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CAPACITY OF FREEWAY MERGE AREAS WITH DIFFERENT ON-RAMP TRAFFIC FLOW

ABSTRACT

This paper is aimed at investigating the influence of different types of traffic flows on the capacity of freeway merge areas. Based on the classical gap-acceptance model, two calculating models were established specifically considering randomly arriving vehicles and individual difference in driving behaviours. Monte-Carlo simulation was implemented to reproduce the maximum traffic volume on the designed freeway merge area under different situations. The results demonstrated that the proposed calculating models have better performance than the conventional gap-acceptance theory on accurately predicting the capacity of freeway merge areas. The findings of research could be helpful to improve the microscopic traffic flow simulation model from a more practical perspective and support the designing of freeway merge areas as well.

KEY WORDS

gap-acceptance model; driver behaviour; capacity; freeway merge areas;

1. INTRODUCTION

On freeways, all entering maneuvers take place on ramps that are designed to facilitate smooth merging of on-ramp vehicles into the freeway traffic stream [1]. Interactions are dynamic in freeway merge areas, and so the operating conditions on the freeway can affect the operating conditions on the ramp, and vice versa [2]. Thus, understanding the behaviour of on-ramp flows and estimating the capacity of freeway merge areas are of critical importance in developing effective operational strategies for freeway management [3-5].

In the recent twenty years, many scholars have focused their studies on the capacity of freeway merge areas. The most widely used method of calculating the capacity of freeway merge areas has been known as the gap-acceptance model [6-9]. Previous researchers have suggested that gap-acceptance models cannot only provide a scientific approach to studying traffic control maneuvers [10, 11], but they can also calculate vehicle queue length on the ramp [12, 13] and the service quality of the freeway merge areas [14, 15]. In those studies, gap-acceptance is defined as the process in which a driver accepts an available gap between two vehicles. The model requires that drivers of on-ramp vehicles inevitably choose to merge onto the freeway as soon as they come across the best headway. In the existing studies, on-ramp vehicles are supposed to be homogeneous and the best headway is usually considered to be the minimum headway into which the on-ramp vehicles could merge onto the freeway. According to this hypothesis, drivers of on-ramp vehicles often make similar choices when encountering the same headway. They either choose to merge onto the freeway or wait for a more reasonable headway.

However, Winnie Daamen et al. concluded that the existing gap-acceptance theories have not accurately taken driver behaviour into consideration during the entire research process [16]. Vehicles which merge onto the freeway from a ramp follow a certain probability and show up randomly. Sometimes, there is no vehicle on the ramp when the best headway appears. Besides, Alexandra Kondyli and Lily Elefteriadou suggest that driver behaviour can have a significant impact on the capacity of freeway merge areas [17]. Moreover, various studies have been conducted to analyze the influence of the behaviour of drivers. Cheng-Chen Kou and Randy B. Machemehl certified that the ramp vehicle merging position in relation to the freeway has a statistically insignificant effect upon individual traffic parameters [18]. J. Wu, M. McDonald and K. Chatterjee carried out a detailed investigation to study the potential impacts of ramp metering on driving behaviour [19]. They believed that ramp metering indeed results in changes of driver behaviour when considered in the overall context of traffic on the motorway, the carriageway and the on-ramp. Alexandra Kondyli and Lily Elefteriadou developed a ramp-merging model to consider the merging process as perceived by drivers and to investigate the contribution of individual drivers' merging behaviour to the breakdown event [20]. The study indicated that when applying the gap-acceptance model to estimate the capacity of a freeway merge area, if different driving behaviours are seriously considered, more valid results will be achieved.

Accordingly, as a branch of those studies, this paper takes random arrivals of vehicles and different types of driving behaviour into account when evaluating the capacity of the freeway merge areas. In the rest of this paper, at first, based on the gap-acceptance model, the study compares two calculation models in order to assess the capacity of freeway merge areas. After that, the characteristics of different models are further studied by applying the Monte-Carlo simulation technique. The main research achievements and potential directions for future research are presented in the section of conclusions.

2. PROPOSED MODELS

2.1 Gap-acceptance model

Gap-acceptance is defined as a process by which a driver accepts an available gap between two vehicles [21]. A gap is the time between two successive vehicles from the rear bumper of the front vehicle *i* to the front bumper of the second vehicle $i-1$. It can be described as t_{mgi} in *Figure 1*.

