공학석사학위논문

Heterogeneous Traffic Flow Model of Freeway Weaving Sections for Ramp Metering Control

램프 미터링 제어를 위한 엇갈림구간의 이질적 교통류 모형

2015 년 8 월

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지도교수 고 승 영

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Abstract

Heterogeneous Traffic Flow Model of Freeway Weaving Sections for Ramp Metering Control

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Faced with traffic jams on freeways, one of the hottest issues is Dynamic Traffic Management, which manages the freeways dynamically according to real time traffic conditions. However, the controllers use traffic flow models that usually ignore or simplify the heterogeneity effects.

Particularly in ramp areas, a traffic flow model considering vehicle types is much more important. The frequent lane changings near on-ramps and off-ramps result in bottlenecks on freeways. Besides, it seriously decreases the capacity of the ramp-weaving sections that the trucks prefer to travel on the outer lanes. To set a proper capacity, which is deduced from the traffic flow model, is the key factor in deciding the ramp metering rate.

Therefore, the focus of this research is to develop the heterogeneous traffic flow model of the ramp-weaving sections. The main focus of this research is to clarify the heterogeneous vehicles' influence to the areas around them including the lanes they belong to as well as next to them. The characteristic of the truck influence on traffic flow is reflected as the increase on the mean time gap T.

Then apply the heterogeneous traffic flow model to ramp metering in order to verify the necessity of the heterogeneous traffic flow model and point out the risk of using traffic flow models that ignore vehicle types. The result came out to be that slight congestion happens in the ramp-weaving section if the optimized ramp-metering rate with the homogeneous model is applied. But more critically, serious congestion happens in the areas before on-ramps. "NGSIM U.S. 101 Data" is used to verify the characteristics and embody the modified model.

This research proves that it is necessary to use traffic flow models that consider the heterogeneity of the traffic flow. The benefits of the heterogeneous traffic flow model will come to light with its development.

Keywords: Heterogeneous traffic flow, Freeway weaving section,
Macroscopic traffic flow model, Ramp metering
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Chapter 1. Introduction

1.1 Research Background

With the boom of the vehicle uses, the traffic problems have been one of the most striking social issues in these decades. There appeals traffic jams in every big city every day. To solve these traffic problems, or at least to minimize the travelers' delay on the road, the transportation engineers, the governments, and many other institutions have poured their efforts in kinds of aspects. And many operation strategies and policies, such as parking policy, freeway pricing, ramp metering, and dynamic traffic signals, have been implemented.

Freeways, which are mostly used for fast travels, although with big capacities, always meet traffic jams as well. It makes freeways cannot play their role efficiently and reduce their reliabilities to users. Therefore, a lot of scholars have studied various theories and strategies to get the traffic condition in the optimized state. One of the hottest issues is Dynamic Traffic Management (DTM), which manages and operates the freeways dynamically according to the real time traffic conditions. However, the fickle traffic conditions are the first thing to know well enough whenever evaluating the transportation level of service or implementing an operation strategy.

However, the controllers are operated regardless of the vehicle types.

While, the differences between the characteristics of different vehicle types do exist. And “the differences that characterize the mixed traffic systems, otherwise known as heterogeneous traffic systems, are mainly due to the wide variation in the operating and performance characteristics of vehicle types”.[6] Therefore, the traffic flow models, which describe the traffic conditions in mathematical way and contribute a lot to penetrate the traffic conditions, should consider the vehicle types in order to be more suitable. While most of the current traffic flow models ignore or simplify the heterogeneity effects caused by multi-class vehicles. Especially in ramp areas, a traffic flow model considering vehicle types (shortened as heterogeneous traffic flow model in this research) is much more important. The frequent lane changings near on-ramps and off-ramps result bottlenecks on freeways. Besides, it seriously declines the capacity of the ramp-weaving sections that the trucks prefer to travel on the outer lanes. To set a proper capacity, which is deduced from the traffic flow model, is the key factor in deciding the ramp metering rate.

Therefore, to prove the necessity of the traffic flow model considering vehicle types and to develop the heterogeneous traffic flow model for the ramp-weaving sections is the focus of this research. The main specialty of this research is to clarify the heterogeneous vehicles’ influence to the areas around them including the lanes they belong to as well as next to them. And then apply the modified model to ramp metering in order to verify the necessity of the heterogeneous traffic flow model and point out the risk of using the traffic flow models ignoring the vehicle types.

1.2 Current Studies

Over the past several years, “contributions from engineers, physicists, mathematicians, and behavioral psychologists have lead to a better understanding of driver behavior and vehicular traffic flow”.[20] Various kinds of studies have been carried out, and the traffic flow theory is one of the most significant theories.

However, as mentioned above, one of the most important properties is that the traffic is composed of multiple vehicle types. But most traffic flow models deal with homogeneous traffic conditions. This assumption causes critical errors especially when applied to dynamic traffic management, like ramp metering systems. For the dynamic traffic management usually try to control the traffic conditions near capacity conditions in order to take full advantage of the systems.

Passenger car equivalent (PCE) or passenger car unit (PCU) is used to convert a heterogeneous traffic flow into an equivalent (homogeneous) passenger car flow. Many variables, including delay time, flow rate, speed and so on, are proved to be related to PCE value (see <Table 2.1>). And the traffic managements, which take the vehicle types into account, usually use PCE as the only modification factor. Even though PCE value helped to modify the traffic flow with multiple vehicle types in a simple way, it only considers the influence to the same lane, and obviously ignored the influence of the trucks to the heterogeneous traffic flow.

To present and predict the traffic conditions more suitably, the traffic flow models should also count in the influence to the surrounding areas including the lanes next to them led by the heterogeneity, which contains the following vehicles' long headway, lane changing and their secondary influence on the nearby lanes. And this research is to supplement this point to the heterogeneous traffic flow model.

1.3 Overview of Research

This research focused on developing a heterogeneous traffic flow model for the ramp areas which reflects the influence of the heterogeneity of the traffic flow on the surrounding areas.

The rest part of this paper is composed as following sequence:

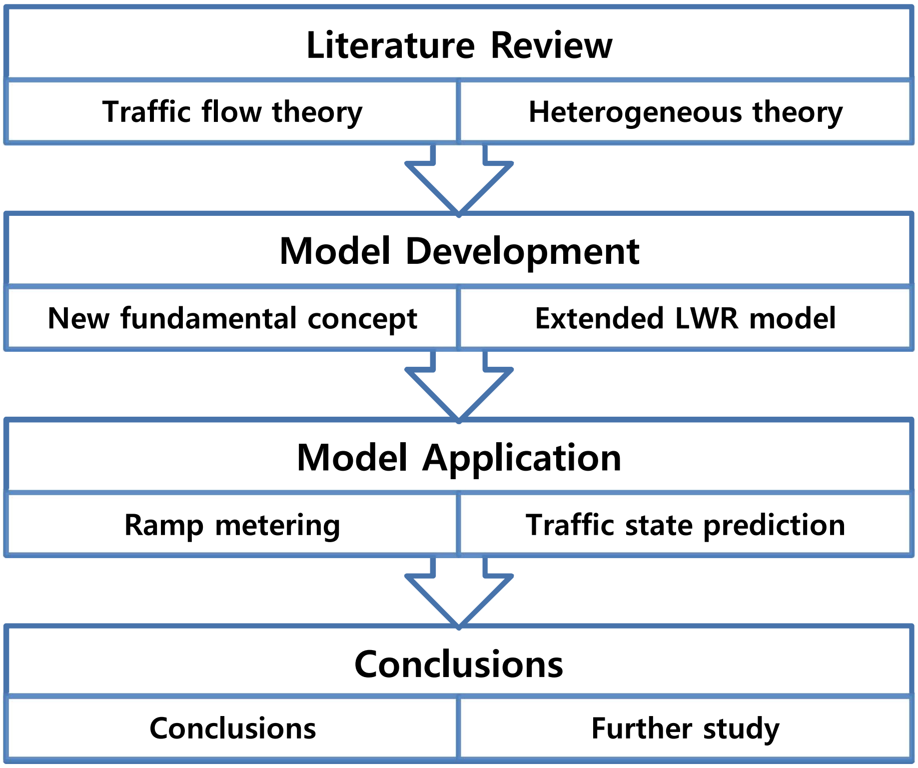
In Chapter 2, the literatures are reviewed briefly in two categories: the basic traffic flow theory and the heterogeneous theory;

Chapter 3 introduces the model development procedure including the concept of the modified model and the characteristics of the heterogeneous traffic flow;

Chapter 4 shows the verification of the model, in which the analysis proceeding of the heterogeneous traffic flow using the actually detected data is presented;

Then ramp metering is implemented in Chapter 5 in order to compare the results of using the homogeneous model and the developed model presented in Chapter 3, as well as to show the necessity of the heterogeneous traffic flow model and the risk of using the homogeneous traffic flow;

Finally in Chapter 6, the conclusions and the further studies are presented.



<Figure 1.1> Research Flow Chart

Chapter 2. Literature Review

The scientific study on the traffic flow is considered to begin in the 1930's along with the application of the probability theory to describing the traffic flow (Adams 1936), the studies conducted by Greenshields at the Yale Bureau of Highway Traffic, the models relating to the volume and speed (Greenshields 1935) and the investigation of the traffic performance at intersections (Greenshields 1947).[4] Then numerous branches of the traffic flow theory have been developed in later decades, such as kinematic theory, car following theory, lane changing theory and so on. And the researches considering vehicle types have also got a certain development, which are called as heterogeneous traffic, mixed traffic or multi-class traffic. For conciseness, this research uses the word "heterogeneous traffic" to present the traffic flow consisting of two or more types of vehicles and "homogeneous traffic" to describe the traffic with only passenger cars.

Based on the topic of this research, the literature review, which focuses on the macroscopic traffic flow theory, is divided into the homogeneous traffic flow theory and the heterogeneous traffic flow theory.

2.1 Homogenous Traffic Flow Theory

Homogenous traffic flow theory was expended under the assumption that the vehicles in the traffic flow share the same type and other physical characteristics. Many of the theories are developed under this assumption including traffic flow fundamentals, LWR models and so on.

2.1.1 Traffic flow fundamentals

Traffic flow fundamental describes the relationship between three traffic flow properties: flow rate $\mathrm{q}(\mathrm{veh/h \cdot lane})$, speed $u(km/h)$ and density $k(veh/km \cdot lane)$. (These notations are set as default notations in this paper, and the annotations are omitted.) Kinds of models are proposed to find out their relationships, including linear model and exponential model.

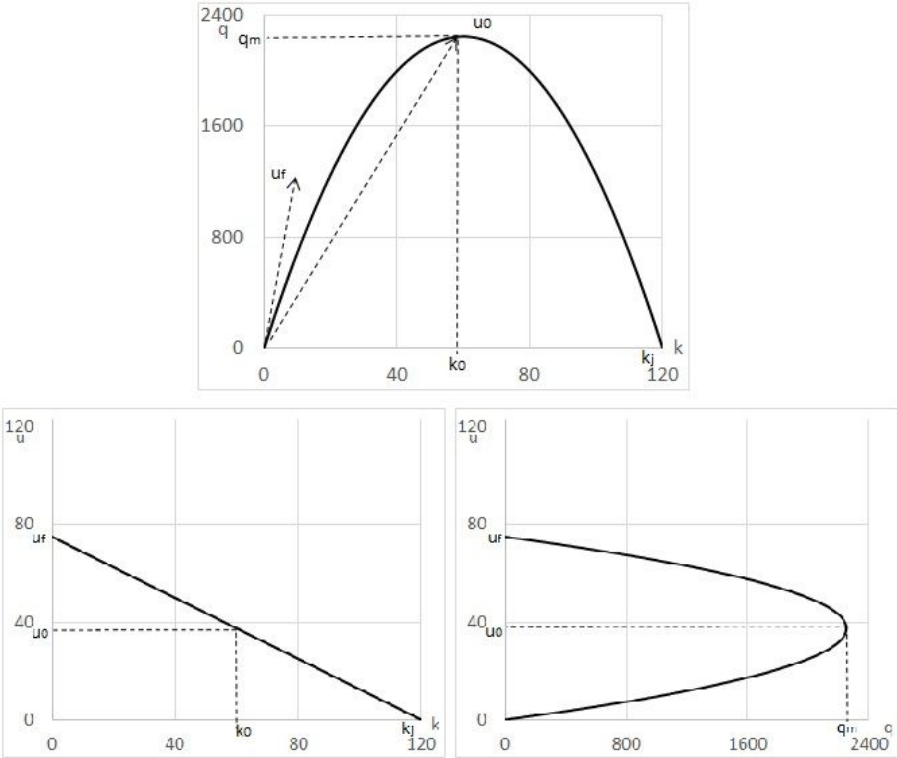
i. Basic fundamental diagram

According to the basic fundamental, the relationship between the three traffic flow characteristics can be presented as following equation:[10]

$$\begin{aligned} \mathrm{q} &= u_f k - \frac{u_f}{k_j} k^2 \\ u &= u_f - \frac{u_f}{k_j} k \\ k_j u - \frac{k_j}{u_f} u^2 &= q \end{aligned} \tag{2-1}$$

Where, u_f means free flow speed; u_0 means optimum speed; k_j means jam density; k_0 means optimum density; and $q_m(= u_0 \cdot k_0)$ is the maximum flow rate or capacity.

