Development of a Model to Predict Proportion of Vehicles Entering Merge Influence Areas at Freeway Ramp Junctions

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Abstract

Vehicles entering on-ramps or exiting off-ramps on freeway create turbulence to freeway traffic. The Level of Service (LOS) in a merge and diverge an influence area is determined by the density of traffic on influence area. To estimate the density of traffic on an influence area, flows entering the influence area are required. The Highway Capacity Manual by the Transportation research board has developed models to predict flow entering an influence area. However, these models were developed based on the driving behavior of people in the USA which may differ from those in Thailand.

In this study, the statistical comparison between the proportion of volume in lane 1 and 2 (P_{FM}) according to the Highway Capacity Manual's model and from field data shows significant differences for both test sites; Interstate 25 in USA and Srirath Expressway in Thailand. It was found that average time headway time affects the P_{FM} values. The proposed regression model was developed on the first data set from a section of Srirath Expressway and tested on the second data set. The model proposed fitted well with Srirath Expressway data. There is no significant differences between P_{FM} estimated from the proposed model and filed data at the 95 percent confidence level.

Keywords: Level of Service, Ramp Junction, Merge Influence Area

1. Introduction

Turbulence occurs at ramp junctions caused by vehicles entering on-ramps and/or exiting off-ramps. The Level of Service (LOS) in a merge and diverge influence area is determined by density of traffic on the influence area. [Flow entering an influence area is required, to estimate the density of traffic in the influence area.] The Highway Capacity Manual by Transportation research board (TRB) [1] has developed models to predict flow entering an influence area. However, these models were developed based on driving behavior of people in the USA which may be different from Thailand. Examination of the model developed by TRB with data collected from a freeway in Thailand is necessary. In order to obtain a model that suits local driving behavior, the proposed model to predict the proportion of vehicles entering a merging influence area, based on data collected from a freeway in Thailand, should be developed.

2. Objectives

The objectives of this study are to:

1) Examine the model developed by the Transportation Research Board.
2) Develop a model to predict the proportion of volume entering an influence area based on data collected in Thailand.
3) Test the model developed with data collected in Thailand.

3. Literature Review

Several research groups have investigated freeway merge areas or freeway on-ramp junctions. However, a model to predict proportions of vehicles entering merge influence can only be found in HCM2000 [1]. This section reviews studies of ramps and ramp junctions and then presents the models to predict proportions of flow entering merge influence areas according to HCM2000.

Evans [2] has developed a methodology for the prediction of breakdown at freeway merges using Markov chains and probability of the time of breakdown. As expected, the results show that higher arrival rates, provide higher probability of breakdown. Bunker [3] presents the development and application of a limited priority gap acceptance model to freeway merging. The model developed provides a prediction of minor stream delays. Cassidy [4] presents that by metering, an on-ramp can recover the higher discharge flow at a merge, and increase merge capacity. Laval [5] presents that lane-changing vehicles create voids in traffic streams and that these voids reduce flow rate.

Flows entering ramp junctions on freeways, both at on-ramps and off-ramps cause turbulence to traffic from mainline. The proportion of flows entering an influence area is one of variables affecting the level of service. The Transportation research board (HCM2000) developed models to predict the proportion of flow entering an influence area. This model was developed based on data collected from freeways in the USA. A merge influence area is where main traffic is disturbed by on-ramp traffic (Figure 1). According to the Highway Capacity Manual 2000, the merge influence area is in lane 1 and lane 2 at the junction. The level of service from with A to F of the influence area is classified by its density. The level of service with A is the best, and F is the worst. The level of service with F represents flow that is greater than the capacity. Table 1 shows the range density corresponding to each level of service.