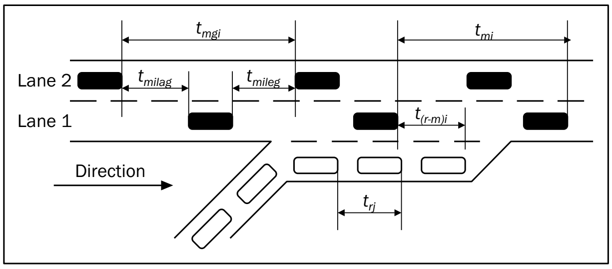

Figure 1 - Key time variables of freeway merge areas

In the gap-acceptance model, drivers will never merge onto the freeway if the gap of freeway stream is smaller than the critical gap t_{milag}. Conversely, drivers will normally merge onto the freeway as long as the freeway stream gap is larger than t_{milag}.

In practical use, it is difficult to detect the vehicle gap data. In this paper, headway, as the most widely used variable is introduced into the model. Headway is defined as the time gap between two successive vehicles from the front bumper of the front vehicle *i* to the front bumper of the following vehicle $i-1$ and is represented as t_{mi} in *Figure 1*. So, in a strict sense, t_{mgi} is not same as t_{mi} and the relationship between the two variables can be described as

$$t_{mi} = t_{mgi} + l_{vehiclei}/v_{second} \tag{1}$$

where,

$l_{vehicle}$ – the length of the front vehicle *i*,

v_{second} – the speed of the second vehicle $i-1$.

Suppose t_{r-m} represents the minimum headway that a ramp vehicle needs to merge onto the freeway. It is t_r that represents the minimum headway of a vehicle platoon on the ramp that can merge onto the freeway in the same time interval t_{mi}. So, when $t_{r-m} \le t_{mi}$ and $t_{mi} < t_{r-m} + t_r$, only one vehicle can merge onto freeway from the ramp. When $t_{r-m} + (k-1)\,t_r \le t_{mi} < t_{r-m} + kt_r$, a *k* vehicle platoon can merge onto the freeway in one gap at the same time.

The discussions of this problem generally assume that the headway of freeway vehicles is exponentially distributed with mean $\mu_m = 1/\lambda_m$. That is to say, the probability of $t_{r-m} \le t_{mi}$ can be described as

$$P(t_{mi} \ge t_{r-m}) = e^{-\lambda_m t_{r-m}} \tag{2}$$

So, the probability of $t_{r-m} + (k-1)\,t_r \le t_{mi} < t_{r-m} + kt_r$ can be described as

$$P_k = e^{-\lambda_m[t_{r-m}+(k-1)t_r]} - e^{-\lambda_m[t_{r-m}+kt_r]} = \\ = e^{-\lambda_m t_{r-m}}\left[e^{-\lambda_m(k-1)t_r} - e^{-\lambda_m k t_r}\right] \tag{3}$$

For a time interval of duration *T*, the total number of freeway vehicles is $\lambda_m T$. Meanwhile, the amount of time gap where a *k* vehicle platoon could merge onto freeway from the ramp is $\lambda_m T P_k$. In this situation, the total volume of vehicles that can merge onto the freeway from the on-ramp could be calculated by Equation (4).

$$N_r = \sum_{k=1} \lambda_m T P_k k \tag{4}$$

where *n* is the flow rate of vehicles that can merge onto freeway from the ramp in headway t_{mi}. Substituting Equation (3) into Equation (4) yields

$$N_r = \lambda_m T e^{-\lambda_m t_{r-m}} \sum_{n=1} n\left(e^{-\lambda_m(n-1)t_r} - e^{-\lambda_m n t_r}\right) \tag{5}$$

There is a geometric progression that can be found in the right side of Equation (5). According to the geometric sequence sum calculation, Equation (5) can be represented as

$$N_r = \lambda_m T e^{-\lambda_m t_{r-m}}\left[\frac{1 - e^{-n\lambda_m t_r}}{1 - e^{-\lambda_m t_r}} - ne^{-\lambda_m n t_r}\right] \tag{6}$$

The maximum flow volume of vehicles on ramp can be calculated as

$$C_{ramp} = \lim_{n\to\infty} N_r = \lambda_m T\frac{e^{-\lambda_m t_{r-m}}}{1 - e^{-\lambda_m t_r}} = q_1\frac{e^{-\lambda_m t_{r-m}}}{1 - e^{-\lambda_m t_r}} \tag{7}$$

Therefore, the formula for calculating the capacity of the merge area can be inferred as

$$q_{R12} = q_2 + q_1\left(1 + \frac{e^{-\lambda_m t_{r-m}}}{1 - e^{-\lambda_m t_r}}\right) \tag{8}$$

where,

q_{R12} is the capacity of freeway merge area;

q_1 is the capacity of freeway lane 1;

q_2 is the capacity of freeway lane 2.