<Figure 2.1> is the diagrams of Equation (2-1), which shows the basic fundamental diagram.



<Figure 2.1> Basic Stream Flow Diagrams[①]

This is a parabolic shape's q~k graph and the relationships between k_j and k_0; u_f and u_0 are as follows:

$$
\begin{aligned}
k_0 &= \frac{1}{2}k_j \\
u_0 &= \frac{1}{2}u_f
\end{aligned}
\tag{2-2}
$$

From Equation (2-1) and <Figure 2.1>, we can see that the relation is

① Reappeared according to: Adolf D. May, Traffic flow fundamentals, Prentice Hall, 1990, pp.284.

continuous in a certain range. The flow rate will increase with the density rises, while the speed goes in the opposite direction. When the flow rate reaches the maximum value, the traffic enters the breakpoint. If the density still grows, the flow rate will fall down and the speed continues to decrease.

It should be noted that the speed-density relationship is assumed linear and that space-mean-speed rather than time-mean-speed should be employed. Besides, this is always continuous in its domain.

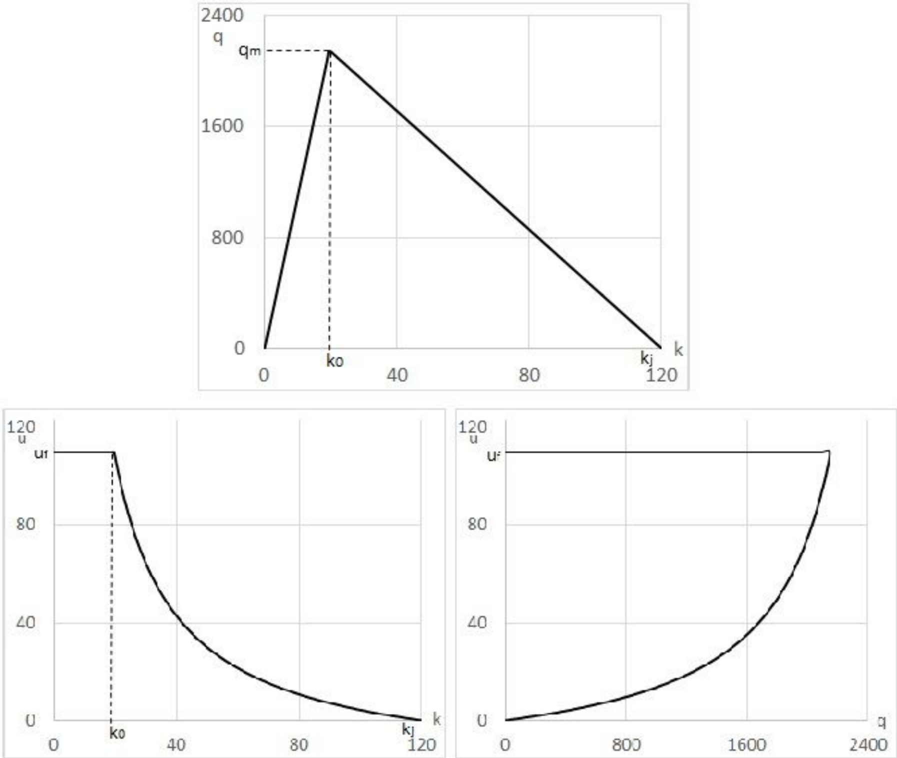
ii. Triangular fundamental diagram

In some cases, the triangular fundamental diagram is used for its numerical efficiency. It assumes that the traffic flow speed remains the same as the free flow speed until the traffic arrives at the breakpoint.

The relationship can be presented as following equation:[20]

$$
\begin{gathered}
\mathrm{q} = \begin{cases} u_f k & 0 \le k < k_0 \\ \frac{1}{T}\left(1 - \frac{k}{k_j}\right) & k_0 < k \le k_j \end{cases} \\
u = \begin{cases} u_f & 0 \le k < k_0 \\ \frac{1}{T}\left(\frac{1}{k} - \frac{1}{k_j}\right) & k_0 < k \le k_j \end{cases} \\
u = \begin{cases} u_f & 0 \le k < k_0 \\ & or \\ \frac{1}{k_j}\left(\frac{q}{1-T\cdot q}\right) & k_0 < k \le k_j \end{cases}
\end{gathered}
\tag{2-3}
$$

Where, u_f, k_j, k_0, and $q_m (= u_f \cdot k_0)$ have the same meaning as in Equation (2-1); T means the average time gap.



<Figure 2.2> Triangular Fundamental Diagrams②

The calculation formula of the optimum density k_0 is as follows:[20]

$$k_0 = \frac{1}{u_f T + l_{eff}} \tag{2-4}$$

Where, l_{eff} presents the minimum distance headway, which equals the average vehicle length plus the average minimum distance gap in stopped traffic.

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② Reappeared according to: Martin Treiber and Arne Kesting. Traffic flow dynamics: data, models and simulation. Springer, 2013, pp.92.

In many field observations, the triangular fundamental diagrams match the detected data and have smaller deviations. And it also improves mathematical efficiency within a regime. But the multi-regime linear model is discontinuous near breakpoint, which leads to mathematical difficult in cross-regime.

2.1.2 LWR model

In 1950s', Lighthill, Whitham[6] and Richards[8] independently proposed the dynamic traffic flow model (named as LWR model) using conservation law. And extended models are developed to modify the unrealistic shortcomings of LWR model. LWR model also became the basic theory for many heterogeneous traffic researches.

i. Basic LWR model

The basic LWR model, which is called to be distinguished from the extended LWR model, is presented as following equation:

$$\frac{\partial k}{\partial t} + \frac{dq(k)}{dk}\frac{\partial k}{\partial x} = 0 \tag{2-5}$$

Since Equation (2-5) LWR model refers to a whole class of models, and doesn't specify the functional form of the fundamental diagram $q(k)$, a lot of specific functions have been proposed. Thus all models in this class only have one dynamic continuity equation. Therefore, they are also called as the first-order models.[20] In contrast, the second-order models assume that the local speed is an independent dynamic quantity which should be modeled by an additional equation.

ii. Extended LWR model

From microscopic point, the associated instantaneous speed adaptations of LWR model indicate unlimited acceleration value which is unrealistic. Besides, the finite speed adaptation times and reaction times are the main factors leading to growing traffic waves and capacity drop phenomena, or to traffic flow instabilities in general.[20] Consequently, LWR model cannot overcome these shortcomings.

In the second-order models, the acceleration equation is a second partial differential equation of the general form as follows:

$$\frac{\partial k}{\partial t}+\frac{\partial (k\cdot u)}{\partial x}=\frac{\partial}{\partial x}\left(\mathrm{D}\frac{\partial k}{\partial x}\right)-\frac{k\cdot u}{I}\cdot\frac{dI}{dx}+v_{rmp}(x,t) \tag{2-6}$$

$$\frac{du}{dt}=\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}=\frac{u_e^*(k,u,k_a,u_a)-u}{\tau}-\frac{1}{k}\frac{\partial P}{\partial x}+\frac{1}{k}\frac{\partial}{\partial x}\left(\eta\frac{\partial u}{\partial x}\right)+A_{rmp}(x,t) \tag{2-7}$$

Where, D is the diffusion constant; I is the number of lanes; the subscript rmp means the parameters of ramp flow; u_e^* presents the generalized targeted speed that can be seen as the steady-state speed; τ means the speed adaptation time; η is the order of magnitude of the diffusion constant; P is a traffic pressure term; A is the acceleration term.

One of the representatives of the second-order macroscopic model is Payne's model. Payne's model is a special case of the general macroscopic acceleration equation for the constant speed relaxation time τ, zero diffusion η, and the density dependent traffic pressure term $P=-u_e(k)/2\tau$.[14]

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}=\frac{u_e(k)-u}{\tau}+\frac{u_e{'}(k)}{2k\tau}\frac{\partial k}{\partial x} \tag{2-8}$$

2.1.3 Multi-lane model

Daganzo (2002)[2][3] proposed the macroscopic traffic dynamic theory for homogeneous, multi-lane freeways. The model used 2-pipe regime model on two lanes and the drivers were divided into two types: slugs and rabbits. He mathematically resolved the capacity drop as well as the pumping effect caused by “entrance” and “merge”.

There are other such models that are developed considering the driving speed as the factor of vehicle types. However, it is still important to distinguish the vehicle types because of their obviously different physical characteristics which would result in the different influence to the traffic flow.

2.2 Heterogeneous Traffic Flow Theory

Distinguished with the homogeneous traffic flow theory, in which the vehicles share the same characteristics as the passenger car, there are many studies and managements deal with the heterogeneity of the traffic flow. And most of the studies calculated the different vehicle types using Passenger Car Equivalent (PCE) value. While some studies used parameters like area occupancy to calibrate heterogeneous traffic flow.

2.2.1 Passenger Car Equivalent

PCE value is mainly used in highway capacity analysis and estimate to convert the heterogeneous traffic flow into homogeneous traffic flow with an equivalent passenger car value. And it is related to the determination of the capacity, lane requirement, the level of service and some other indexes on highway planning, and the estimate of them on highway operations.[5] PCE is also used in the traffic signal operations to decide the optimized signal timing and lane arrangement.

PCE, which is reported to be decided by the percent and length of grade of the basic freeway segments and the proportion of heavy vehicles in Highway Capacity Manual (HCM), has been studied in many researches in different ways. According to Ingle, A. (2004), PCE value is considered to be related with kinds of variables such as delay, flow rate, speed, and so on (see details in <Table 2.1>).[5]

<Table 2.1> Variables Related to Passenger Car Equivalent

Variables	Relationship
Delay	Combined the relative number of passings and the relative delay method.
Flow rate	Flow rate of base flow and mixed flow for a given v/c ratio considered
Speed	Speed and headway considered
v/c ratio	Multiple linear equation whereby the v/c ratio
Headway	Average headways or minimum headways considered
Queue discharge flow	Congested condition is represented by queue discharge flow, where v/c=1
Density	Use equal density to relate mixed flow rate and base flow rate

In many traffic flow models and the dynamic traffic managements, PCE is used differently in different cases. Here are some representatives.

i. Extended LWR model using PCE

Logghe, S. and B. Immers (2003) used constant PCE to extend LWR model to a heterogeneous traffic flow model. They divided the vehicles into classes and assumed the fundamental diagrams to be similar in shape. The extended LWR model was examined through an analytical and numerical case study so as to illustrate the relation with the much used PCE.

ii. Independent ramp metering for multi-class

Schreiter, T. et al (2011) proposed a multi-class ramp metering strategy using simple dynamic PCE value: PCE=2 for free speed and PCE=6 for traffic congestion. The queue in the on-ramp consisted of two lines for passenger cars and trucks respectively. This research aimed to minimize the total values of time considering the different values of passenger cars and trucks.

iii. Fastlane model

Schreiter, T. et al (2012) adopted dynamically changing PCE values for traffic management in Fastlane model. And the PCE values were decided by the vehicle speed. The model, which is reformulated in the Lagrangian coordinate system, is used for a series of DTM applications.

