

CHAPTER 1

Introduction

There are many indications of the complexity of living in today's world. The road-using behavior is one of them. As the trend of increasing travel demand, planning, design, prediction, control and management of transportation system become more and more important, traffic flow theory and traffic control strategies provide description of the fundamental traffic flow characteristics and analytical and control techniques. This new science has addressed issues related to the understanding of traffic processes and optimization of these processes through proper design and control. The former issues could be described as basic research and the latter as applied research. Dynamic traffic management and control is an efficient tool to solve traffic congestion. Among the research topics of dynamic traffic management and control, dynamic traffic flow theory and traffic control strategies are important parts. Therefore, a new microscopic equation model and dynamic traffic control strategy are proposed to describe the ramp/loop-ramp metering control which two lane multi-class traffic at separated connecting collector-distributor (C-D) roadway on freeway interchanges in this study. In the remainder of this introduction, we will discuss motivations, objectives, approaches, and overview of the research in detail. The research motivations will be presented first.

1.1 Research Motivations

The worsening of traffic congestion in developed countries causes lots of economical loss every year. Thus, transportation and traffic management and control becomes a wide research topic. In the evolution of transportation and traffic research, dynamic traffic management and control seems to be a feasible way to relieve congestion. According to the types of data collection and applications, dynamic traffic control researches can be classified to three categories; that is, dynamic capacity estimation, dynamic traffic flow theory and dynamic traffic control strategies. A complete dynamic traffic management and control includes three kernels; the relation between them is illustrated in Figure 1.1. Since the whole procedure is a cycle, each part can be the initial step. If the dynamic traffic flow of each roadway in the network is known, the dynamic capacity will generate the dynamic capacity estimation models. If the dynamic capacity of each roadway and the traffic density and speed are known, the dynamic assignment control model will obtain the control signal timing. Therefore, dynamic traffic flow theory is an important part of dynamic traffic management and control. How to develop the dynamic traffic flow theory at the weaving section on the freeway? First, we must know the driver's behaviors and characteristics of the traffic flow on the weaving sections. Second, the fundamental analysis of traffic flow characteristics and impact variables will be used; those analysis theories include the car-following, lane-changing, and critical lag acceptance theory, etc.; those variables

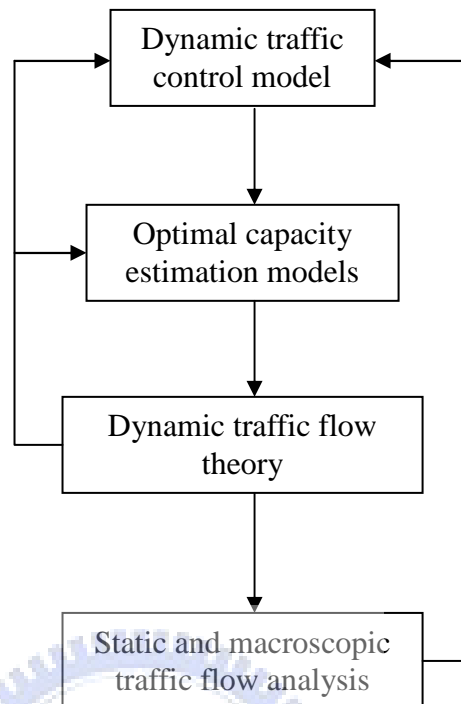


Figure 1.1 The architecture of traffic management and control procedure

include traffic speed, density and flow rate, etc. The location of capacity analysis is focused on the separated connecting C-D roadway weaving section, and the location of ramp metering control is focused on the semi-cloverleaf interchange, respectively.

Lertworawanich and Elefteriadou (2003) pointed out that no methods are available for estimating freeway weaving capacity and performing analysis establishing the capacity models on weaving sections, that more than all the analysis of capacity of the separated connecting collector-distributor (C-D) roadway weaving sections are very deficient recently. HCM (2000) suffered from limitations in terms of capacity analysis methodology that it does not specifically address the following subjects (without

modifications by the analyst): 1. Ramp metering on entrance ramps forming part of the weaving segment, and 2. Weaving segments on collector-distributor roadways.