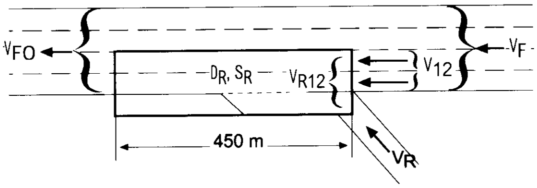

Figure 1. Merge Influence Area [1]

Table. 1 Level of Service of Ramp Junctions

Level of Service	Density (pc/km/ln)
A	<=6
B	>6-12
C	>12-17
D	>17-22
E	>22
F	>Capacity

The density at a merge influence area can be calculated as:

$$D_R=3.402+0.00456V_R+0.0048V_{12}-0.01278L_A \quad (1)$$

Where;

D_R = Density at merge influence area

V_R = on-ramp demand flow rate (pc/h)

V_{12}= flow rate in lanes 1 and 2 of freeway immediately upstream of merge area (pc/h)

L_A = length of acceleration lane (m)

The independent variables used to calculate the density include:

- Peak 15-minute on-ramp volume
- Flow entering lane 1 and 2 of a merge influence area
- Length of acceleration lane

Flows entering lane 1 and 2 of a merge influence area may be calculated where the variables which affect the proportion of vehicles are:

1. Total freeway flow approaching merge area (pc/h)
2. Total ramp flow (pc/h)
3. Length of acceleration lane (m)
4. Free flow speed of ramp at point of merge area (km/h)

Total flow on mainline is the variable that plays an important role on volume in an influence area. A longer length of acceleration lane will reduce the effect to mainline traffic. High on-ramp free flow speed would cause mainline traffic shift to lanes away from ramps.

Models to predict volume on lane 1 and 2 for a 3-lane freeway can be expressed as:

$$V_{12}=V_F*P_{FM} \quad (2)$$

Where;

$$P_{FM}=0.5775+0.000092L_A \quad (3)$$

$$P_{FM}=0.7289-0.0000135(V_F+V_R)-0.002048S_{FR}+0.0002L_{up} \quad (4)$$

$$P_{FM}=0.5487+0.0801V_D/L_{down} \quad (5)$$

Where;
V_{12}= flow rate in lanes 1 and 2 of freeway immediately upstream of merge area (pc/h)
V_F = freeway demand flow rate (pc/h)
P_{FM}= Proportion of flow in lanes 1 and 2
L_A = length of acceleration lane (m)
V_R = on-ramp demand flow rate(pc/h)
S_{FR} = free-flow speed of ramp (km/h)
L_{up} = distance to adjacent upstream ramp (m)
V_D = demand flow rate on adjacent downstream ramp (pc/h)
L_{down}=distance to adjacent downstream ramp(m)

Selection of Equations 3, 4, or 5 is described in Table 2. Equation 3 is selected where an adjacent ramp does not affect the subject ramp behavior. Equation 4 addresses cases with an adjacent upstream off-ramp and Equation 5 addresses cases with an adjacent downstream off-ramp.

The decision to use Equations 4 or 5 instead of 3 is made by determining the equilibrium separation distance. If the distance to an adjacent ramp is lower than the equilibrium separation distance, Equations 4 or 5 may be used.

Table. 2 Equations for P_{FM} for six-lane freeways

U/S ramp	Subject ramp	D/S ramp	Equation
No	On	No	3
No	On	On	3
N	On	Off	5 or 3
On	On	No	3
Off	On	No	4 or 3
On	On	On	3
On	On	Off	5 or 3
Off	On	On	4 or 3
Off	On	Off	5 , 4,or 3

U/S=Upstream, D/S=Downstream

In case a choice between Equation 3 and 4 must be made, equilibrium separation distance may be calculated as:

$$L_{EQ}=0.0675(V_F+V_R)+0.46L_A+10.24S_{FR}-757 \quad (6)$$

In case a choice between Equation 3 and 5 must be made, equilibrium separation distance may be calculated as:

$$L_{EQ}=V_D/(0.3596+0.001149L_A) \quad (7)$$

Where;
L_{EQ}= Equilibrium separation distance
V_F,V_R, L_A,S_{FR}, and V_D as previously defined.

4. Data Collection

The test links are a section of Interstate 25 (north bound direction) in Colorado, USA (Figure 2) and a section of Srirath Expressway (south bound direction) in Thailand (Figure 3). The section of Interstate 25 consists of three on-ramps: Colorado NW, Colorado NE, and Evans. The five-minute interval traffic volume data by lane collected from loop detectors on March 3, 1999 6:00 AM to 7:00 PM was obtained from the Colorado Department of Transportation (CDOT), USA. Another test link in Thailand consists of two on-ramps: Ratchadapisak and Phahonyothin. The 5-minute interval traffic volume data was collected during peak hours (3:00 PM to 6:00 PM) and off-peak hours (11:00 AM to 2:00 PM) on November 17 and 22, 2004.