2.2.2 Others

There are some traffic flow models adopted other properties rather than PCE to calibrate the heterogeneous factor. Mohan, R. and G. Ramadurai (2013) adopted "Area Occupancy" as the new variable to develop the heterogeneous model (Equation (2-9)).

$$\begin{cases} AO = \frac{\sum_m t_m a_m}{TA} \\ \frac{\partial k}{\partial t} + \frac{\partial (ku)}{\partial x} = 0 \\ \frac{\partial (u+p(AO))}{\partial t} + u\frac{\partial (u+p(AO))}{\partial x} = \frac{u_e(AO)-u}{\tau} \end{cases} \tag{2-9}$$

Where, t_m is the occupancy time of the vehicle m, T is the total occupancy time, a_m means the area of vehicle m, A is the sum of areas, and $p(AO)$ is the traffic pressure term which is decided by area occupancy.

The modified model is proposed as a good measure of traffic concentration instead of the measure density. And when compared with the

LWR model, this model could better capture the congestion propagation. However, it models for the roads where the lane discipline is not strictly followed.

Those heterogeneous traffic flow models and managements simplify the heterogeneity effects by using PCE value or other index. The distinction of this research is to analyze the heterogeneous vehicles' influence to the areas around them.

Chapter 3.   Model Concept and Development

To develop the heterogeneous traffic flow model, the model concept and the characteristics of the heterogeneous traffic flow are presented in this part. Then, the theories are reorganized in model development.

For the generality and effectiveness, this research only deals with two classes of the vehicles: automobile and truck. The class “truck” represents the heavy vehicles including various buses and trucks.

3.1 Traffic Flow Fundamental Combining PCE

The parabolic traffic flow fundamental was developed based on the continuum theory. However, this research would adopt the triangular fundamental as the base of the model development. As mentioned above, the triangular fundamental has an advantage over the parabolic fundamental in the numerical efficiency. Another important reason is that many field observations are proved to more match with it.

The homogeneous fundamental of the automobiles has already been introduced in literature review. Therefore, the PCE value of the truck is applied to the triangular flow fundamental diagrams in order to compare the fundamental diagrams of the automobile and the truck. Besides, to observe the features of every case in which using different PCE value is one of the main purposes.

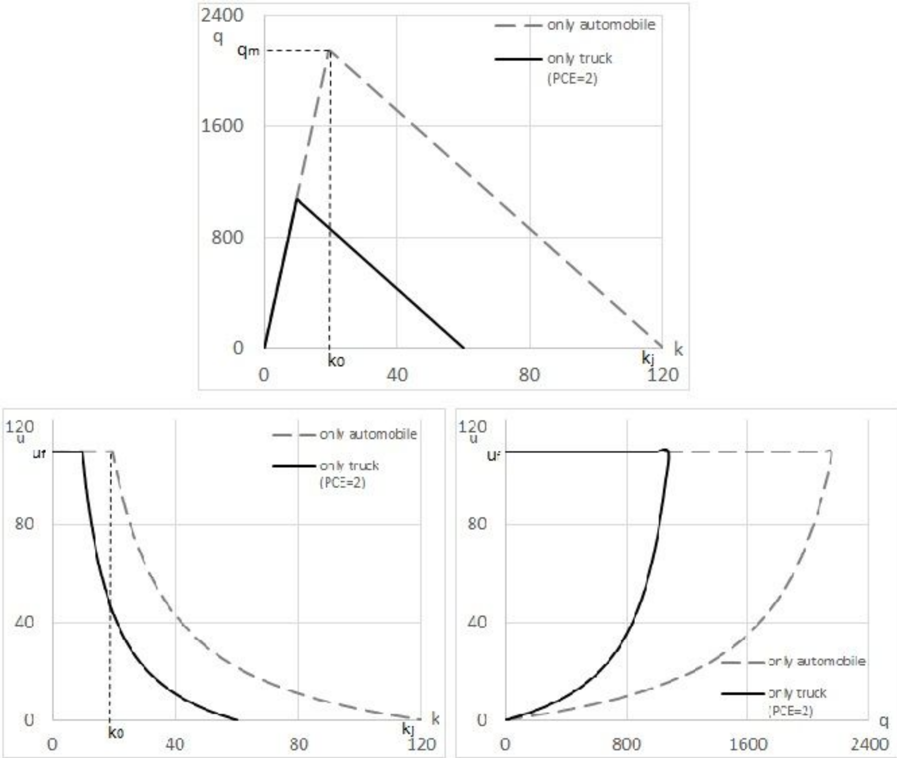
The relationship between three traffic flow properties ($\mathrm{q(veh/h \cdot lane)}$, $u(km/h)$ and $k(veh/km \cdot lane)$) of truck, which is going to be shown below, presents the traffic flow fundamentals when the traffic consists of only trucks. And it can be re-written as following equation:

$$
\begin{aligned}
q^t &= \begin{cases} u_f k^t & 0 \le k^t < {k_0}^t \\ \frac{1}{T}\left(\frac{1}{PCE} - \frac{k^t}{{k_j}^t}\right) & {k_0}^t < k^t \le {k_j}^t \end{cases} \\
u^t &= \begin{cases} u_f & 0 \le k^t < {k_0}^t \\ \frac{1}{T}\left(\frac{1}{PCE \cdot k^t} - \frac{1}{{k_j}^t}\right) & {k_0}^t < k^t \le {k_j}^t \end{cases} \\
u^t &= \begin{cases} u_f & 0 \le k^t < {k_0}^t \\ & or \\ \frac{1}{{k_j}^t}\left(\frac{q^t}{\frac{1}{PCE} - T \cdot q^t}\right) & {k_0}^t < k^t \le {k_j}^t \end{cases}
\end{aligned}
\tag{3-1}
$$

Where, the superscript t represents the word "truck" so as to differentiate from the origin. It is obvious that the maximum density and the optimum density would also be different from that of the automobiles. And they depend on the PCE value.

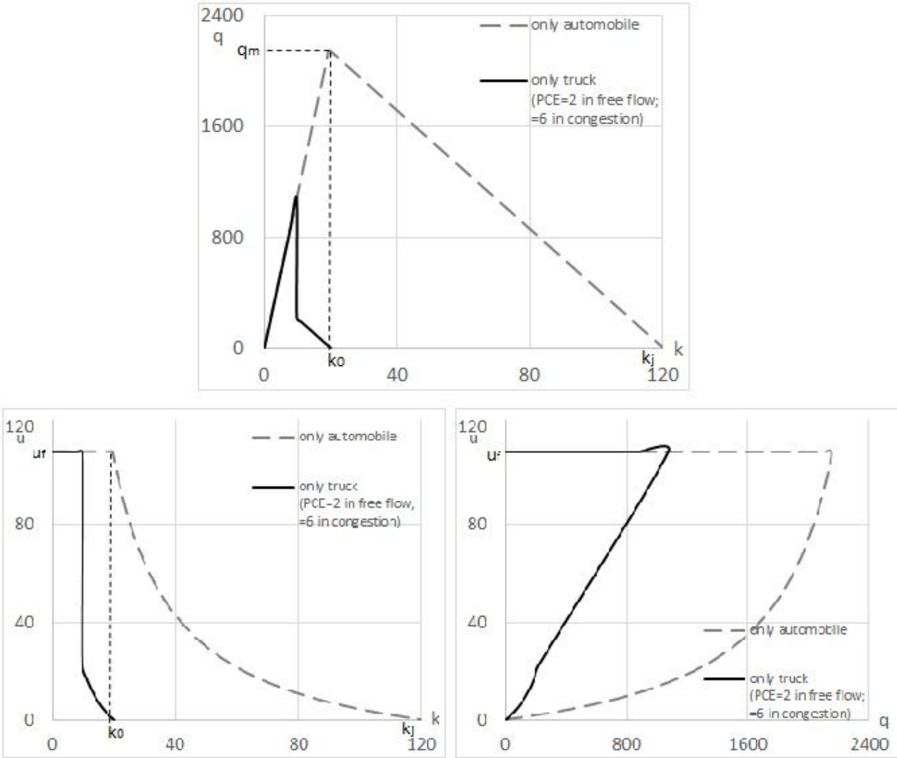
Three cases refer to the differently adopted PCE values: i) constant as 2; ii) equals to 2 in free flow, 6 in congestion; and iii) linearly increasing from 2 in zero density to 4 in the maximum density. The fundamental diagrams for the three cases are shown in <Figure 3.1> ~ <Figure 3.3> respectively.

Here, it should be noted that the density $\mathrm{k(veh/km \cdot lane)}$ wasn't converted to the passenger car unit. And the free flow speed is assumed to stay the same.



<Figure 3.1> Comparison of Triangular Fundamental Diagrams between Automobile and Truck (PCE=2)

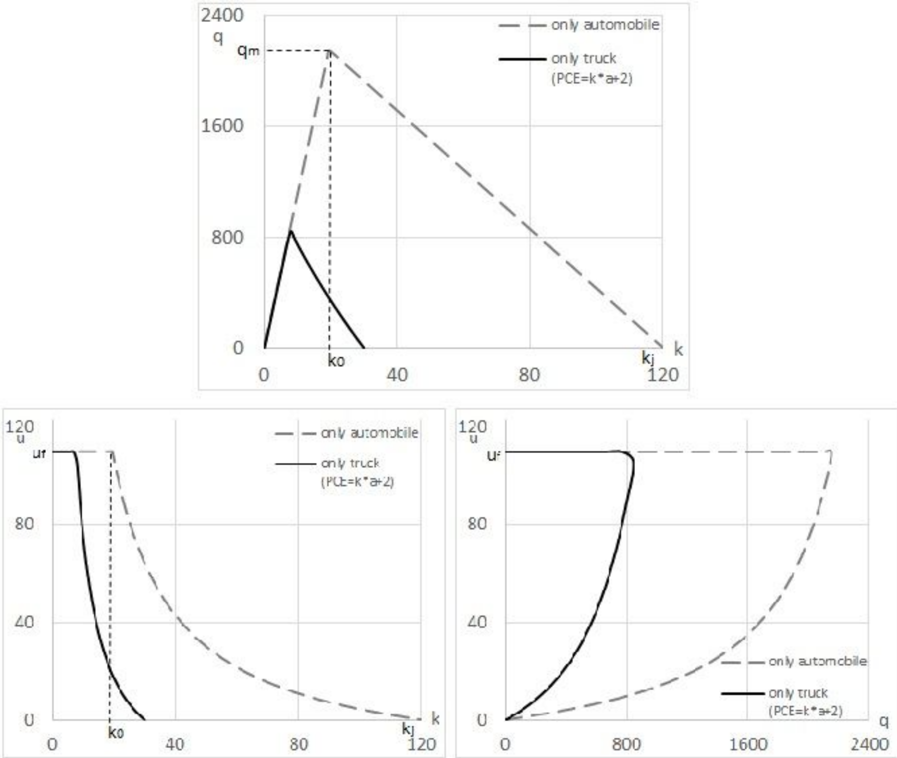
From <Figure 3.1>, we can see that the breakpoint and the maximum density of the truck traffic flow come twice faster than those in the automobile traffic flow. Because using 2 as the PCE value, the density and the flow rate are reduced to the half of the original values under the same speed.



<Figure 3.2> Comparison of Triangular Fundamental Diagrams between Automobile and Truck (PCE=2 in free flow; 6 in congestion)

Setting PCE value as 2 in free flow and 6 in congestion makes the fundamental diagrams much diverse. The diagrams are similar to those of case i before breakpoint. After breakpoint, for PCE value of 6, the flow rate and the speed fall down rapidly as the density increases.

The discontinuity PCE even leads to the fundamental diagrams divide into two pieces at breakpoint. This is unpractical as well as inefficient for calculation.



<Figure 3.3> Comparison of Triangular Fundamental Diagrams between Automobile and Truck ($\mathrm{PCE} = \mathrm{k} \times \mathrm{a} + 2$)

As in <Figure 3.3>, the linearly increasing PCE value ($\mathrm{PCE} = \mathrm{k} \times \mathrm{a} + 2$, from 2 in zero-density to 4 in maximum density with the constant increasing rate of a) reduces the speed greater than that of the constant PCE 2. And the maximum density goes down to one fourth of the original one because of the maximum PCE value. This is more suitable to the cases with the slope road or the frequent stop-and-go phenomena. It is because that the trucks are not good at crawling or changing speeds neatly as the automobiles.