A typical separated connecting C-D roadway weaving section is illustrated in Figure 1.2. The 1965 HCM defined the separated connecting C-D roadway weaving section as follows: *“The dual-purpose weaving section attached to the compound weaving section can be arranged to separate weaving traffic from non-weaving traffic; this type is the separated weaving section, while the two flanking sections to be free of weaving and only carry the other flow. Such a separated weaving section, which is only provided on outer roadway, is characteristic of the introduced section of collector-distributor road along a freeway, thus removing weaving from the through roadway”*. The capacity analysis for the separated connecting C-D roadway weaving section was not mentioned in HCM 1985 and 2000. Recently in the United States, the collector-distributor (C-D) roadway definition and design criteria were included in “Design Manual M 22-01” for the improvement of traffic safety and efficiency on freeway system interchanges (Design Manual, 2005). Roadway improvements were proposed by the “I-405 Plan”, which included collector-distributor and auxiliary lanes (I-405 Plan, 2002). Rehabilitation in the freeway applied the C-D road to improve the traffic operations (Leisch, 1978).

Three weaving configurations, type-A, type-B and type-C, were defined based on

the lane change behaviors of weaving vehicles as described in the Highway Capacity Manual (HCM, 1965, 1985, 2000). Lertworawanich and Elefteriadou (2000, 2003) estimated the weaving area capacity for type-A and type-B based on the renewal processes of gap acceptance modeling and linear optimization.

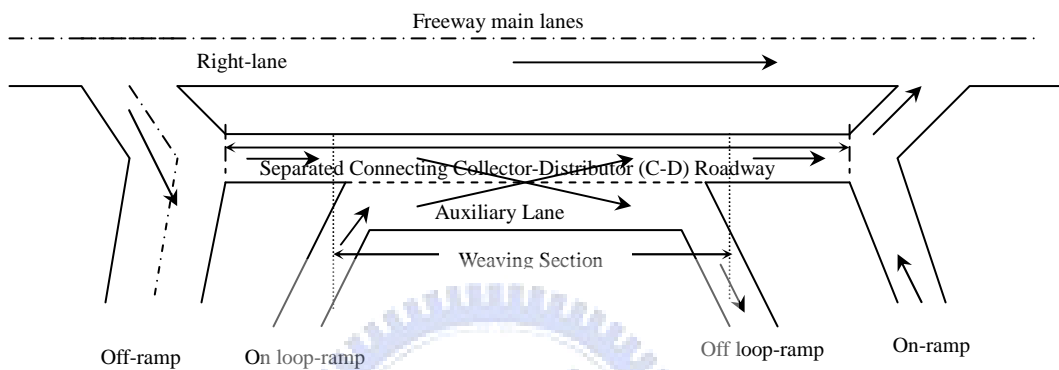


Figure 1.2 Typical separated connecting collector-distributor roadway

Operations of freeway weaving section are characterized by intense traffic flow characteristics and influenced by several geometric and traffic characteristics, such as number of lanes, length of weaving section, percentage of classification of vehicles on traffic stream, respectively. Weaving is the cause of disturbance in traffic stream, because weaving vehicles must take lane changes within the length of weaving section. Hence, weaving section must be turbulent, great time headway and low capacity, and as a result, come to be a bottleneck of freeway system (Fazio and Roupail, 1986).

Analysis on traffic operational characteristics of weaving sections is the key

process of weaving section treatment. These important characteristics should be basis of analytical methods of weaving sections. Three geometric variables influence weaving section operations, including weaving section configuration, length, and width (Pignataro, 1973; Traffic flow theory, 2000; and McGhee, 2001). This study focuses on investigating the traffic characteristic of freeway weaving traffic, which could impact upon the traffic operation on weaving section if not well-treated.

Freeway dynamic traffic control includes the freeway mainline dynamic traffic control and the entrance ramp/loop-ramp dynamic traffic controls; in previous study, these controls are the key process of freeway and weaving sections based on the static conditions of traffic flow. The results of these studies differentiated the analysis of the freeway from weaving sections dynamic conditions of traffic flow. In this study the focus is on the freeway and separated connecting C-D roadway local entrance ramp and loop-ramp metering control algorithm, applied the linear dynamic model with the dynamic flow control and/or density control of a section strictly below critical values (capacity or critical density) (Gazis, Hernauk and Weiss, 1962; Blumentritt *et al.*, 1981; Chang and Li, 2002; Zhang and Levinson, 2004). This study presents a ramp-metering control model capable of optimizing the C-D roadway and freeway mainline traffic by providing metering rates for the control sections, and its implementation would reduce the total queue lengths (Cho and Tsai, 2009).