However, there are two problems that have not been sufficiently considered in Equation (8). On the one hand, although vehicles which merge onto freeway from the ramp, turn up following a particular distribution; in general their emergence takes place at random. This kind of randomness cannot ensure the vehicle merging onto freeway when the best headway comes across. This means that the actual traffic which can merge onto freeway from the ramp is smaller than the theoretical value. On the other hand, different types of driving behaviours will make great impact on the capacity of freeway merge areas. Therefore, in order to improve the accuracy of the equation, random arrivals of vehicles and different types of driving behaviour will be further studied in this paper.

2.2 Case A: on-ramp vehicles arrive at random

In this case, on-ramp vehicles arrive at random. The headway of ramp vehicles is exponentially distributed with mean $\mu_r = 1/\lambda_r$. So, at time interval t_{mi}, the probability of the occurrence of a *k* vehicle platoon on the ramp is given by

$$P_{kr} = P(k) = \frac{(\lambda_r t_{mi})^k e^{-\lambda_r t_{mi}}}{k!} \tag{9}$$

So, the probability for *k* vehicles which can merge onto freeway in the platoon is

$$P_k = P_{kg} * P_{kr} = \\ = \frac{(\lambda_r t_{mi})^k e^{-\lambda_r t_{mi}}}{k!} e^{-\lambda_m t_{r-m}}\left[e^{-\lambda_m (k-1) t_r} - e^{-\lambda_m k t_r}\right] \tag{10}$$

For time interval of duration *T*, the maximum actual ramp flow rate that can merge onto the freeway can be calculated as

$$C_{ramp} = \sum_{k=1}^{n}\lambda_m T P_k k = \\ = \sum_{k=1}^{n}\lambda_m T k\frac{(\lambda_r t_{mi})^k e^{-(\lambda_r t_{mi} + \lambda_m t_{r-m})}}{k!}\left[e^{-\lambda_m (k-1) t_r} - e^{-\lambda_m k t_r}\right] \tag{11}$$

Therefore, the capacity of freeway merge areas can be calculated as

$$q_{R12} = q_{12} + \\ + \sum_{k=1}^{n}\lambda_m T k\frac{(\lambda_r t_{mi})^k e^{-(\lambda_r t_{mi} + \lambda_m t_{r-m})}}{k!}\left[e^{-\lambda_m (k-1) t_r} - e^{-\lambda_m k t_r}\right] \tag{12}$$

In this section, all variables are the same as previously defined.

2.3 Case B: on-ramp vehicles with different driving behaviours

Apparently, different types of drivers choose different headways to merge onto the freeway [20]. As *Figure 2* shows, aggressive drivers are more likely to force their cars onto the freeway even when the headway is not long enough. In this type of situation, the drivers of freeway vehicles have to decelerate remarkably or change lanes suddenly in order to avoid traffic conflict. Average drivers are likely to choose relatively modest headway to merge onto the freeway, while conservative drivers are more likely to choose a relatively longer headway before they choose to initiate merging.

From *Figure 2*, aggressive, average and conservative drivers make totally different decisions in the practical traffic stream. For example, the conservative drivers often refuse the headway that aggressive drivers often recognize. As a result, the calculation model could be promoted when the behaviour of drivers is taken into consideration. Based on the previous research [20], drivers' behaviour can be divided into three types, which was observed in in-vehicle experiments. The influence of different types of driving behaviour is expressed by the specific type of headway that is chosen. Suppose t_{jr-m} represents the minimum headway that can allow on-ramp vehicle *j* to merge onto the freeway. It is t_{jr} that represents the minimum headway of ve-

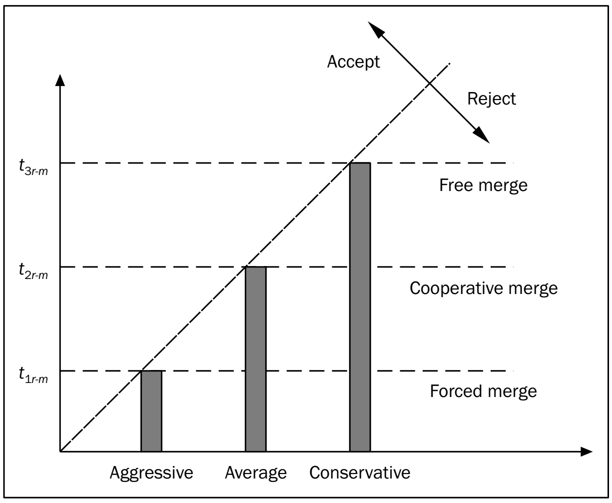

Figure 2 - Acceptable headway for different types of drivers

hicle j in the platoon that can merge onto the freeway from the ramp.