Then in the case of the heterogeneous traffic flow, assuming that the traffic flow consists of 90% automobiles and 10% trucks, the traffic flow fundamentals can be derived simply by linear combination of 90% automobiles and 10% trucks as Equation (3-2).

$$
\begin{aligned}
q^H &= \begin{cases} u_f k^H & 0 \le k^H < {k_0}^H \\ \frac{1}{T}\left(\frac{0.1+0.9\cdot PCE}{PCE} - \frac{k^H}{k_j}\right) & {k_0}^H < k^H \le {k_j}^H \end{cases} \\
u^H &= \begin{cases} u_f & 0 \le k^H < {k_0}^H \\ \frac{1}{T}\left(\frac{0.1+0.9\cdot PCE}{PCE\cdot k^H} - \frac{1}{k_j}\right) & {k_0}^H < k^H \le {k_j}^H \end{cases} \\
u^H &= \begin{cases} u_f & 0 \le k^H < {k_0}^H \\ & or \\ \frac{1}{{k_j}^H}\left(\frac{q^H}{\frac{0.1+0.9\cdot PCE}{PCE} - T\cdot q^H}\right) & {k_0}^H < k^H \le {k_j}^H \end{cases}
\end{aligned} \tag{3-2}
$$

Where, the superscript H represents the word “heterogeneous” so as to differentiate from others. The maximum density and the optimum density should also be different from that of the automobiles or the trucks. In addition to the PCE value, they also depend on the traffic flow ratio of the trucks.

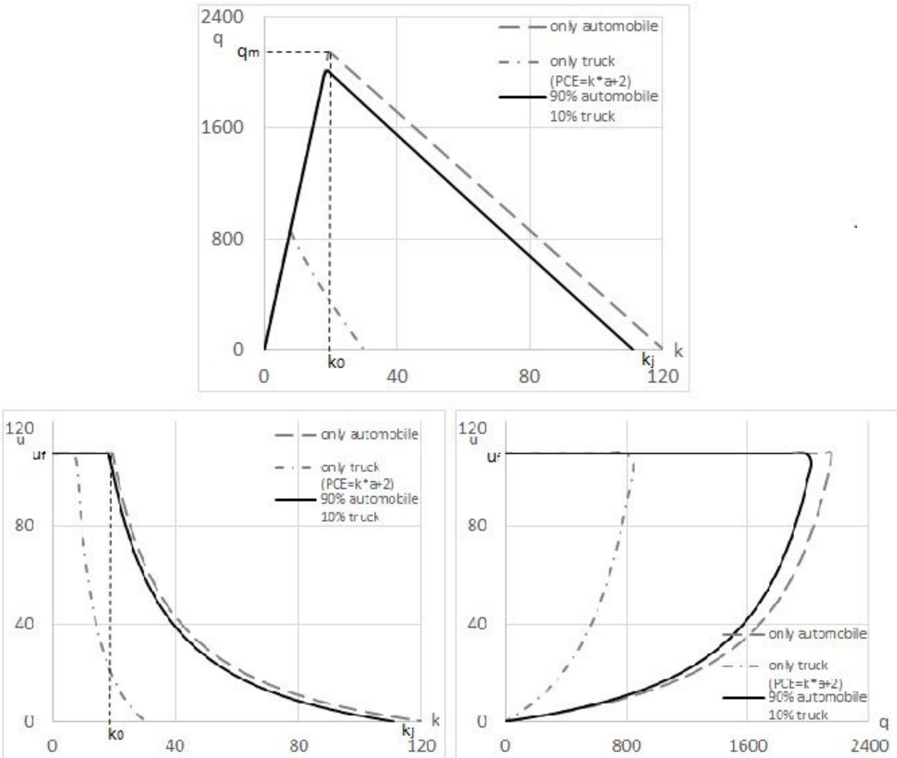
Assume that the traffic flow consists of $x(\le 10\%)$ trucks and $(1-x)$ automobiles, then the ratio of flow rates when using different PCE values PCE_1 and PCE_2 is as follows.

$$
\frac{x\cdot PCE_1+(1-x)}{x\cdot PCE_2+(1-x)} = 1 + \frac{x(PCE_1-PCE_2)}{(1-x)+x\cdot PCE_2} = 1 + \frac{PCE_1-PCE_2}{\frac{1}{x}-1+PCE_2} \tag{3-3}
$$

From the equation, for $1/x \ge 10$ and $\frac{1}{x} - 1 + PCE_2 > 10$, the value of the expression in Equation (3-3) would be near 1. Therefore, it is obvious that using different PCE value of 2 or 4 is not that significant when the truck ratio is small enough.

Then if substitute the linearly increasing PCE value, which changes from 2 in zero-density to 4 in maximum density as in <Figure 3.3>, in Equation (3-2), the traffic flow fundamentals would be like <Figure 3.4>.

This shows how the heterogeneous traffic flow is usually treated when using PCE to calibrate it. It obviously ignored the influence of the trucks to the heterogeneous traffic flow, which contains the following vehicles' long headway, lane changing and their secondary influence on the nearby lanes. And this point will be presented in the following part.



<Figure 3.4> Linear Combination of Heterogeneous Flow

3.2 Characteristics of Heterogeneous Traffic Flow

However, in the heterogeneous traffic flow, the trucks influence the driving behavior of the nearby vehicles, which include deceleration, lane changing of the following vehicles and their secondary influence on the nearby lanes. That is to say, a passenger car which was following a truck would try to decelerate to change its lane, then its target lane would have larger density for the newly came vehicle, and the vehicle which was the following vehicle of the lane changing gap might decelerate when the newcomer’s lane changing happened.

Especially in the ramp-weaving sections, the on- and off-ramp vehicles improve the lane changing frequency. While the trucks, which prefer to travel on the outer lanes, may lead to chaos in this section. Therefore, the influence of the trucks is much more obvious in the ramp-weaving sections.

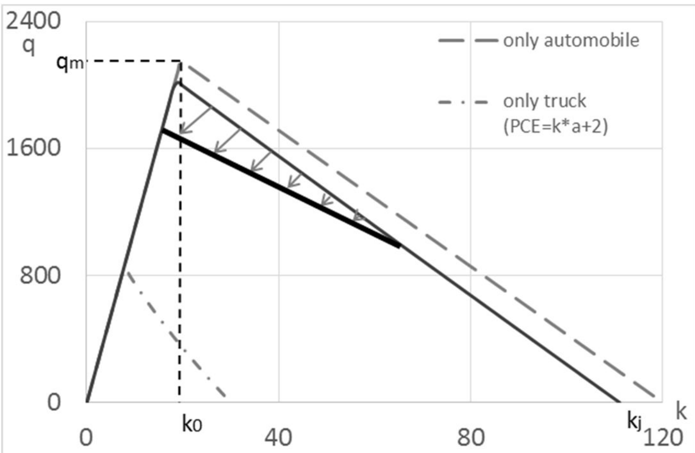
PCE is apparently finite to reflect this phenomenon because it is not just the difference between headways. As a result, the heterogeneous traffic flow fundamentals, which should count in the influence to the close lanes led by the heterogeneity, need to be different from the simple combination as Equation (3-2).

In free flow, for the interactions between vehicles are negligible, the trucks are almost not affected by the traffic flow and also play little influence on the traffic flow. And in traffic jam, the interactions exist just between the leading vehicles and the following vehicles. That is to say, the vehicles almost follow their leading vehicles and have little choices to change their lanes. In this case, the trucks influence the traffic flow due to their sizes, which can be calibrated with PCE. When the traffic flow gets close to the capacity state, the interactions between the vehicles would amplify, and so do the influence of the trucks. Therefore this state is the focus of this research.

Consequently, it can be considered to be equivalent to the average time gap T's increasing because of the heterogeneity of the traffic under influence range (equals transition flow $(k_0,\ k_f)$).

It is different from drastically increasing PCE value. For the different PCE value can decrease the traffic flow rate almost the same as the density varies. However, the influence of trucks happens only in certain range which is near capacity. This would be shown in Chapter 4.

Then in k~q diagram, the raise of the time gap can be presented as the drop of the flow rate (<Figure 3.5>).



<Figure 3.5> Heterogeneous Traffic Fundamental Diagram

Therefore, the new heterogeneous traffic flow fundamental expressions should be contain the heterogeneous traffic flow reduction factor $\alpha(1/s)$ in the transition flow range $\left(k_0{}^N, k_f{}^N\right)$:

$$q^N = \begin{cases} u_f k^N & 0 \le k^N < k_0{}^N \\ \left(\frac{1}{T} - \alpha\right)\left(p - \frac{k^N}{k_j}\right) & k_0{}^N < k^N \le k_f{}^N \\ \frac{1}{T}\left(p - \frac{k^N}{k_j}\right) & k_f{}^N < k^N \le k_j{}^N \end{cases}$$

$$u^N = \begin{cases} u_f & 0 \le k^N < k_0{}^N \\ \left(\frac{1}{T} - \alpha\right)\left(\frac{p}{k^N} - \frac{1}{k_j}\right) & k_0{}^N < k^N \le k_f{}^N \\ \frac{1}{T}\left(\frac{p}{k^N} - \frac{1}{k_j}\right) & k_f{}^N < k^N \le k_j{}^N \end{cases} \tag{3-4}$$

$$or$$

$$u^N = \begin{cases} u_f & 0 \le k^N < k_0{}^N \\ \frac{1}{k_j{}^N}\left(\frac{q^N}{p - \frac{1}{\left(\frac{1}{T} - \alpha\right)} \cdot q^N}\right) & k_0{}^N < k^N \le k_f{}^N \\ \frac{1}{k_j{}^N}\left(\frac{q^N}{p - T \cdot q^N}\right) & k_f{}^N < k^N \le k_j{}^N \end{cases}$$

Where, $\mathrm{p} = \frac{x + (1-x) \cdot PCE}{PCE}$ is the term of converting heterogeneous traffic flow rate to homogeneous with PCE; and x is the ratio of truck in whole traffic flow; the superscript N represents the word "new" so as to differentiate from others.

The reduction factor α is decided by the number of lanes and the truck ratio. The maximum density, the transition flow range and the optimum density are also different from others, which depend on the PCE value and the truck ratio in traffic flow.

To reflect this point of view in a simplified and realistic way is the key issue of this research. It would be shown that considering the heterogeneity is necessary. While it is also important to keep the traffic flow model simple when taking the influence of the heterogeneity into account.

3.3 Model Development

3.3.1 Continuum model

For the changes in expression of the traffic flow fundamental diagrams, the propagation velocities of transitions between different traffic states which can be calculated by LWR model would also change.

According to LWR model (Equation (2-5)), Treiber and Kesting (2013) derived the propagation velocity of kinematic waves $\tilde{c}$ using by the traveling-wave ansatz. The velocity of free flow and congestion, as well as the velocity of transitions from free flow to congestion c_{up} and downstream jam fronts c_{down} can be derived with triangular fundamental diagrams as following equation:

$$
\begin{gathered}
k(x,t) = k_0(x - \tilde{c}t) \\
\begin{cases} c_{free} = \frac{dq_e}{dk}|_{k<k_0} = u_f \\ c_{cong} = \frac{dq_e}{dk}|_{k>k_0} = -\frac{1}{k_j T} \end{cases} \\
\begin{cases} c_{up} = c_{12} = \frac{q_2 - q_1}{k_2 - k_1} = \frac{q_2 - q_1}{k_{max}(1 - q_2 T) - k_1/u_f} \\ c_{down} = -\frac{1}{k_j T} \end{cases}
\end{gathered}
\tag{3-5}
$$

Where, $k_0(x) = k(x,0)$ defines the initial density distribution, which uniformly moves with velocity $\tilde{c}$; c_{free}, c_{cong}, c_{up} and c_{down} present the propagation velocity of free flow and congestion, transitions from free flow to congestion, and downstream jam fronts, respectively.