1.2 Research Objectives

The ramp metering control by dynamic traffic flow conditions is an important research topic in freeway systems' traffic flow control development; a control modeling approach is presented in this study. Firstly, the review of related researches is presented. According to the review and real traffic situations, a dynamic traffic flow ramp metering control model is proposed. The main considerations of modeling are (Zhang and Levinson, 2004):

1. Use of upstream (rather than downstream).
2. Use of flow-based (rather than occupancy-based) set values and measurements.
3. Automatic, real-time adaptation of set values, so as to maximize the separated connecting C-D roadway and freeway downstream flow.
4. Efficient ramp-queue control to avoid interference with surface street traffic.
5. Multilane traffic and multiple types of driving behavior.

Owing to the considerations, a systematic model, which consists of a capacity estimation model and an optimal ramp metering control model, is proposed. After the systematic model is developed, a numerical analysis method is proposed. Numerical examples are also employed to explain the model. Consequently, the main research objectives of this dissertation are as follows.

1. Analyzing the driver's characteristics of traffic operation at weaving section on

separated connecting C-D roadway.

2. Modeling maximum capacity estimation on the separated connecting C-D roadway of freeway weaving sections.
3. Developing a modeling procedure of dynamic traffic ramp metering control research.
4. Presenting a numerical analysis method for the optimal ramp metering control model.
5. Discussing and developing the relationship of maximum capacity and optimal ramp metering control.

1.3 Research Approach

The study is consisted of the following basic steps: literature review, study site selection and field data collection and analysis, fundamental relationships analysis of traffic flow characteristics on the weaving section, capacity analyzed on the weaving section, and on optimal ramp metering control for congested weaving section. The basic research activities in each of these steps are briefly reviewed in the following.

In respect of driving behavior analysis and dynamic optimal C-D roadway capacity estimation and optimal ramp metering control, since spatial and characteristics are concerned. This study tries to apply the macroscopic systems of statistic analysis methods to analyze the weaving section driver's behavior and traffic flow characteristics,

and developed the estimation model for estimating the maximum capacity and optimal ramp metering control at the separated connecting C-D roadway weaving sections on freeway systems. These are macroscopic and microscopic models. The equations of the macroscopic system are systematic conservation laws; i.e., conservation of vehicle numbers. Thus, the evolution of traffic flow, capacity and ramp metering control models are described by the system. A complete system includes state equations and dynamic traffic flow equations to make it self-consistent. The state equation is derived from the car-following theory, lane-changing theory and gap acceptance theory in this study (Lertworwanich and Elefteriadou, 2003).

In the case of stochastic dynamic traffic flow model, a simple model, which is based on the Lighthill-Witham-Richard (LWR) model, is presented first. The traditional statistics analysis methodology is used in this study, including the percentage and probabilities analysis for the traffic flow variables, the renewal processes to develop the maximum rear lag and lane change distributions and critical lag, to estimate the maximum capacity, and the use of the Poisson processes and maximum likelihood method to develop the lane-changing models. Last, this study applies the dynamic traffic flow theory to develop the optimal ramp/loop-ramp metering control models, and using the results of this model to compare the field data.

1.4 Overview of Thesis

This dissertation is organized as follows. First, the introduction chapter gives an overview of the motivation, research objectives and approach of this study. A literature review of related researches in the relevant areas is presented in the second chapter. The topics discussed in the chapter include (1) lane changing behavior analysis that includes microscopic models analysis and macroscopic models analysis, (2) the car following theory, (3) traffic flow models, (4) gap acceptance theory, (5) developed the estimation optimal weaving capacity model, and (6) developed the optimal ramp/loop-ramp metering control model, respectively.