$$t_{jr-m}=\begin{cases}t_{1r-m}, \text{in case of an aggressive driver}\\ t_{2r-m}, \text{in case of an average driver}\\ t_{3r-m}, \text{in case of a conservative driver}\end{cases} \tag{13}$$

$$t_{jr}=\begin{cases}t_{1r}, \text{in case of an aggressive driver}\\ t_{2r}, \text{in case of an average driver}\\ t_{3r}, \text{in case of a conservative driver}\end{cases} \tag{14}$$

In this case, specific headway h_{mi} which allows a k vehicle platoon to merge onto the freeway can be expressed as Equation (15):

$$t_{jr-m}+\sum_{j}^{j+k-1}t_j \le h_{mi} < t_{jr-m}+\sum_{j}^{j+k}t_j \tag{15}$$

The probability of k vehicles in the platoon that can merge into the traffic flow on the freeway from the ramp can be shown as

$$P_{kg}=e^{-\lambda_m\left(t_{jr-m}+\sum_{j}^{j+k-1}t_j\right)}-e^{-\lambda_m\left(t_{jr-m}+\sum_{j}^{j+k}t_j\right)}=$$
$$=e^{-\lambda_m\left(t_{jr-m}+\sum_{j}^{j+k-1}t_{jr-m}\right)}\left[1-e^{-\lambda_m t_{(j+k)r-m}}\right] \tag{16}$$

At the time interval t_{mi}, the probability of k vehicle running on the ramp is given,

$$P_{kr}=P(k)=\frac{(\lambda_r t_{mi})^k e^{-\lambda_r t_{mi}}}{k!} \tag{17}$$

So, the probability for k vehicles which can merge onto freeway from the ramp in the platoon is

$$P_k=P_{kg}*P_{kr}=$$
$$=\frac{(\lambda_r t_{mi})^k e^{-\lambda_r t_{mi}}}{k!}e^{-\lambda_m\left(t_{jr-m}+\sum_{j}^{j+k-1}t_{jr-m}\right)}\left[1-e^{-\lambda_m t_{(j+k)r-m}}\right] \tag{18}$$

At a time interval of duration T, the maximum actual ramp flow rate that succeed in merging onto freeway can be calculated as

$$C_{ramp}=\sum_{k=1}^{n}\lambda_m T P_k k$$
$$=\sum_{k=1}^{n}\lambda_m Tk\frac{(\lambda_r t_{mi})^k e^{-\lambda_r t_{mi}}}{k!}e^{-\lambda_m\left(t_{jr-m}+\sum_{j}^{j+k-1}t_{jr-m}\right)}$$
$$\left[1-e^{-\lambda_m t_{(j+k)r-m}}\right] \tag{19}$$

Therefore, the capacity of freeway merge areas can be calculated as

$$q_{R12}=q_{12}+\sum_{k=1}^{n}\lambda_m Tk\frac{(\lambda_r t_{mi})^k e^{-\lambda_r t_{mi}}}{k!}e^{-\lambda_m\left(t_{jr-m}+\sum_{j}^{j+k-1}t_{jr-m}\right)}$$
$$\left[1-e^{-\lambda_m t_{(j+k)r-m}}\right] \tag{20}$$

In this section, all variables are the same as previously defined.

3. DATA COLLECTION

In reality, it is difficult to detect all the parameters of dynamic behaviour in the proposed models. Previous researchers have conducted studies in which the qualities of gap-acceptance model can be checked by simulation model [22]. Microscopic simulation could be used to study on-ramp metering [23] and enhance the effectiveness of traffic control technique [24]. In this study, to predict the dynamic behaviour of those parameters, Monte-Carlo simulation was applied to analyze the susceptibility and validity of the proposed models.