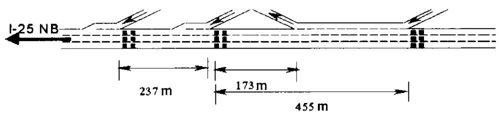

Figure. 2 Test Link at Interstate 25, USA

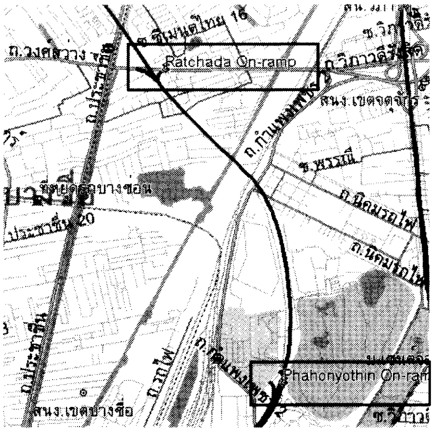

Figure. 3 Test link at Srirath Expressway, Thailand

5. Findings

According to the Highway Capacity Manual 2000 [1], Equation 3 may be used to calculate the proportion of vehicles entering lane 1 and 2 for all three on-ramps of Interstate 25 test link and both on-ramps of the Srirath expressway. The variable affecting the P_{FM} value is the length of the acceleration lane. The study compared the P_{FM} value from field data with that computed from Equation 3. The effect of headway time to P_{FM} value was also investigated and a new model was then developed.

5.1 Comparison of P_{FM} values between field data, and calculated according to HCM 2000

The proportion of vehicles entering an influence area collected from the field was compared with that calculated according to HCM 2000 using t-test. For all three locations on Interstate 25 in US; at Colorado NW, Colorado NE, and Evans, it was found that the P_{FM} values calculated according to HCM 2000 are significantly different from those collected from the field at the 95 percent confidence level. Similar results can also be found at two test locations on Srirath expressway in Thailand; Ratchadapisak (p-value < 0.0001) and Phahonyathin (p-value < 0.0001). Table. 3 and Table 4 show t-test results.

Figure 4 and Figure 5 show the plots between P_{FM} values calculated according to HCM2000 and those collected from field at various flow the rates. By inspection, the P_{FM} values according to HCM 2000 are clearly underestimated for all the test locations.

Table. 3 t-test to compare P_{FM} according to HCM 2000 and collected at Evans

	P_{FM}	$P_{FM\ (HCM2000)}$
Mean	0.61985542	0.59
Variance	0.000546029	0
Observations	140	140
Hypothesized Mean Difference	0	
df	139	
t Stat	15.11747917	
P(T<=t) one-tail	1.90962E-31	
t Critical one-tail	1.655889719	
P(T<=t) two-tail	3.81924E-31	
t Critical two-tail	1.977177817	

Table 4. t-test to compare P_{FM} according to HCM 2000 and collected at Ratchadapisak.

	P_{FM}	$P_{FM\ HCM}$
Mean	0.627226815	0.5821
Variance	0.000991531	3.17634E-30
Observations	152	152
Hypothesized Mean Difference	0	
df	151	
t Stat	17.6686472	
P(T<=t) one-tail	6.97565E-39	
t Critical one-tail	1.655007387	
P(T<=t) two-tail	1.39513E-38	
t Critical two-tail	1.97579889	

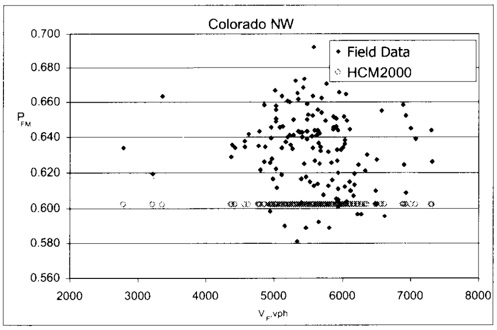

Figure. 4 P_{FM} Values calculated according to HCM 2000 and collected at Colorado NW, Interstate 25, USA.