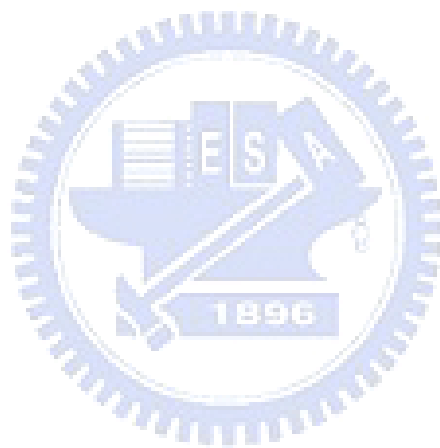
In the third chapter, the conformity of multi-dimensional macroscopic traffic flow variables and driver's behaviors on the separated connecting C-D roadway-weaving sections are analyzed and discussed. The driver's behavior analysis uses the traditional statistics methodologies to analyze. And the weaving traffic flow characteristics analysis uses regression methods to build up the prediction models, and describe the relationships of traffic flow rate, traffic speed and traffic density, respectively. This part is discussed in the fourth chapter. The fifth chapter analyzes relationships between the lane-changing behaviors and weaving traffic flow characteristics; it uses the Poisson processes and maximum likelihood method to develop the lane-changing models.

Application and analysis of the estimation maximum capacity model are discussed in the sixth chapter, using the renewal processes to develop the maximum rear lag and

lane change distributions and the critical lags, to estimate the lane capacity or roadway capacity. These results transform into the input data applied the next step to simulate or to calculate the optimal ramp metering control signal timing. In the seventh chapter, numerical analysis of lane or/and weaving section capacity and ramp metering control models by field traffic flow data.

The last chapter presents the conclusions and perspectives of this dissertation.





CHAPTER 2

Literature Review

Weaving traffic characteristics and optimal ramp metering control studies analyze including the lane changing, car-following, gap-acceptance, and relationships of the traffic speed, density, flow rate, and capacity, level of service (Zarean and Nemeth, 1988), and optimal ramp metering control models development etc. These studies began when the 1950 highway capacity manual (HCM) presented the first freeway weaving analysis method, which predicted the capacity and operating speeds of freeway weaving sections (Windover and May, 1994).

Macroscopic models, either static or dynamic, consider the average traffic stream characteristics (traffic flow, speed, and density), incorporate analytical procedures to evaluate existing conditions, and predict performance under different design and control scenarios (Skabardonis *et al.* 1989). These published materials of weaving traffic characteristics studies were classified into seven major categories that included methods for design/analysis of freeway weaving sections, simulation models, theoretical studies, data collection, related capacity analysis, non-freeway weaving sections, and California studies (Cassidy, Skabardonis and May 1989). These studies use the methodologies that included two theories. One is collecting field data of weaving flows on the weaving sections, applied statistic analysis (Roess, 1986) and to establish regression models

(Fredericksen and Ogden, 1994; and Cassidy, Skabardonis and May, 1989) to forecast the capacity and traffic flow on the weaving sections. Another discusses the level of service (LOS) and traffic safety on the weaving sections (Roess and Ulerio, 2001, Golob, Recker and Alvarez, 2003). These studies belong to the macroscopic model analysis that discussed and predicted the relationship of traffic speed, flow, density and safety. The optimal ramp metering control development has applied linear programming method to achieve the control optimum (Wattleworth and Berry, 1965), and evaluated with total travel time, that the minimization of total travel time equivalent to the minimization of total delay for a given travel demand pattern (Wu and Chang, 1999).

Microscopic traffic characteristics are described in terms of the car-following model (Wicks and Lieberman, 1977; Kachroo and Ozbay 1999; Helbing and Treiber, 2002). That simulation model consists of the INTRAS (Integrated Transportation Simulation) and TESS (Tongji Transportation nEtwork Simulation System) simulation model, it is a stochastic model, and simulates the movement of each individual vehicle on the freeway and surface street network, based on car-following, lane changing, and queue-discharge algorithms (Skabardonis, Cassidy, May and Cohen, 1989, Sun, Yang and Ma, 2005). The microscopic traffic flow simulator (MITSIM) laboratory was applied to evaluate the freeway traffic control (Ben-Akiva *et al.*, 2003). The evaluation study of two ramp metering algorithms, one is local control algorithm: ALINEA and the

other is area wide coordinated algorithm: FLOW. These algorithms uses microscopic simulation to evaluate systematically how the level of traffic demand, queue spillback handling policy and downstream bottleneck conditions affect the performance of the algorithms (Scariza, 2003; Chu and Yang, 2003; Papageorgiou *et al.*, 1998; Papageorgiou *et al.*, 1991; List *et al.* 2000; Hasan *et al.* 2002; Sun and Horowitz, 2006).