For the modified fundamental relationship of the heterogeneous traffic flow, the propagation velocities should be changed.

$$\begin{cases} c_{free}{}^{N} = \frac{dq_e}{dk}|_{k^N<k_0{}^N} = u_f \\ c_{cong}{}^{N} = \frac{dq_e}{dk}|_{k^N>k_0{}^N} = -\frac{1}{k_j\frac{T}{1-T\cdot\alpha}} \end{cases}$$
$$\begin{cases} c_{up}{}^{N} = c_{12} = \frac{q_2-q_1}{k_2-k_1} = \frac{q_2-q_1}{k_{max}(1-q_2T)-k_1/u_f} \\ c_{down}{}^{N} = -\frac{1}{k_jT} \end{cases} \tag{3-6}$$

3.3.2 Model summary

To sum up, the developed heterogeneous traffic flow model is reorganized as follows:

$$q^N = \begin{cases} u_f k^N & 0 \le k^N < k_0{}^N \\ \left(\frac{1}{T}-\alpha\right)\left(p-\frac{k^N}{k_j}\right) & k_0{}^N < k^N \le k_f{}^N \\ \frac{1}{T}\left(p-\frac{k^N}{k_j}\right) & k_f{}^N < k^N \le k_j{}^N \end{cases}$$
$$u^N = \begin{cases} u_f & 0 \le k^N < k_0{}^N \\ \left(\frac{1}{T}-\alpha\right)\left(\frac{p}{k^N}-\frac{1}{k_j}\right) & k_0{}^N < k^N \le k_f{}^N \\ \frac{1}{T}\left(\frac{p}{k^N}-\frac{1}{k_j}\right) & k_f{}^N < k^N \le k_j{}^N \end{cases}$$
$$or$$
$$u^N = \begin{cases} u_f & 0 \le k^N < k_0{}^N \\ \frac{1}{k_j{}^N}\left(\frac{q^N}{p-\frac{1}{\left(\frac{1}{T}-\alpha\right)}\cdot q^N}\right) & k_0{}^N < k^N \le k_f{}^N \\ \frac{1}{k_j{}^N}\left(\frac{q^N}{p-T\cdot q^N}\right) & k_f{}^N < k^N \le k_j{}^N \end{cases} \tag{3-7}$$
$$\begin{cases} c_{free}{}^{N} = \frac{dq_e}{dk}|_{k^N<k_0{}^N} = u_f \\ c_{cong}{}^{N} = \frac{dq_e}{dk}|_{k^N>k_0{}^N} = -\frac{1}{k_j\frac{T}{1-T\cdot\alpha}} \end{cases}$$
$$\begin{cases} c_{up}{}^{N} = c_{12} = \frac{q_2-q_1}{k_2-k_1} = \frac{q_2-q_1}{k_{max}(1-q_2T)-k_1/u_f} \\ c_{down}{}^{N} = -\frac{1}{k_jT} \end{cases}$$

Chapter 4. Model Verification

To present the theoretical heterogeneous traffic flow more suitable to the actual conditions, this research developed the traffic flow model by combining both the theory and the field observation.

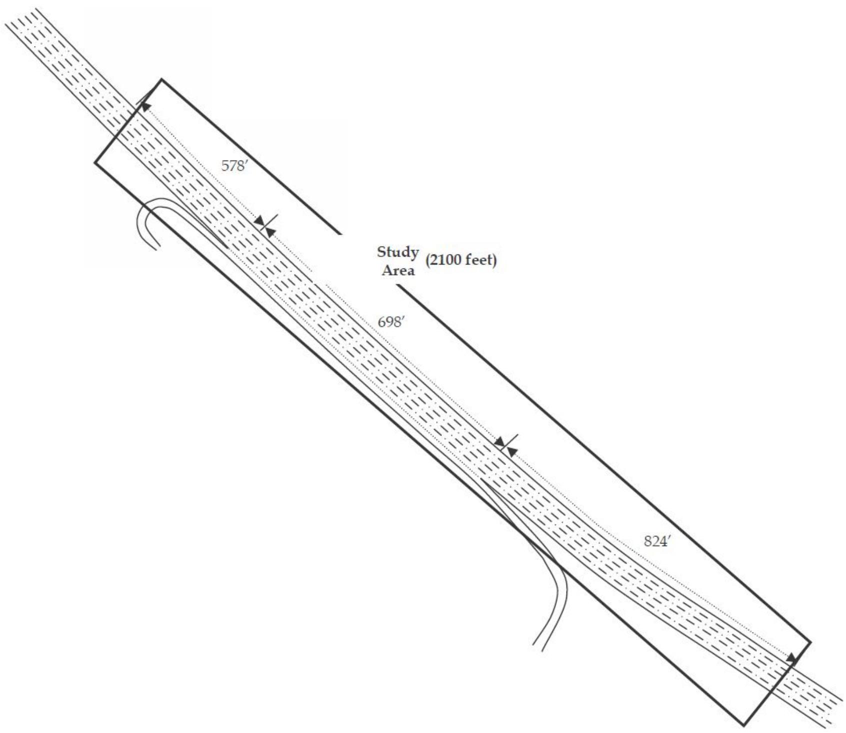
4.1 Data Construction

To present the characteristics of the multi-class vehicles, this paper used the data with the trajectories of the classified vehicles near on- and off-ramps. The data used for the analysis is "NGSIM U.S. 101 Data", which is a dataset of vehicle trajectory data on a segment of U.S. Highway 101 completed as part of the Federal Highway Administration's (FHWA) Next Generation Simulation (NGSIM) project. The data analyzed in this research is collected from 7:50a.m. to 8:05a.m. on June 15th, 2005.

The geometric structure of the study area is as shown in <Figure 3.5>. It can be divided into three sections: before on-ramp section, ramp-weaving section, and after off-ramp section. The number of lane is 5 and one more lane is added on the ramp-weaving section. The total length of the study area is 640.1m (2100ft) and the six-lane's ramp-weaving section is 212.8m (698ft) long. The lengths of the before on-ramp section and the after off-ramp section are 176.2m, 251.2m respectively.

The average traffic flow rate is 8612veh/h and the average space mean speed is 25.66mi/h. The vehicles consist of 96.2% of automobile, 2.4% of truck and buses, and 1.4% of motorcycle. (Here the truck and buses amounts to the class “truck”.) All of the trucks entered from the main stream and 1 truck got out through the off-ramp.

The data, which is aggregated every 0.1 second, consists of vehicle ID, frame ID, global and local XY coordinates, vehicle length, width, class, velocity, lane, acceleration, preceding vehicle, following vehicle, space headway and time headway.



<Figure 4.1> Study Area[3]

[3] Cambridge Systematics, Inc. NGSIM U.S. 101 Data Analysis summary report.

4.2 Data Analysis

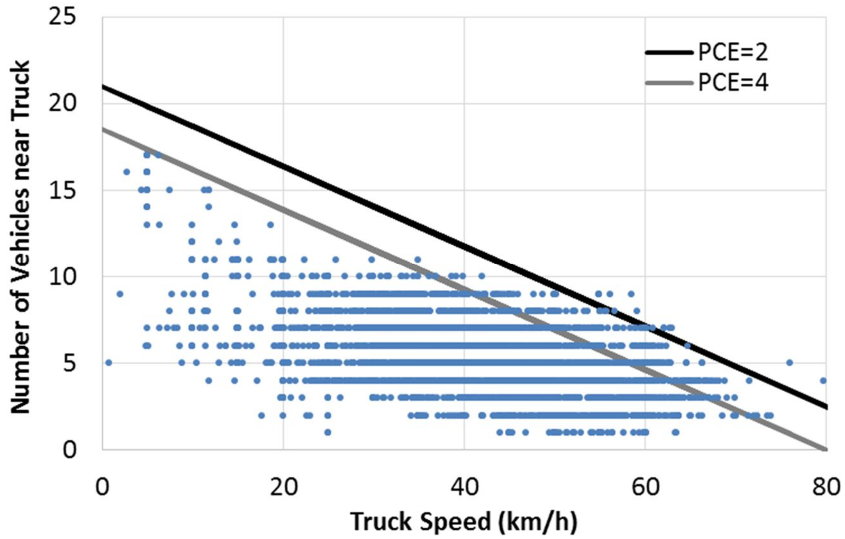
4.2.1 Microscopic influence of heterogeneity

In microscopic view point, the influence of heterogeneity, or replace as trucks distinguishing from homogeneous automobiles, can be shown through the number of the vehicles in a certain area around a truck. Because of the reasons mentioned above, the distributions of vehicles around an automobile and a truck would different.

To make the term "near the truck" more exact, the following vehicles of some sample truck were tracked for the positions of lane-changing. And then it is concluded that all of the positions of lane-changing happened within 30 meters' distance. Therefore, "near the truck" is defined as the rectangular area of $60\mathrm{m} \times 3\mathrm{lanes}$ with a truck posited in its center.

By plotting the aggregated number of the vehicles near the trucks with the speed of the trucks, <Figure 4.2> is drawn. It can be compared with the theoretical value of using PCE=2 and PCE=4. Then we can see the numbers almost linearly decrease with the speed increases respectively in free flow and congestion flow, but unregularly drop under the transition flow state ($\mathrm{u} = 30{\sim}50\mathrm{km/h}$).

This proves the point of view that PCE is finite to reflect the influence of the truck. As a result of the heterogeneity influence to the traffic flow, it can be considered to the average time gap T's increasing.

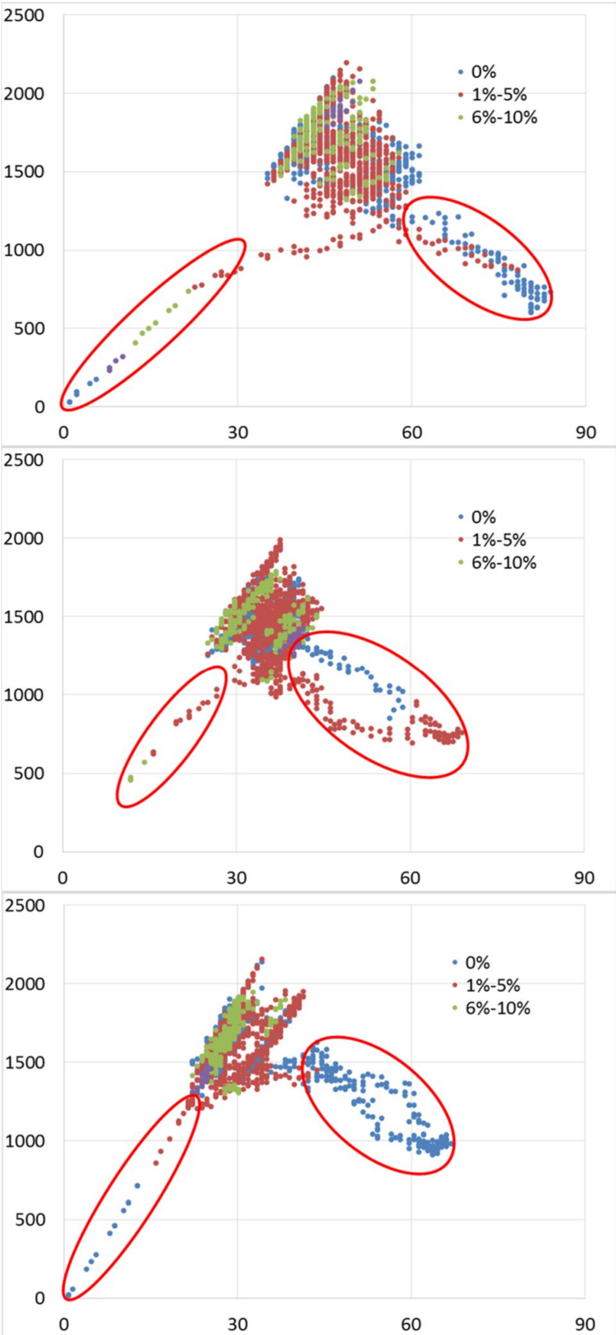


<Figure 4.2> Distribution of Vehicles Numbers near Truck

4.2.2 Macroscopic influence of heterogeneity

To get the macroscopic traffic flow model, the next step is to plot the $k \sim q$ points for the three sections as <Figure 4.3> (the horizontal axis is the speed; the vertical axis is traffic flow rate). The $k \sim q$ points are aggregated every 0.5 second. For the limitation of the data size, the percentages of the trucks are categorized as 0%, 1%-5%, 6%-10% and so on.

