 15 or more cycles required to average the saturation flow rate for an approach.*
>
> *If an analyst feels that headway factors or 85th percentile speed would improve the calibration but the timeline, scale of project, and/or budget do not allow for field measurements, the following ranges (found from this study) can be applied in place of the SimTraffic/SYNCHRO defaults.*
>
> > *85th percentile speed – the average found was ~5 mph higher than the posted*
> >
> > *Headway factors – Using the APM defaults of 1900 veh/hour for METRO, Salem, and Eugene and 1750 for all other areas gives headway factors of 0.98 and 1.10, respectively.*

The "true" calibration is to be determined with the use of the MOE, "Vehicles Exited". The visual calibration can be considered as approaching calibration, or adding accuracy to the calibration. Using the quantitative measure of "Vehicles Exited" adds reassurance and precision to the calibration, quantitatively reaffirming that the number of vehicles seen passing through the system matches the number of vehicles passing through the model, and that there are no trouble spots where vehicles are incorrectly queuing or being blocked. "Vehicles Exited" represents the number of vehicles that make it through an intersection over a given period of time. This should equal the volume coded in the network for the "visit site hour". A tolerance of 1% for each intersection over the analysis period (hour) will be required to achieve calibration for the calibration volume set (not required for the 30th highest hour or build year network). The tolerances for each movement may vary depending on volume, but any movement over 100 vehicles/hour should be within 5% of the coded volume. Movements with less than 100 vehicles/hour should be checked to make sure that the vehicles exiting is reasonable. One of the extra benefits to using "Vehicles Exited" is that this is an automated report from SimTraffic and does not require any external software or manipulation of the data.

1001: Route 20 & Spring Hill Drive Performance by movement Entire Run

Movement	EBL	EBT	WBT	WBR	SBL	SBR	All
Total Stops	38	353	704	25	298	14	1432
Avg Speed (mph)	12	26	16	23	9	14	19
Vehicles Entered	39	1161	1198	476	317	19	3210
Vehicles Exited	39	1164	1200	476	320	19	3218
Hourly Exit Rate	39	1164	1200	476	320	19	3218
Input Volume	38	1169	1187	489	311	17	3211
% of Volume	103	100	101	97	103	112	100

Note that SimTraffic distributes vehicles to the network randomly based on coded volumes. Seeding offsets and random occurrence can cause the exiting volume to be greater than the coded volume. This is usually negligible, although if an analyst sees a consistent over assignment at a location further investigation may be necessary. The primary purpose for reviewing "Vehicles Exited" is to ensure that the number of vehicles exiting the intersection is not greatly less (~5%) than the coded volume. Less volume can indicate an improper blockage, bottle network, or miscoding in the network, and needs to be investigated and corrected.

After the "visit site hour" volume set calibration has been established, the second base year volume set, the 30th highest hour, would be grown to the 30th highest hour volumes and re-balanced (if the sight visit was at the 30th hour, this set could be skipped). This volume set would likely show worse conditions than the calibrated state.

The last volume set would have the volumes grown, adjusted, and balanced to the build year or analysis year (Design Hour Volume – DHV). For the Future No Build, any headways used for calibration would typically be brought back to statewide defaults (averages) if the headways in the field were greater than the defaults, ie., as congestion increases over time, headways will approach statewide averages, all else remaining the same. If the field measured headways were found to be less than the defaults, they could be kept for the future scenarios.

This simulation guidelines work took two years to complete and involved members from all over ODOT, not just within TPAU. Many important insights and notes were collected from different people involved through out the course of this work. These notes will be come apart of the APM Chapter 8, but are best communicated here as informative bullets:

- The analyst must visit the location on or within a reasonable time period of the count collection, as the analyst will need to fully understand what is actually occurring during the peak so that they can verify that the simulation is within reasonable bounds, before any calibration is attempted.

- Phasing detail improves calibration in SimTraffic 7 (S7), however in some cases, a short min gap on turn bays or side streets causes the phase to gap out while a queue still exists. This can be found by visual inspection and should be corrected by adjusting the minimum gap to two seconds or to the vehicle extension, if the vehicle extension is less than two seconds. Also note, for S7, lost time must be adjusted and detectors must be added. The detector spacing defaults provided by TPAU are for state facilities. Side streets should be treated as left turn bays, with call detectors if better information is not available. Without call detection, the side streets can be skipped for many cycles. Actual detector spacing and settings should be used in place of defaults if they are available.

- Correct turn bay length and geometry is crucial. If users are "lengthening" the turn bays by illegally using shoulder or the median, this should be coded in the model.

- The first step before calibration is to perform a visual "laugh test", addressing any problems through application of engineering judgment, further field collection, or discussing issues with the local jurisdictions responsible for the area or side streets (turning speeds, sat flow studies, speed studies, geometry or detection issues, validating/rechecking signal phasing, pedestrians, blocked intersections, OD patterns, lane (turn) alignment, taper & turn bay lengths, positioning lengths…)

- All simulation runs need to be checked to make sure that a run didn't blow up or freeze up. If the network freezes or performs much worse than expected (use judgment), either adjustments to the network must be made, or if it is just for random runs, remove (rerun) the poor runs.

- In running all of the tests it was found that five runs was enough to ensure that "Vehicles Exited" reached a reasonable average value. Other measures like stops, speed and queues see a lot more variability and would require more than five runs and extra work to ensure that convergences had been reached, but using "Vehicles Exited" allows five runs to continue to be the standard. Future work will have to be done to determine how many runs are needed to ensure that queue lengths have reached a stable average.