Weaving flow on freeway weaving sections have many different characteristics, including four main strategies with vehicle driving, that are dynamic traffic flow situation. These main strategies are car-following, gap-acceptance, lane-changing, merging and diverging control, etc. (Peter Hidas, 2005). The study of this dissertation focuses on analysis of the lane changing driver behaviors of weaving, and discusses the capacity on the weaving section and development the optimal ramp metering control model, respectively.

2.1 Lane Changing Behavior Analyses

The lane-changing driver behaviors on weaving section process is a very common phenomenon in traffic flows on the freeway and the separated connecting collector-distributor (C-D) roadway, and is the only operation a driver can exercise except changing speed. These aspects depend on the traffic situation and the rule of behavior (Gipps, 1986; Leutzbach, 1988). It is one of the must critical aspects in driving behavior, and it has received relatively little attention, but the treatment of the lane

changing is not well, thus conflicts occurred (Chang and Kao, 1991). Therefore, several studies use the statistic analysis of the operational characteristics of the weaving sections, to understand the characteristics of traffic operation on weaving sections that include traffic speed, relationship between traffic speed and flow rate, lane distributed, and lane-changing etc. (Chen, Liu and Ren, 2000). Others are using the macroscopic lane-changing characteristic analysis to analyze the lane-changing behavior of driving vehicles on freeway weaving sections (Chang and Kao, 1991). The study uses the microscopic simulation model to develop a new lane change model to test the feasibility of the action depending on the prevailing traffic scenario (Chang, 1990; Ahmed *et al.*, 1996 ; and Peter Hidas, 2002). In this thesis we are focusing on analyzing the lane-changing driver behavior by using the macroscopic, microscopic and statistic methodology to review that behaviors have impacted the traffic operation on the weaving section on Taipei interchange of Taiwan National Freeway Systems.

The lane changing process is a very common phenomenon and maneuver in traffic flow on weaving sections of freeway, and is the only operation a driver can exercise except changing speed. It is one of the most critical aspects in driving behavior, but it has received relatively little attention, and the treatment of the lane changing is not well, thus conflicts occurred. Therefore, several previous studies had used the microscopic simulation models to analyze the interrelation of traffic flow characteristics and lane

changing behavior of driving vehicles on freeway weaving sections. The two most common conflicts of freeway and weaving sections are lane change (LC) and rear-end (RE) conflicts. The simulation model is to test in terms of their sensitivity to geometric and flow variables, and measures of effectiveness (MOEs) to evaluate the quality of service in freeway weaving sections, more effective than speeds (Fazio, Holden and Roupail, 1993; Fazio and Roupail, 1990). WEAVSIM (Weaving Simulation) simulation model investigated the effect of the difference in arrival speeds of the two merging traffic streams on the speed and delay in the weaving section. It allows the simulation of traffic flow through a three lane weaving section of any length and under varying traffic volume conditions. Speed and delay represent operational conditions (Zarean, 1987; Zarean and Nemeth, 1988; Hall and Lotspeich, 1996). NETSIM (Network simulation) simulation model this computer simulation model used field data to develop a relationship between number of lane changing and weaving volume. It was used to identify relationships between the average number of lane changes per hour on a weaving link and other variables, and also to predict various MOEs (e.g., speed and delay) under different conditions (Fitzpatrick and Nowlin, 1996). INTEGRATION simulation model has the ability to determine volumes in addition to speeds; it was used to select the design solution based on the ratio of demand volume to capacity and to evaluate the weaving section design. This model can also analyze the relationship of

flow and capacity, and predicted the flow and capacity vs. core length (Stewart, Baker and Aerde, 1996). Some studies use the lane-changing intensity (LCI) formulation of a weaving section on frontage road to measure effectiveness. That is a more direct measure of the turbulence experienced within a weaving section than speed. That formulation can express the number of lane changes per hour per mile per lane, as shown in the following equation:

$$LCI = \frac{\text{Number of lane changes per hour}}{\text{Number of lanes} \times \text{Length of weaving section}}$$

This equation was found sensitive to flow (Frederickson and Ogden, 1994). Apply the microscopic fixed-based driving simulator to determinate the safety margins of lane change maneuvers (Winsum, Waard and Brookhuis, 1999; Tan, Guldner, Chen, Patwardhan and Bougler, 2000; Jimenez, Mussi and Siegel, 2004).