It can be observed easily that the capacity region (breakpoint region) occupies most of the points. However, the purpose of this research is to draw mathematical expressions of the relationship of the traffic flow properties. So the red circled parts which include the free flow and congestion conditions are the foci.



a) Before on-ramp section

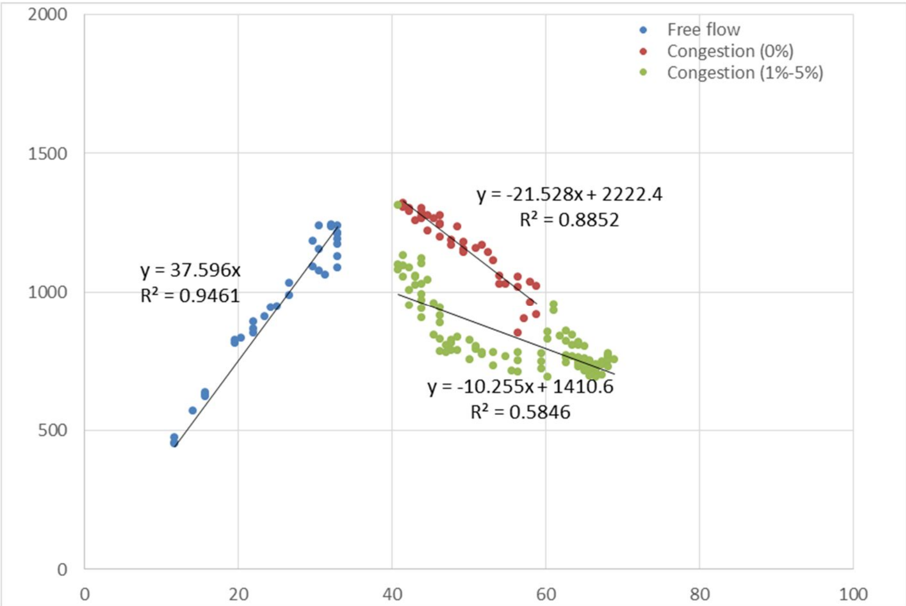
b) Ramp-weaving section

c) After off-ramp section

<Figure 4.3> k~q Plot with All Data for Three Sections

From these figures, we can find that the most obvious region which is sensitive to the trucks is the ramp-weaving section. This matches the point of view which is proposed in the part of model concept.

Therefore, the red circled parts of the k~q plot on the ramp-weaving section with the capacity condition eliminated are enlarged for the linear regression as <Figure 4.4>.



<Figure 4.4> q~k Plot on Ramp-weaving Section without Capacity State

From <Figure 4.4> and its linear relation functions, it can be observed that the distribution is obviously different between the points without trucks and those with trucks. It shows the average time gap T's increasing and the capacity reduction of the heterogeneous traffic when compared to the homogeneous traffic.

This $q \sim k$ relationship of the homogeneous and the heterogeneous traffic flow can be expressed respectively as follows:

$$q^N = \begin{cases} 52.3k^N & 0 \leq k^N < 30.1\ (0\%) \\ -21.5k^N + 222.4 & 30.1 < k^N \leq 103.4 (0\%) \\ & or \\ 52.3k^N & 0 \leq k^N < 22.5(1-5\%) \\ -10.3k^N + 1410.6 & 22.5 < k^N \leq 72.5(1-5\%) \\ -21.5k^N + 222.4 & 72.5 < k^N \leq 103.4(1-5\%) \end{cases} \qquad (4\text{-}1)$$

Therefore, we got the traffic flow reduction factor $\alpha = 775.63/\mathrm{s}$.

However, the focus of many traffic managements is to find out the break point so as to control the traffic flow under the breakpoint and get the flow rate at the maximum value.

While from the equation we can get that the breakpoint falls down from $\mathrm{k} = 30.1\mathrm{veh/km} \cdot \mathrm{lane}, \mathrm{q} = 1574.8\mathrm{veh/h} \cdot \mathrm{lane}$ (in homogeneous flow) to $\mathrm{k} = 22.5\mathrm{veh/km} \cdot \mathrm{lane}, \mathrm{q} = 1178.3\mathrm{veh/h} \cdot \mathrm{lane}$ (in heterogeneous flow) drastically.

This significant drop of the breakpoint indicates the risk of using the homogeneous model to the heterogeneous traffic flow on the ramp-weaving sections. If the ramp metering rates are calculated according to the homogeneous model in spite of the heterogeneous traffic, the redundant input flow rate would result in congestions on the ramp-weaving sections.

Finally, the total heterogeneous traffic flow model can be concreted for “NGSIM U.S. 101 Data” as following equation (in which “homo” is short for homogeneous traffic flow and “heter” is short for heterogeneous traffic flow).

$$q^N = \begin{cases} 52.3k^N & 0 \le k^N < 30.1 \\ -21.5k^N + 222.4 & 30.1 < k^N \le 103.4 \end{cases} (homo) \\ or \\ \begin{cases} 52.3k^N & 0 \le k^N < 22.5 \\ -10.3k^N + 1410.6 & 22.5 < k^N \le 72.5 \\ -21.5k^N + 222.4 & 72.5 < k^N \le 103.4 \end{cases} (heter)$$

$$u^N = \begin{cases} 52.3 & 0 \le k^N < 30.1 \\ -21.5 + 222.4/k^N & 30.1 < k^N \le 103.4 \end{cases} (homo) \\ or \\ \begin{cases} 52.3 & 0 \le k^N < 22.5 \\ -10.3 + 1410.6/k^N & 22.5 < k^N \le 72.5 \\ -21.5 + 222.4/k^N & 72.5 < k^N \le 103.4 \end{cases} (heter) \quad (4\text{-}2)$$

$$c_{free}{}^N = 52.3$$

$$c_{cong}{}^N = \begin{cases} -21.5 & 30.1 < k^N \le 103.4 \quad (homo) \\ & or \\ -10.3 & 22.5 < k^N \le 72.5 \\ -21.5 & 72.5 < k^N \le 103.4 \end{cases} (heter)$$

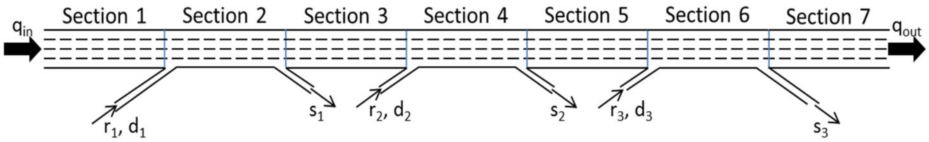
Chapter 5. Model Application to Ramp Metering

To verify the developed model, this part presents the model application to ramp metering. Ramp metering, as a branch of the dynamic traffic management, uses the traffic signals at on-ramps of freeways to control the entering rate of vehicles. Ramp metering has been claimed to be efficient in reducing traffic congestion by controlling the traffic flow rate under capacity.

5.1 Ramp Metering Network Information

In order to implement a ramp metering strategy, a network is essential. This research constructed a random network and its time-dependent flow rate and demand table (<Table 4.1>). Before applying the model to ramp metering, the basic assumptions of the network are presented.

The network is divided by 7 sections, with 3 pairs of on- and off-ramps in 2^{nd}, 4^{th}, and 6^{th} Section (see <Figure 4.1>). And the lengths of these sections are set as 700m, 500m, 600m, 750m, 650m, 600m, and 750m respectively. There are 4 lanes all along the freeway and the acceleration lanes are ignored. The average free flow speed of freeway is set as 52.3km/h, which is derived from the real detected data in Chapter 4.



<Figure 5.1> Geographic Structure of Network

All of the traffic inflow rates and the demand of the on-ramps are time-dependent, but assumed to stay the same in one period. In the picture, $q_i, i = 1 \sim 6$ is the traffic flow in Section i, $r_i, i = 1,2,3$ means ramp metering rate, $d_i, i = 1,2,3$ means the on-ramp demand, and $s_i, i = 1,2,3$ means the off-ramp flow rate. The ratio of truck in traffic flow is fixed as 5% all the time.

<Table 5.1> Traffic Inflow and On-ramp Demand Rate Table

T	Inflow(veh/h)	On-ramp demand(veh/h)			Off-ramp flow(veh/h)		
	q_{in}	d_1	d_2	d_3	s_1	s_1	s_1
0	2000	560	650	450	600	550	500
1	3200	500	540	400	550	600	550
2	4700	800	570	560	580	590	500
3	4000	700	600	590	550	560	530
4	3000	750	570	570	600	550	550
5	2000	800	610	580	580	570	550
6	4500	780	600	620	620	560	560
7	3000	700	580	600	590	560	570

8	2500	600	560	570	570	550	550
9	2500	800	570	580	580	570	570
10	4000	700	550	580	590	580	570
11	4500	700	560	550	680	570	560
12	2000	560	650	450	600	550	500

Period $T = 0$ means that the network had been arrived in a steady state before the inflow rate and the demand vary. And every period T equals to 5 minutes, therefore, the total ramp metering time is one hour.

5.2 Ramp Metering Optimization

To control the ramp metering efficiently, different traffic flow models are adopted to provide different capacities of freeways, in which is very important to use suitable traffic flow models to predict the real time traffic condition and avoid congestions.

On the other hand, using proper ramp metering strategy is another big part in ramp metering management. There have been developed a variety of ramp metering strategies. And the time-of-day control strategy considering the evolution of traffic flow would be used in this research.

As to the time-of day control, the early linear programming approach of optimizing time-of-day control strategy was developed by Wattleworth (Wattleworth and Berry, 1965; Wattleworth, 1967). Then Papageorgiou proposed a modified strategy considering the evolution of traffic flow according to the time delay between a flow rate change at a ramp and a downstream point of freeway.[13]

The strategy applied here is to seek the optimized ramp metering rate which maximizes the input ramp metering flow under the capacity constraint (Equation (5-1)). And to show the effect of the developed heterogeneous traffic flow model, both the homogeneous and the heterogeneous traffic flow model are applied respectively. Their main difference in ramp metering is the capacity of the ramp-weaving sections.

$$\begin{aligned} &\max \textstyle\sum_{i=1,2,3} r_i(t) \\ s.t.\ & \\ &r_i(t) \le d_i(t) + \textstyle\sum_{j=0}^{t-1}\left(d_i(j) - r_i(j)\right) \\ &q_i(t) \le q_{i\_max} \\ &r_i(t) \ge o \end{aligned} \tag{5-1}$$

To get more sensible results, the sections and the time step are discretized. The sections are divided into 100 meters' segments, and the time step is set as 5 seconds. And the segments are counted in $1,2,3,\cdots$ from upstream to downstream respectively.

For the traffic flow rate $q_{ij}(t)$ of the segment j in section i, its values are obtained according to the inflow rate, the on-ramp rate and off-ramp rate at the upstream, and their arrival time to that segment. Therefore, when the inflow rate, the ramp metering rate and the off-ramp flow change as the period T changes, the traffic flow rate $q_{ij}(t)$ would not change immediately. It takes some time to get a new balance.

Take the 2nd segment in 3rd section as example. The inflow rate $q_{in}(T-1), q_{in}(T)$, the ramp metering rate $r_1(T-1), r_1(T)$, the off-ramp flow $s_1(T-1), s_1(T)$ and their travel time of the traffic flow t_{in-32}, t_{1r-32}, t_{1s-32} are known. Then the flow rate $q_{32}(t)$ changes with the change of the inflow rate, the ramp metering rate and the off-ramp flow.

$$q_{32}(t) = \begin{cases} q_{in}(T-1) + r_1(T-1) - s_1(T-1), & T \le t < T + t_{1s-32} \\ q_{in}(T-1) + r_1(T-1) - s_1(T), & T + t_{1s-32} \le t < T + t_{1r-32} \\ q_{in}(T-1) + r_1(T) - s_1(T), & T + t_{1r-32} \le t < T + t_{in-32} \\ q_{in}(T) + r_1(T) - s_1(T), & T + t_{in-32} \le t \end{cases} \tag{5-2}$$

Then the optimized ramp metering rate solutions for the two scenarios are obtained as <Table 5.2>. The total input ramp metering rates in the two scenarios are 1833veh/h and 1128veh/h.