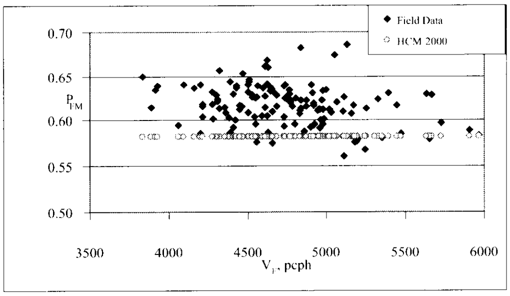

Figure. 5 P_{FM} Values calculated according to HCM 2000 and collected at Phahonyothin, Srirath Expressway, Thailand.

5.2 Examination of the factors affecting the P_{FM} values

As mentioned earlier, according to HCM 2000 Equation 3, the only factor that affects P_{FM} values is the length of the acceleration lane. This section examines other factors that may affect the P_{FM} values. Normally, when flow rate increases or headway time decreases, drivers tend to maneuver to the lane away from a ramp junction to avoid congestion at the junction. As a result, the P_{FM} values may decrease as the flow rate increases or headway time decreases. Figure 6 shows that as the average headway time decreases, the P_{FM} value decreases.

Table 5 shows the linear regression of the P_{FM} values at various values of average headway time. The hypothesis to test if average headway time affects the P_{FM} values is expressed as follows [6]:

$$H_0 : \beta_1 = 0 \; Versus \; H_A : \beta_1 \neq 0 \quad (8)$$

$$t_{statistic} = \frac{\beta_1}{s.e.(\beta_1)} = \frac{\beta_1 \sqrt{S_{xx}}}{\sigma} \quad (9)$$

$$= 6.47$$

$$t_{critical} = t_{0.025,300} = 2.25$$

Where; H_0 = Null hypothesis
H_A = Alternative hypothesis

Since the $|t|$ is greater than $t_{critical}$, the null hypothesis is rejected.

It was found that the slope of the linear regression differs significantly from zero at the 95 percent confidence level. The average headway time has significant affect on P_{FM} values. Similar analysis was preformed for the flow rate. It was also found that the flow rate has significant affect on P_{FM} values ($t_{statistic}$ = -6.87)

The flow rate and average headway time are both significantly affecting the P_{FM} values at the 95 percent confidence level. Since flow rate and average headway time are highly correlated, a variable may be selected to avoid the multi-colinearity issue in the regression analysis, in model development process. The selection of which variable, is based on the performance of the model on the test set discussed in the next section.

The next section presents the development of model to predict P_{FM} values using data collected in Thailand.

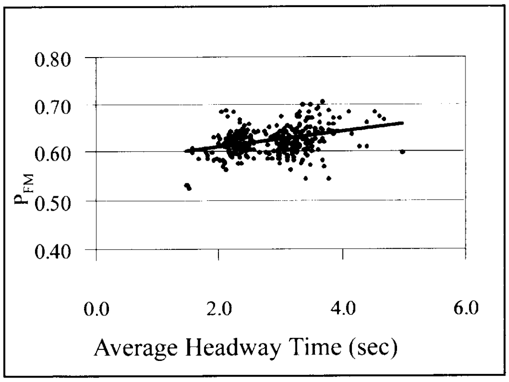

Figure. 6 P_{FM} values at various average time headway for both test sites in Thailand

Table. 5 Linear regression for P_{FM} at different average time headway

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.35018869
R Square	0.122632118
Adjusted R Square	0.119707559
Standard Error	0.028380502
Observations	302

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	0.033774108	0.033774108	41.93182391	3.85684E-10
Residual	300	0.24163586	0.000805453		
Total	301	0.275409967			

	Coefficients	*standard*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	0.575992809	0.007325811	78.6251204	3.0618E-202	0.561576323	0.590409
average headway Time	0.016216956	0.002504364	6.47548663	3.85684E-10	0.011288611	0.021145

5.3 Development of the model to predict the P_{FM} values

The model was developed on the first data set collected on November 17, 2004. Its performance was tested on a second data set collected on November 22, 2004. Figure 7 shows the linear regression model developed. The relationship between P_{FM} and average time headway may be expressed as follows:

$$P_{FM} = 0.0107h + 0.5934 \qquad (10)$$

Where;

P_{FM} = Proportion of flow entering an influence area in lane 1 and 2.

h= Average headway time (second) = 3600/q

q = Flow rate (pc/h)

The proposed model was tested on the rest of the data. It was demonstrated that the model fitted well with field data. There is no significant difference at the 95 percent confidence level. The p-value is 0.08 for the Phahonyothin location and 0.728 for Ratchadapisak location.