2.2 Car-Following Theory

Car-following models are a form of stimulus-response equation (Gerlough and Huber 1975), where the response is the reaction of a driver to the motion of the vehicle immediately preceding him in the traffic stream. The response of successive drivers in the traffic stream is to accelerate or decelerate in proportion to the magnitude of the stimulus at time t and is to begin after a time lag T .

The basic equation of these models is of the form:

$$\text{Response}(t + T) = \text{Sensitivity} \times \text{Stimulus}(t)$$

Microscopic traffic characteristics are described in terms of the car-following models (Kachroo and Ozbay 1999). Reuschel (1950) and Pipes (1953) considered these models in the early 1950s, which represented the behavior of individual cars as they fought for survival and a place in a line of cars moving along a highway. Herman, Montroll, Potts and Rothery (1959) developed a series model of car following. Car following has been the subject of numerous modeling efforts in the past (Drew 1975; Gerlogh and Huber 1975; Papacostas 1987; Zarean 1987; Zarean and Nemeth 1988; Leutzbach 1988; May 1990; Daganzo 1997; Aycin and Benekohat 1998; Dijker, Bovy and Vermijs 1998; Kachroo and Ozbay 1999; Rakha and Crowther 2003; and Jimenez, Mussi, and Siegel 2004).

Every driver who finds himself in a single-lane traffic situation is assumed to react mainly to a stimulus from his immediate environment. The stimulus was assumed to be a function of the position of the car, the position of its neighbors, and the time-derivatives of these positions. It was conjectured, and verified experimentally, that the strongest stimulus was the relative speed of the car with respect to the car in front. If we further assume that the sensitivity λ is constant, we'll obtain the "linear car-following model":

$$\frac{d^2 x_{n_c}(t+T)}{dt^2} = \lambda \left[\frac{dx_{n_c-1}(t)}{dt} - \frac{dx_{n_c}(t)}{dt} \right] \quad (2.1)$$

$$\lambda = \frac{a(x'(t+T))_{n_c+1}^m}{[x_{n_c}(t) - x_{n_c+1}(t)]^l} \quad (2.2)$$

The model of Equation (2.1) was used to investigate the following equations:

1. Assuming a pair of cars, one following the other, under what circumstances would a maneuver causes a collision?
2. Assuming a long line of cars, under what circumstances does a perturbation by the car in front get simplified as it propagates down the line of cars?

The first question refers to the “local stability” and the second to the “asymptotic stability “of traffic.

Gazis, Herman and Rothery (1960) checked the linear car-following model and discussed nonlinear car-following models. In studies of nonlinear car-following model, the main difference between them was the formulation of sensitivity (Newell, 1961; Evans and Rothery, 1977; and Gipps, 1981). They combined Eqs (2.1) and (2.2) and obtained:

$$x''_{n_c+1}(t+T) = \frac{a(x'(t+T))_{n_c+1}^m [x'_{n_c}(t) - x'_{n_c+1}(t)]}{[x_{n_c}(t) - x_{n_c+1}(t)]^l} \quad (2.3)$$

Equation (2.3) is a generalized sensitivity function, which can be simplified to specific models proposed before. In the study, the model is also related to macroscopic model such as Greenshield’s model, Greenberg’s model and Edie’s model.

There are still different considerations of car-following modeling. Lee (1966)

considered that the reaction of a follower not only depended on the stimuli of his leader, but also his historical experience. Simonsson (1993) suggested a new car-following model. The hypothesis is that a driver reacts to the stimuli deceleration, which is proportional to two components: the difference between the preferred time gap and the actual time gap, and the difference between the preferred speed and the actual speed. The model estimated user impact and environmental effects with a knowledge-based expert system and showed the result with a numerical example. Todosiev *et al.* (1963) and Wiedemann (1974) introduced psycho-physiological considerations into the car-following models. Wiedemann considers so-called reaction thresholds to distinguish different regions of driver behavior and Fritsche (1994) extended the model further.