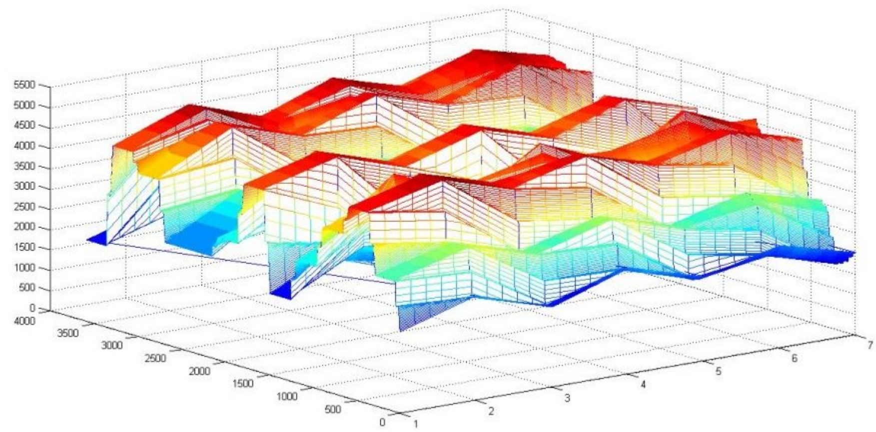
<Table 5.2> Optimized Ramp Metering Rate

T	Scenario 1: Homogeneous			Scenario 2: Heterogeneous		
	r_1	r_2	r_3	r_1	r_2	r_3
0	560	650	450	560	650	450
1	500	540	248	500	514	114
2	456	467	448	13	314	314
3	456	267	448	13	114	314
4	957	667	648	514	514	514
5	957	667	648	514	514	514
6	474	576	602	31	424	469
7	457	567	548	14	414	414
8	957	667	648	514	514	514
9	957	667	648	514	514	514
10	957	667	648	514	514	514
11	700	560	571	213	362	438
12	560	650	548	14	414	414

i. Scenario 1: Homogeneous traffic flow model

When the traffic flow is regarded to be homogeneous, the capacities of the 6 sections are set as the same value 6299veh/h. Then the ramp metering rate is as <Table 5.2>, the traffic flow rate of the network over the whole time is as <Figure 5.2>.

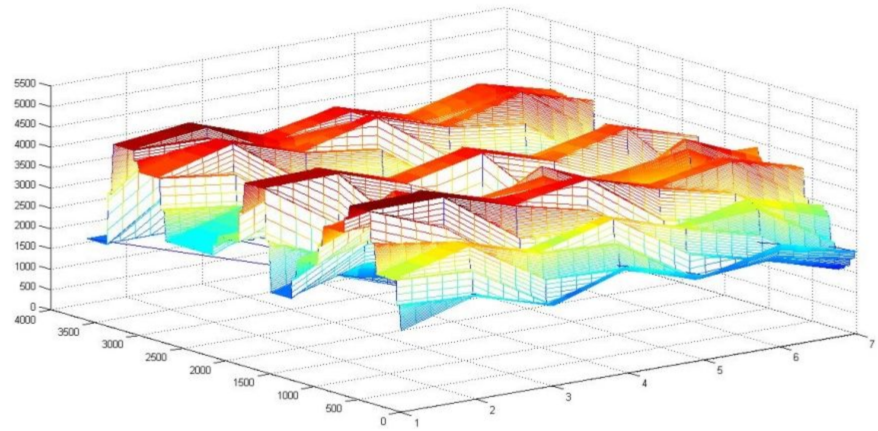
(In <Figure 5.2> and <Figure 5.3>, the horizontal axis on the right is the section order number, the vertical axis on the left is the time axis with unit in second, and the erect axis represents the traffic flow rate in veh/h unit.)



<Figure 5.2> Traffic Flow Rate Using Homogeneous Model

ii. Scenario 2: Heterogeneous traffic flow model

When the traffic flow is regarded as heterogeneity and applies the modified model in this research, the capacities of the 3 ramp-weaving sections are set as 4713veh/h, and the other sections' are still 6299veh/h. The optimized ramp metering rate is as <Table 5.2>, the traffic flow rate of the total network is as <Figure 5.3>



<Figure 5.3> Traffic Flow Rate Using Heterogeneous Model

By comparing these two traffic flow rate space-time diagram, it can be easily seen that the traffic flow rate under the heterogeneous model is lower than that under the homogeneous model, especially in the ramp-weaving sections. And this is consequent result.

This is not enough to show the necessity of the heterogeneous traffic flow model. It only seems to be a conservative model to keep the ramp metering rate down so that the total ramp metering rate is only 62% of the demand. Therefore, the traffic state prediction is followed.

5.3 Traffic State Prediction

5.3.1 Prediction set up

We got a higher ramp metering set with the homogeneous traffic flow model in the previous part. It means that the ramp metering rate will follow this result in ramp metering controls which regard the traffic flow as homogeneous. While the reality is, the blended trucks would pull the capacity down in the ramp-weaving sections. Then the input of the ramp-weaving sections turns out to exceed its capacity and the congestion comes into being.

Therefore, the traffic state is predicted using the heterogeneous model and the extended LWR model, with the ramp metering rate which is obtained from the homogeneous model.

The input flow rate of the main stream is as <Table 5.1> and the ramp metering rate is as the part of Scenario 1 in <Table 5.2>. The developed heterogeneous traffic flow model in Chapter 3 is applied to predict the traffic state.

To manifest the point we want to emphasize, the traffic state in Section 1 and 2 is predicted as representatives. The prediction is implemented for the time period of the first traffic flow peak (0~1500s).

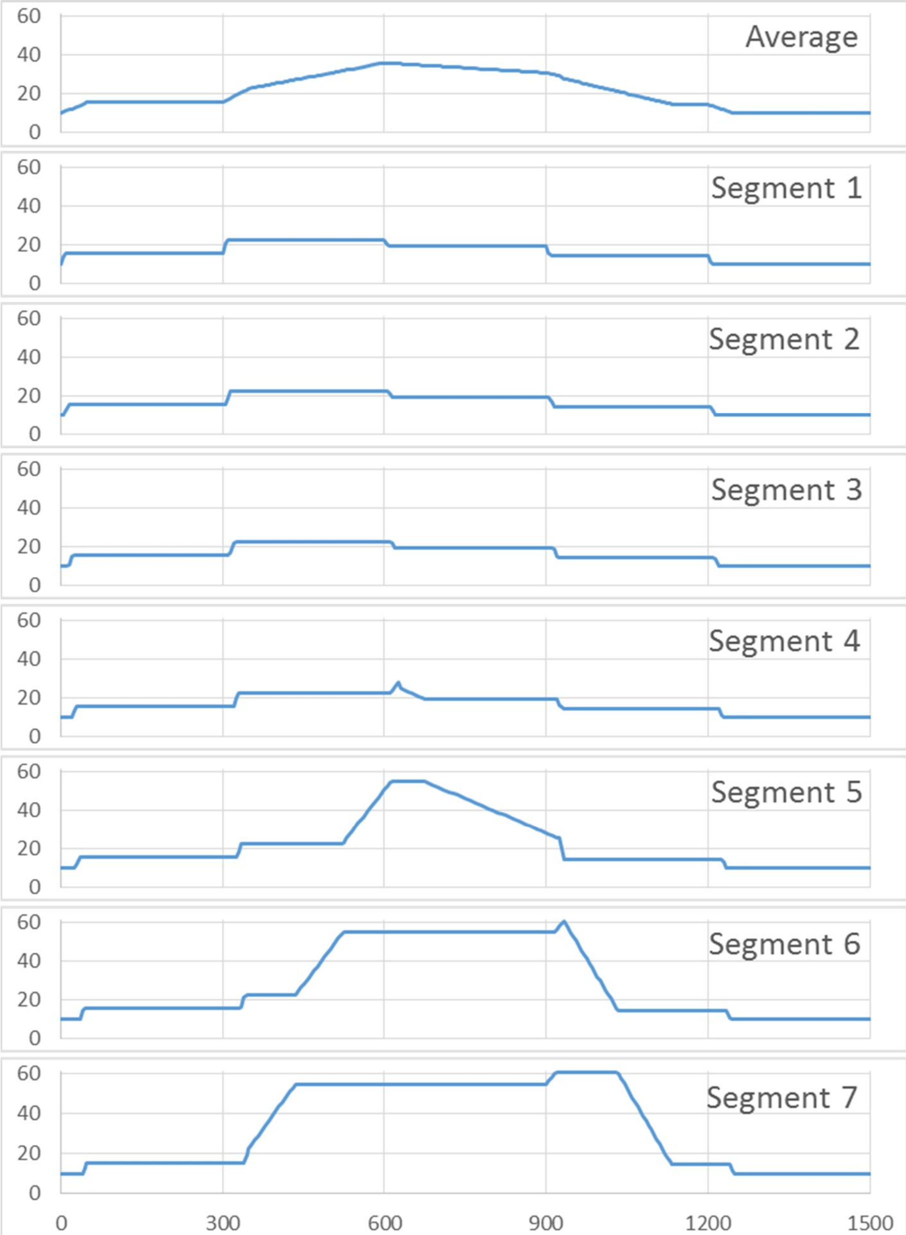
5.3.2 Result analysis

By using the heterogeneous traffic flow model, the traffic state is predicted as <Figure 5.4> ~ <Figure 5.7>. The time-serial plotting figures directly show how the traffic density (<Figure 5.4>, <Figure 5.5>) and the speed (<Figure 5.6>, <Figure 5.7>) vary. The predicted traffic density in Section 1 increases over the breakpoint $k_0 = 30.1veh/km \cdot lane$ during about $500\sim905s$, in Section 2 increases over the breakpoint ${k_0}^N = 22.5veh/km \cdot lane$ during about $385\sim1250s$. And the flow rate is even lower than in the case of using the heterogeneous model, which contains strong constraint.

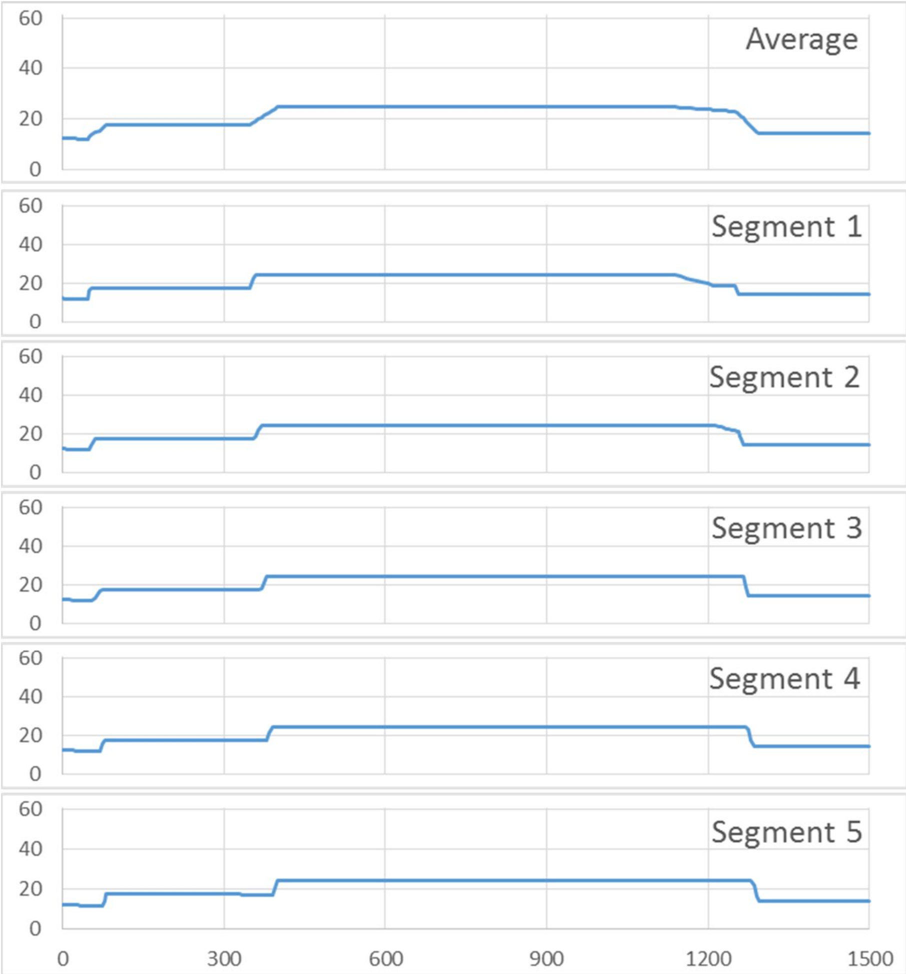
The figures are showing a slight congestion in Section 2, but a serious congestion in Section 1 including considerable speed drop. Since the ramp-weaving Section 2 became the bottleneck in the network, the down-flow in Section 1 frequently faced congestions.

This means that a slight congestion happens in ramp-weaving section if the optimized ramp metering rate with the homogeneous model is applied. But more critically, a serious congestion happens in the sections before on-ramp.