Table 6 and Table 7 show the t-test results of the model developed. Figure 8 shows the P_{FM} values from the model developed at different flow rates. As expected, when the flow rate increases, the proportion of vehicles in lane 1 and 2 decreases. Drivers tend to change to lane 3 when traffic is congested.

Similar model development was also performed with flow rate as an independent variable. The t-test comparison of the P_{FM} values on the test set provides the p-value of 0.0057. This indicates that P_{FM} values estimated by the model with flow rate as the independent variable significantly differ than those from the field. This suggests that the model with average headway time may be more appropriate than the model with flow rate as an independent variable.

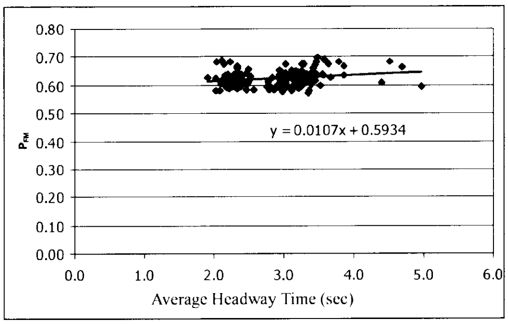

Figure. 7 P_{FM} values at various aver age headway time

Table. 6 t-test result of the model developed at Phahonyothin, Srirath Expressway

	$P_{FM\ (field)}$	$P_{FM\ (Proposed)}$
Mean	0.611804	0.6174169
Variance	0.000753	1.28392E-05
Observations	76	76
Hypothesized Mean Difference	0	
df	78	
t Stat	-1.76758	
P(T<=t) one-tail	0.040521	
t Critical one-tail	1.664625	
P(T<=t) two-tail	0.081042	
t Critical two-tail	1.990847	

Table. 7 t-test result of the model developed at Ratchadapisak, Srirath Expressway

	$P_{FM\ (field)}$	$P_{FM\ (Proposed)}$
Mean	0.629148815	0.63054317
Variance	0.001218288	1.62604E-05
Observations	77	77
Hypothesized Mean Difference	0	
df	78	
t Stat	-0.348228604	
P(T<=t) one-tail	0.364303303	
t Critical one-tail	1.664624645	
P(T<=t) two-tail	0.728606606	
t Critical two-tail	1.990847036	

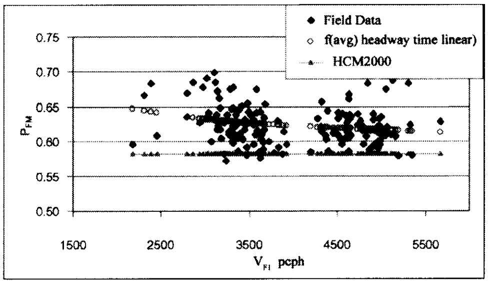

Figure. 8 P_{FM} values from the model developed

6. Conclusions and Recommendations

The HCM2000 model was tested on the data of both the Interstate 25 in the USA and the Srirath expressway in Thailand. It is likely that the proportion of vehicles in lane 1 and 2 (P_{FM}) estimated by HCM2000 is significantly different than field data at the 95 percent confidence level. Considering the analysis on the field data collected from Srirath Expressway, it shows that the average headway time has a significant affect on P_{FM} values at the 95 percent confidence level.

The proposed model developed based on data collected in Thailand with average headway time as an independent variable fitted well with the test set. Since the t-test shows that there is no significant differences at the 95 percent confidence level, using the model developed based on data collected from local locations is very suitable.

The transferability test or the test of model performance on other locations is not included in the scope of this study. Further study on transferability issue is recommended. Further investigation for other cases of ramp junctions is also recommended.

7. Acknowledgement

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