Literature Reviewed

A Report on the Use of Traffic Simulation Models in the San Diego Region. Institute of Transportation Engineers – California Border Section and Highway Capacity Task Force, March 2004.

Bayarri, M. J., J. O. Berger, G. Molina, N. M. Rouphail, and J. Sacks. *Assessing Uncertainties in Traffic Simulation: A Key Component in Model Calibration and Validation.* National Institute of Statistical Sciences, Research Triangle Park, NC, July, 2003.

Benekohal, R. F., and G. Abu-Lebdeh. Variability Analysis of Traffic Simulation Outputs: Practical Approach for TRAF-NETSIM. In *Transportation Research Record 1457*, TRB, National Research Council, Washington, D.C., 1994, pp. 198-206.

Chu, L., H. X. Liu, J.-S. Oh, and W. Recker. *A Calibration Procedure for Microscopic Traffic Simulation*. Cambridge Systematics, Irvine, CA, June 2002

Chu, L., and H. X. Liu. Caltrans TMS Master Plan – Calibration Methodology and Results for the I-405/I-5 Simulation Model. In *TRB 2006 Annual Meeting CD-ROM*, TRB, National Research Council, Washington, D.C., 2006.

Ciuffo, B., V. Punzo, and V. Torrieri. A Comparison Between Simulation-Based and Model-Based Calibrations of Traffic Flow Micro-Simulation Models. In *TRB 2008 Annual Meeting CD-ROM*, TRB, National Research Council, Washington, D.C., 2008.

Dowling, R., J. Holland, and A. Huang. *California Department of Transportation: Guidelines for Applying Traffic Microsimulation Modeling Software.* Dowling Associates, Oakland, CA, September 2002.

Dowling, R., A. Skabardonis, J. Halkias, G. McHale, and G. Zammit. Guidelines for Calibration of Microsimulation Models. In *Transportation Research Record 1876*, TRB, National Research Council, Washington, D.C., 2004, pp. 1-9.

Hellinga, B. R. *Requirements for the Calibration of Traffic Simulation Models.* Department of Civil Engineering, University of Waterloo, Ontario, Canada, 1998.

Hollander, Y. and R. Liu. *The Principles of Calibrating Traffic Microsimulation Models.* In *Transportation Vol 35 Number 3*, Kluwer Academic Publishers, Dordrecht, Boston, January, 2008, pp. 347-362.

Hourdakis, J., P. G. Michalopoulos, and J. Kottommannil. Practical Procedure for Calibrating Microscopic Traffic Simulation Models. In *Transportation Research Record 1852*, TRB, National Research Council, Washington, D.C., 2003, pp. 130-139.

Jayakrishnan, R., J.-S. Oh, and A.-E. K. Sahraoui. *Calibration and Path Dynamics Issues in Microscopic Simulation for Advanced Traffic Management and Information Systems.* Institute of Transportation Studies, University of California, Irvine, CA, December, 2000.

Jones, S. L., A. J. Sullivan, M. D. Anderson, and D. Malave. *Traffic Simulation Software Comparison Study.* University Transportation Center for Alabama, June, 2004.

Liu, H. X., L. Ding, J. X. Ban, A. Chen, and P. Chootinan. A Streamlined Network Calibration Procedure for California SR41 Corridor Traffic Simulation Study. In *TRB 2006 Annual Meeting CD-ROM*, TRB, National Research Council, Washington, D.C., 2006.

Long, G. A Unified Theory of Saturation Flow. In *TRB 2006 Annual Meeting CD-ROM*, TRB, National Research Council, Washington, D.C., 2006.

Park, B. and J. D. Schneeberger. Microscopic Simulation Model Calibration and Validation. In *Transportation Research Record 1856*, TRB, National Research Council, Washington, D.C., 2003, pp. 185-192.

Park, B., J. Won, and I. Yun. Application of Microscopic Simulation Model Calibration and Validation Procedure: Case Study of Coordinated Actuated Signal System. In *Transportation Research Record 1978*, TRB, National Research Council, Washington, D.C., 2006, pp. 113-122.

Pennsylvania Avenue Traffic Alternatives Analysis – Final Report. Parsons Transportation Group, P.C., October 2001.

Shaaban, K. S. and E. Radwan. A Calibration and Validation Procedure for Microscopic Simulation Model: A Case Study of SimTraffic for Arterial Streets. In *TRB 2005 Annual Meeting CD-ROM*, TRB, National Research Council, Washington, D.C., 2005.

Tian, Z. Z., T. Urbanik II, R. Engelbrecht, and K. Balke. Variations in Capacity and Delay Estimates from Microscopic Traffic Simulation Models. In *Transportation Research Record 1802*, TRB, National Research Council, Washington, D.C., 2002.

Traffic Analysis Toolbox Volume III: Guidelines for Applying Traffic Microsimulation Modeling Software. FHWA, U.S Department of Transportation, July 2004.

Traffic Analysis Toolbox Volume IV: Guidelines for Applying CORSIM Microsimulation Modeling Software. FHWA, U.S Department of Transportation, January 2007.

Wan, B., N. Rouphail, and J. Sacks. *Failure Detection and Diagnosis in Micro-Simulation Traffic Models.* National Institute of Statistical Sciences, Research Triangle Park, NC, July, 2005.