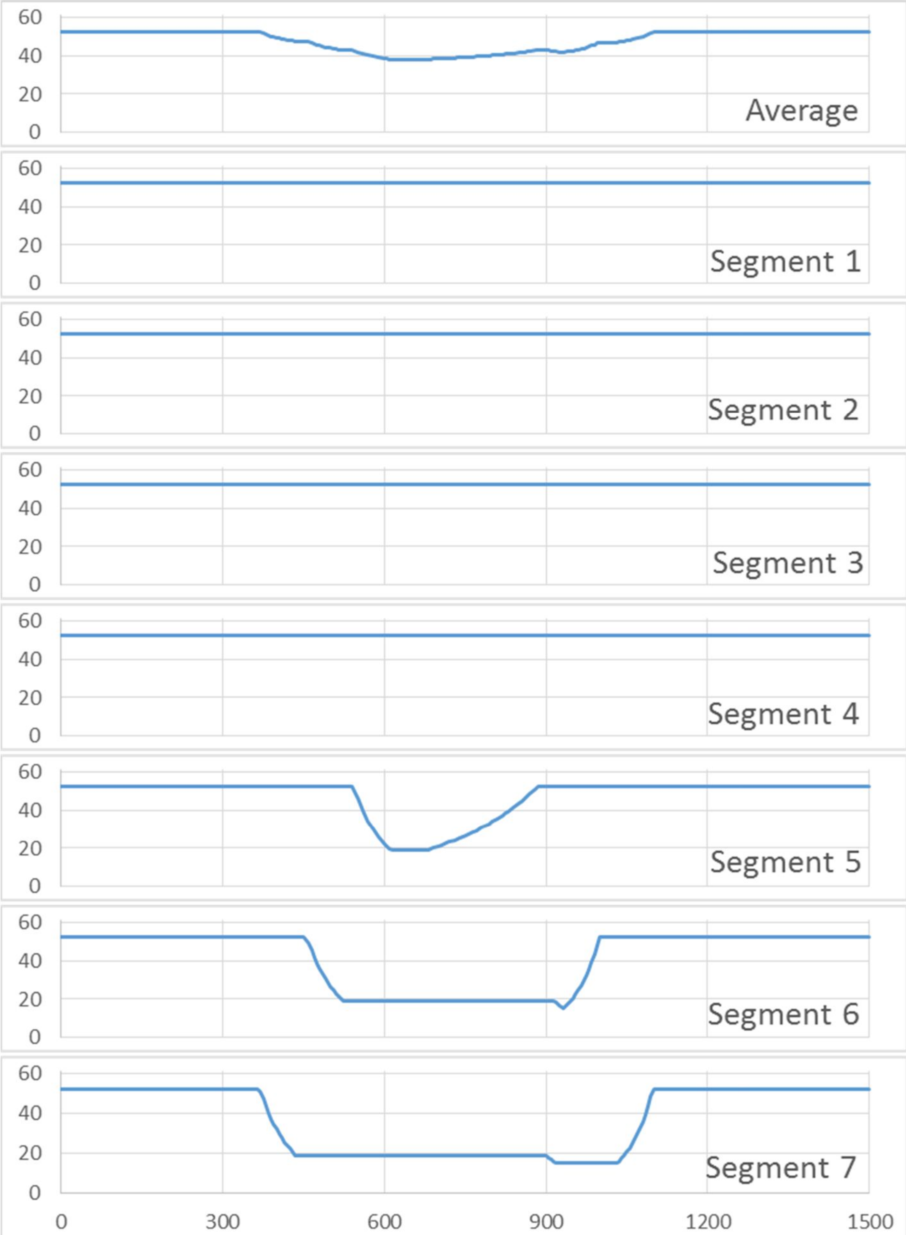
As mentioned above, the speed falls down from the free flow speed at time 348s, as soon as the density increases and the flow rate decreases. The predicted speed decreased by 10.3% during 348~900 second and 21.0% during 900~1006 second. That is to say, the traffic state didn't recover until time 1006s. And the congestion lasted over 10 minutes and the range of influence on before on-ramp section is as long as 300 meters.



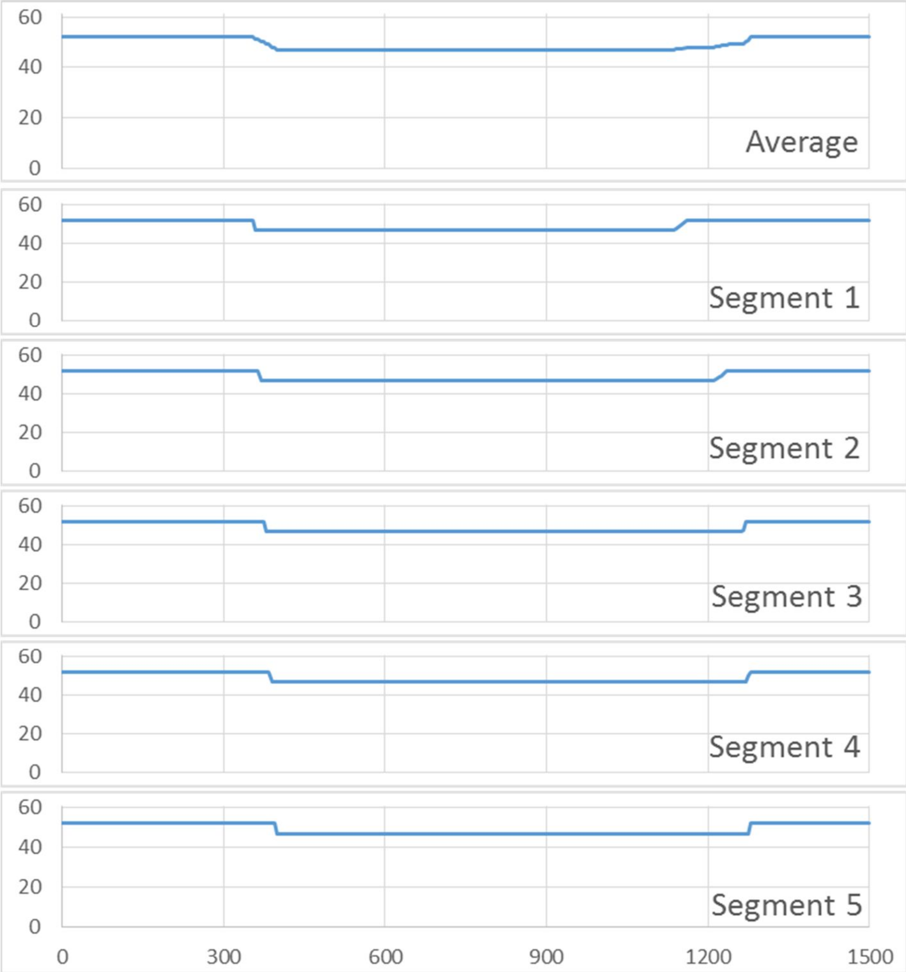
<Figure 5.4> Traffic Density Prediction in Section 1



<Figure 5.5> Traffic Density Prediction in Section 2



<Figure 5.6> Traffic Speed Prediction in Section 1



<Figure 5.7> Traffic Speed Prediction in Section 2

This simple prediction shows the risk of the usage of the homogeneous model to ramp meter. The homogeneous traffic flow model, which ignored the characteristics of the different vehicle type, usually over-valuates the capacity of some certain freeway regions like ramp-weaving sections. This leads to a high ramp metering rate, and then the ramp-weaving section faces congestions. It will result in worse traffic conditions, including lower speed, smaller flow rate and so on, to the whole network in return.

However, the usage of the heterogeneous model to the ramp weaving sections would avoid this risk. Though the stronger constraint obtains lower ramp metering rate, it guarantees the free flow of the network.

On the other hand, pulling down the capacity blindly also has fatal disadvantages. It decreases the efficiency of the freeway system and could not be considered as optimized management.

Chapter 6. Conclusions

6.1 Conclusions

This research has developed the heterogeneous traffic flow model for the ramp metering freeways. Unlike most of the traffic flow models, which ignore or simplify the heterogeneity effects, the modified model took the multi-class vehicles' influence on the traffic flow into account.

The model is developed theoretically at first. Then the characteristics of the heterogeneous traffic flow were extracted from NGSIM data to shape the model. The modified model is then applied to ramp metering and traffic state prediction.

Finally, several conclusions are drawn as follows:

i. The characteristic of the truck influence to traffic flow is reflected as the increase on the mean time gap T, especially in the ramp-weaving sections;

ii. This leads to a falling down of the breakpoint, which is equivalent to the decrease in capacity;

iii. The usage of the homogeneous model contains the risk of congestions on the ramp-weaving sections;

iv. The most serious risk of using the homogeneous model is the congestions on the sections before on-ramps because of the bottleneck;

v. It is necessary to use the heterogeneous traffic flow model which considers the influence of the heterogeneity.

It has been roughly proved that there is necessary to use traffic flow models which consider the heterogeneity of the traffic flow. And the benefit of the heterogeneous traffic flow model will come to light with its development.

6.2 Further Study

There are some further study topics which haven't been researched because of the current limitation but are meaningful and possible to be studied in the future.

i. Collect more data with vehicle types distinguished, with wider range of the truck ratio the heterogeneous traffic flow model can be extended to different truck ratios;

ii. We can solve the ramp metering optimization under the different traffic flow rate on mainstream or the different on-ramp demands, and then make the sensitivity analysis;

iii. The modified traffic flow model can be used to travel time prediction, which may find out those unknown noises when predicting the travel time;

iv. If the functions of the ramp metering equipment allow, the dynamic ramp metering can be implemented in the field according to the homogeneous model and the heterogeneous model in order to compare and analyze the model more credibly.

Traffic flow theory, as a basic learning in transportation engineering, its development decides the growth of transportation engineering. To improve and perfect the traffic flow theory is to contribute to transportation engineering.

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국문초록

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고속도로의 지체는 이미 수년간 많은 연구에서 다루어졌다. 최근 실시간 교통 상황에 따라 고속도로를 제어 및 관리하는 기술인 동적 교통류 관리(Dynamic Traffic Management)가 주요 관심사로 떠오르고 있다. 그러나 실제로 동적 교통류 관리 시스템들은 교통류의 차종 구성을 고려하지 않거나 간소화한 교통류 모델을 사용하여 운영된다.

특히 램프 구간에서 교통류 구성을 반영하는 모델의 적용은 더욱 중요하다. 램프 진입부와 진출부 근처의 빈번한 차선변경으로 인해 이 구간들은 고속도로의 병목구간이 되는데 바깥차선을 선호하는 트럭들은 이런 엇갈림구간의 용량을 더 떨어뜨린다. 이런 현상의 반영은 교통류 모델의 용량 산정, 나아가서 램프 미터링의 운영 결과에 있어서 중요한 요인이 된다.

그러므로 엇갈림구간에서 차종 구분을 고려한 이질적 교통류 모형을 개발하는 것이 본 연구의 초점이다. 본 연구의 주요한 특성 중 하나는 트럭의 주변, 즉 앞·뒤 공간 및 좌·우 차선을 포함한 구역에 미치는 영향을 규명하는 것이다. 트럭의 이러한 영향은 최종적으로 일정한 교통류 상태에서의 평균 시간 갭 T의 증가로 개발된 모형에 반영 되었다.

이질적 교통류의 특성은 "NGSIM U.S. 101 Data"를 통해 검증되었다. 더불어 구체화된 이질적 교통류 모형은 램프 미터링에 적용되어 그 필요성이 검증되었으며 차종 구분을 하지 않은 모형의 부정확성도 증명 되었다. 결론적으로 차종 구분을 하지 않은 모형을 적용하여 얻은 램프 통과 교통량으로 교통류 상태를 예측 해 보면 엇갈림구간에 가벼운 교통 지체가 생기는 것을 관찰할 수 있다. 그러나 보다 심각한 것은 램프 진입부 상류부 구간에 심한 지체가 생기는 것이다.

본 연구에서는 차종을 구분한 이질적 교통류 모형을 개발하고 필요성을 증명하였다. 이질적 교통류 모형의 보다 완벽한 발전은 교통공학 영역에 더 큰 혜택을 가져다 줄 것으로 예상된다.

주요어: 이질적 교통류, 엇갈림구간, 거시적 교통류모형,
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