From Chapter 13 in Highway Capacity Manual (HCM) 2010, it is known that the total maximum volume of lane 1 and lane 2 (see *Figure 1*) should be maintained below 3,600 pcu/h to ensure no worse than a LOS of F on the freeway. That is to say, based on HCM2010, if the maximum traffic volume of two-lane freeway exceeds 3,600 pcu/h, the level of service (LOS) of freeway will fall below F. In addition, based on Zong Z. Tian's research [7], the maximum traffic volume of lane 1 should not exceed 1,200 pcu/h. So, in the process of simulation analysis, the maximum vol-

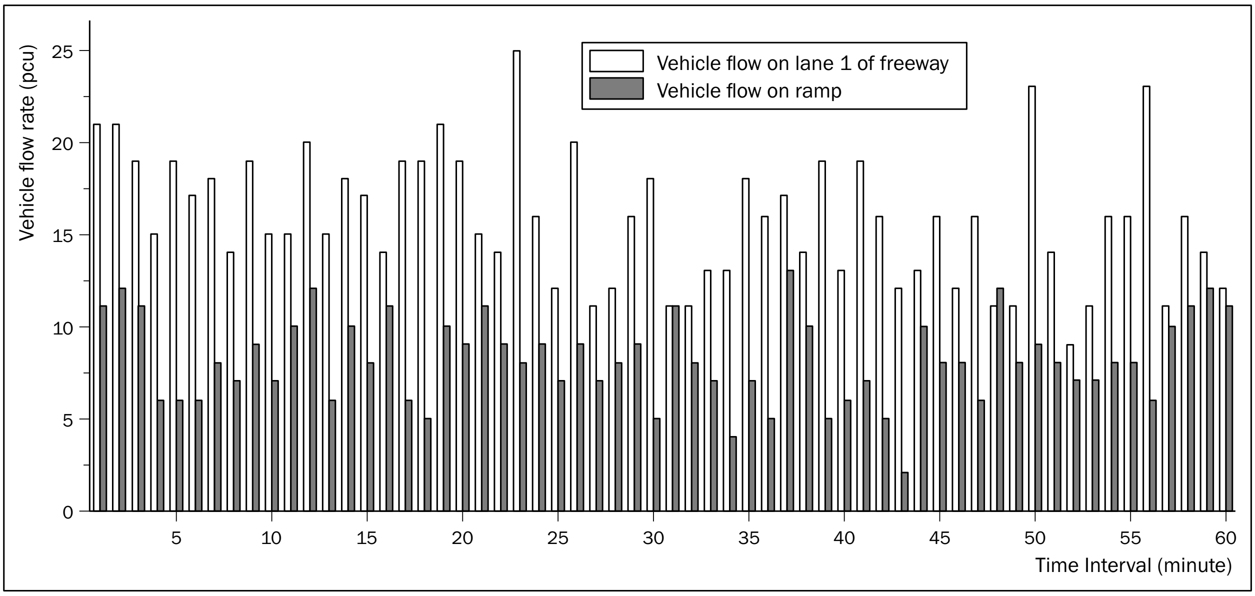

Figure 3 - Flow rates of ramp and freeway per minute

ume of lane 1 is expected to reach 1,200 pcu/h to ensure that LOS of freeway is not lower than F.

Based on the above assumptions, a simulation study has been conducted to test different gap-acceptance models. The two traffic streams (see *Figure 3*) have been generated by Monte-Carlo simulation. In this chart, the headway of ramp vehicles is exponentially distributed with mean μ_r = 6 s and the headway of vehicles on lane 1 is exponentially distributed with mean μ_m = 3 s.

For each simulation run, a combination of constant traffic volumes on freeway has been given. Traffic volumes on the ramp varied based on Case A and Case B model. In addition, repeated simulations have been conducted to reduce uncertainty. T-test has also been applied to analyze the validity of simulated data. The headway that measures less than 0.5 s is excluded in order to raise the validity of the simulated headway.

4. NUMERICAL ANALYSIS

4.1 Case A model

To study the influence of random arrivals of ramp vehicles on the capacity of freeway merge areas, the demand volume of ramp vehicles has been assumed as 1,138 pcu/h, 483 pcu/h, 177 pcu/h and 112 pcu/h, respectively. The minimum headway that an on-ramp vehicle requires in order to merge onto the freeway from the ramp is 6.5 s. The minimum headway of on-ramp vehicles that can merge onto freeway in the platoon is 3 s. In the simulation analysis, the first vehicle on the ramp would miss the headway of vehicles on the freeway as long as the headway was smaller than 6.5 s. The second vehicle would repeat the same process if the headway of vehicles on the freeway were less than 8.5 s. Otherwise, the second vehicle would merge onto the freeway following the first vehicle. This continued until the sum of total headways of the freeway was equivalent to one hour.

In *Table 1*, V1 is the simulated traffic volume of lane 1. Cr-g is the maximum traffic volume that can merge onto the freeway calculated by Equation (8), in which the values are all maintained at 499 pcu/h. Qrp (i) is the traffic volume of the on-ramp vehicles. C-rp (i) is the actual on-ramp volume calculated by Case A model with different mean headways on the ramp. Avg. Delay(i) is the average delay of ramp vehicles that succeed in merging onto the freeway. Avg. row is the average value of simulation data. The statistical results of T-test are displayed in the last two lines in *Table 1*.

In our study, T-test is selected to analyse the significance of difference in simulation data. The criterion for rejecting the hypothesis is 0.05. It means the simulation data are valid by a significance test with p > 0.95. The results of the analysis show that all the simulation data were proven to be valid according to the T-test. The results indicate that the changes of the demand volume on the ramp greatly affect the actual volume that can merge into the freeway. Especially, this effect will expand significantly when the demand volume on the ramp drops below 148 pcu/h. The results also show that the average delay of vehicles that can merge onto the freeway from ramp is enhanced remarkably with the increase of the demand volume on the ramp. The optimal volume that can merge onto the freeway from the ramp is 148 pcu/h. The increase of the demand volume on the ramp will not raise the actual ramp flow rate. Otherwise, this change could result in much more delays of the vehicles on the ramp. Therefore, when the demand volume of the ramp exceeds 148 pcu/h, ramp metering should be implemented to reduce the travel delay of vehicles. So, the actual traffic volume that can merge into the freeway from the ramp can be as described in *Figure 4*.

That is to say, their relationship can be presented as follows

$$\begin{cases} C_{ramp} = q_{ramp}, when\ q_{ramp} \leq 148\ pcu/h \\ C_{ramp} = 148, when\ q_{ramp} > 148\ pcu/h \end{cases} \tag{21}$$

At this moment, the capacity of the freeway merge area is

$$\begin{cases} v_{R12} = 3{,}600 + q_{ramp}, when\ q_{ramp} \leq 148\ pcu/h \\ v_{R12} = 3{,}748, when\ q_{ramp} > 148\ pcu/h \end{cases} \tag{22}$$

4.2 Case B model

In the analysis, the traffic volume of lane 1 is kept at 1,200 pcu/h. Based on the research of Case A model, the capacity of traffic flow is kept at 500 pcu/h in order to study the maximum traffic volume on freeway merge areas with different headways.

In addition, each type of drivers on the ramp was assigned a specific headway. With reference to previous studies [21], the related parameters for the analysis are assumed as, t_{1r-m} = 5.8 s, t_{1r} = 2 s, t_{2r-m} = 6.5 s, t_{2r} = 3 s, t_{2r-m} = 7.2 s, t_{2r} = 4 s. Vehicle i on the ramp will miss the headway of freeway vehicles until the headway is larger than the driver's minimum headway t_{1r-m}. This continued until the sum of the total headway on the freeway was equivalent to one hour. The maximum actual volume that can merge onto freeway from the ramp is illustrated in *Figure 5*.

In *Figure 5*, it is known that the actual volume that can merge onto freeway from the ramp varies with the proportions of driving behaviours, especially sensitive to the proportions of aggressive and conservative drivers. The relationship between actual ramp volume and these different types of vehicles can be illustrated in *Figure 6*.

As shown in *Figure 6*, the correlation of the two regression models is acceptable, in which all the coefficients or R values exceed 0.5, even to a maximum of

Table 1 - Simulation results of Case A model

Dataset	V1 (pcu/h)	Cr-g (pcu)	Qrp1 (pcu/h)	Crp1 (pcu)	Avg.Delay1 (s)	Qrp2 (pcu)	Crp2 (pcu)	Avg.Delay2 (s)
1	1,128	499	1,158	135	1,214	461	144	727.5
2	1,107	499	1,131	156	1,078.6	479	148	694.4
3	1,109	499	1,128	155	1,034.1	492	151	902.9
4	1,055	499	1,156	145	1,016.7	491	143	741.8
5	1,087	499	1,133	167	1,059.6	493	150	846.1
6	1,076	499	1,126	152	996.28	504	144	887.8
7	1,104	499	1,111	150	843.82	456	147	846.1
8	1,082	499	1,147	162	1,038.8	475	143	765.1
9	1,056	499	1,135	134	975.89	508	128	891.9
10	1,105	499	1,155	146	1,069	471	147	743.1
Avg.	1,091	499	1,138	150	1,032.7	483	145	804.7
t	-0.013	--	0.001	0.059	-0.001	0.001	-0.245	-0.001
Sig.	0.990	--	0.999	0.954	0.999	0.999	0.982	0.999
Dataset	V1 (pcu,h)	Cr-g (pcu)	Qrp3 (pcu/h)	Crp3 (pcu)	Avg.Delay3 (s)	Qrp4 (pcu)	Crp4 (pcu)	Avg.Delay4 (s)
1	1,128	499	180	151	165.1	108	106	41.9
2	1,107	499	178	145	143.9	110	107	37.3
3	1,109	499	164	145	160.8	108	104	41.8
4	1,055	499	167	149	136.5	115	113	49.1
5	1,087	499	170	151	198.3	118	114	42.2
6	1,076	499	180	145	185.6	112	108	30.3
7	1,104	499	183	146	189.9	107	105	41.5
8	1,082	499	191	152	191.8	115	112	34.1
9	1,056	499	163	152	160	113	104	41.5
10	1,105	499	194	143	162.1	111	109	37.5
Avg.	1,091	499	177	148	169.4	112	108	39.72
t	-0.013	--	0.001	-0.092	0.001	-0.264	0.171	0.001
Sig.	0.990	--	0.999	0.989	0.999	0.998	0.968	0.999

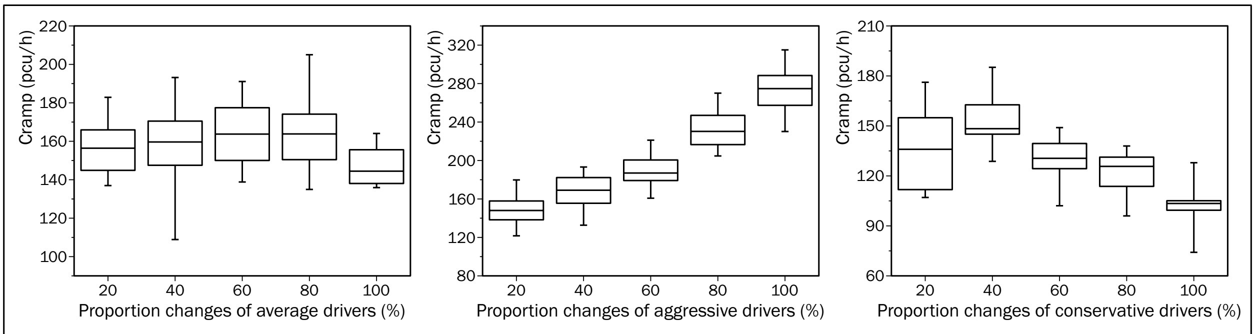

Figure 5 - Relationship between different proportions of drivers and actual volume on ramp

0.89. Therefore, actual volume that can merge onto freeway from the ramp can be derived as

$$C_{ramp} = p_i\left(134.71 + 105.35p_i\right) + p_n 147 + \\ + p_c\left(163.91 - 46.55p_c\right) \tag{23}$$

where,

p_i – represents the proportion of aggressive drivers,

p_n – represents the proportion of average drivers,

p_c – represents the proportion of conservative drivers,

and $p_i + p_n + p_c = 1$.

Thus, the capacity of the freeway merge area can be expressed as

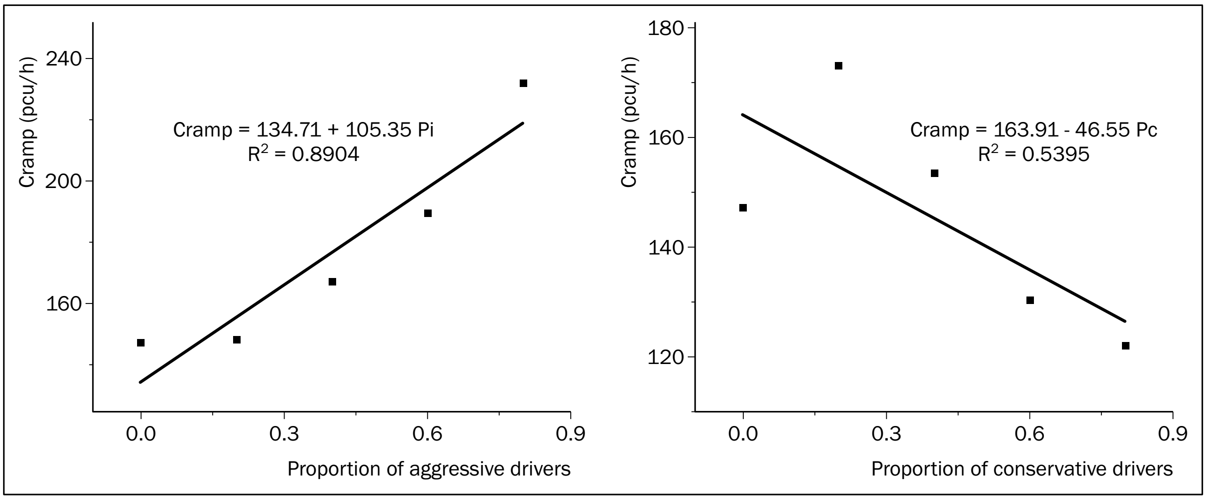

Figure 6 - Relationship between driver types and actual ramp volume

$$v_{R12} = 3{,}600 + p_i\left(134.71 + 105.35 p_i\right) + p_n 147 + $$
$$+ p_c\left(163.91 - 46.55 p_c\right) \quad (24)$$

By Equation (24), the maximum traffic volume of the two lanes freeway merge area can be calculated as 3,839 pcu/h when $p_i = 1$, and 4,320 pcu/h by the traditional gap-acceptance model. The maximum traffic volume of the two lanes freeway merge area can be calculated as 3,717 pcu/h when $p_c = 1$, the result is enlarged to 3,948 pcu/h by the traditional gap-acceptance model.

5. CONCLUSION

On the basis of gap-acceptance models, this paper establishes a more practical model for calculating the capacity of freeway merge areas. The most relevant findings of the study presented in this paper are as follows:

From the analysis results of the model in Case A, the uncontrolled random arrivals of ramp vehicles have a significant influence on the maximum actual volume that can merge into freeway from the ramp. The capacity of two lanes freeway merge areas can be calculated at 4,099 pcu/h by the traditional gap-acceptance model when the traffic flow of freeway lane 1 is maintained at 1,200 pcu/h. Under the impact of the arrival randomness of on-ramp vehicles, the maximum actual volume of the two lanes freeway merge areas was estimated at 3,748 pcu/h. In this situation, the increase of the demand volume on the ramp will not raise the actual ramp volume. On the contrary, this change could trigger much more delays to the vehicles on the ramp. Therefore, when the demand volume of the ramp exceeds 148 pcu/h, effective traffic management methods should be applied in controlling the vehicles which are planning to merge onto the freeway from the ramp. Otherwise, the traffic congestion will be inevitable.

According to the research results of Case A model, the study of Case B in this paper shows that different driving behaviours exert a significant influence on the capacity of freeway merge areas. There is a linear relationship between the maximum actual volume of the ramp and the proportions of different types of drivers. The maximal capacity of two lanes freeway merge area obtained from Equation (24) with a completely aggressive driver population ($p_i = 1$) is 3,839 pcu/h while the corresponding value from the gap-acceptance model is 4,320 pcu/h. Similarly, for maximal capacity from Equation (24) with only conservative drivers along, is 3,717 pcu/h while the value from gap-acceptance model is 3,948 pcu/hr.

From the research above, it is found that the influence of arrival randomness of on-ramp vehicles and characteristics of different drivers should be highly considered in the practical engineering application. For example, the driving behaviour of local drivers should be an important attribute and greatly considered when assessing the capacity of freeway merge areas, which could help relieve the traffic congestion brought by excessive estimation of the capacity. However, the achievements inferred from the proposed models depended on the given parameters. Although those parameters were carefully selected from previous results, it may be questionable to use those values in specific traffic conditions. Therefore, there should be a greater level of importance attached to the calibration of these parameters on different occasions while using these formulas.

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摘要

考虑不同匝道交通流特性的高速公路合流区通行能力研究

为了研究匝道上不同类型交通流对高速公路入口匝道合流区通行能力的影响，得到更符合实际情况的高速公路入口匝道合流区通行能力计算模型。基于可接受间隙理论，本文分别从匝道车辆到达随机性和匝道驾驶员驾驶行为差异性两个方面构建了不同的高速公路入口匝道合流区通行能力计算模型。为了对不同的通行能力计算模型进行定量分析，本文选择蒙特卡洛仿真方法对本文提出的两个模型以及传统的可接受间隙理论模型进行了数值模拟。数值分析的结果表明，本文提出的通行能力计算模型比传统的可接受间隙理论能更好的反应实际的高速公路入口匝道的实际情况。本文的研究成果可以用来改善交通流的微观仿真模型，也可以用于高速公路入口匝道的规划设计。

关键词

可接受间隙理论；驾驶行为；通行能力；高速公路合流区；

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