Mostly, applications of car-following theory are done by simulation such as signal control, effect of small cars (McClenahan and Simkowitz, 1969) and motorcycles and cars mixed traffic flow. Researchers of automatic highway system (AHS) have also based on car-following theory. The advantage of car-following theory is that it has behavioral meaning. On the other hand, it is also a disadvantage because vehicular behavior is hardly. Another disadvantage of car-following theory is rebuts of computing. This makes car following theory hardly provide real-time prediction, especially in large-scale traffic network (Zhang and Owen, 1998). In order to solve in the computer application, and then develop a succession of simulates the tool, processing with the

traffic stream of simulation situation the changing lanes and the car following behavior has been taken (Jimenez, Mussi and Siegel, 2004).

2.3 Traffic Flow Models

The local traffic dynamics of simple location considered herein (Figure 2.1) consists of three parts –freeway traffic dynamics, separated connecting C-D roadway, and queue dynamics on the ramps that link the separated connecting C-D roadway to the freeway. We assume that the traffic behavior of both freeway and separated connecting C-D roadway traffic follows the rules prescribed by the well-known Lighthill-Witham-Richard (LWR) theory (Lighthill and Witham, 1955; Richards, 1956); they were the first persons that presented macroscopic kinetic traffic model. They used the kinematical concepts to describe waves in traffic. The basic premises of their model are that traffic is conserved and that there exists a one-to-one relationship between velocity and density. The LWR model can be viewed as a good and basic approximation.

Mathematically, LWR model states that the traffic density k and flow rate q satisfy:

$$\frac{\partial q}{\partial x} + \frac{\partial k}{\partial t} = 0$$

$$q = k * u(k) = f(k) \tag{2.4}$$

Equation (2.4) expresses the conservation of vehicles. In addition, q, k and velocity u is assumed to satisfy (2.4). We assume in this exposition the triangular flow-density relationship proposed by Newell (1993):

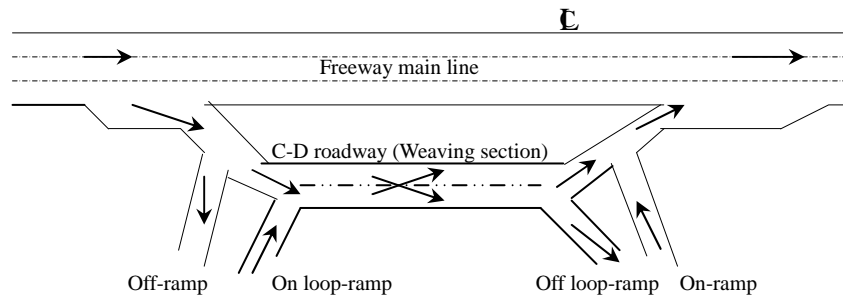


Figure 2.1 Samples coding for the separated connecting C-D roadway

$$q = v_f(k), \text{ if } k \leq k_c$$

$$q = c_w(k_{jam} - k), \text{ if } k \geq k_c \quad (2.5)$$

We further assume that the freeway and separated connecting C-D roadway have the same critical and jam densities, but different capacities (Zhang and Recker, 1999). Equation (2.4) implies that in homogenates, such as, changes in density of cars; propagate along a stream of cars with constant wave speed c with respect to a stationary observer. The effect of this simplifying assumption is to ignore the details of traffic control (e.g. signal control on the arterial streets) and, rather, assume that the traffic flow conditions on the freeway and separated connecting C-D roadway can be represented adequately by uninterrupted flow with these equivalent properties.

The control of the traffic input by platoon the entering vehicles are shown to eliminate the shock waves and allow higher flows (Cho and Lo, 2002). Michalopoulos and Pisharody (1980) and Michalopoulos *et al.* (1981) and Arnott *et al.* (1993) and Liu *et al.* (1996), also employed the LWR model and shock wave analysis to derive isolated real time signal control.

2.4 Gap Acceptance Theory

Gap acceptance is one of the most important components in microscopic traffic characteristics (Hwang and Chang, 2005). Tsongos and Wiemer (1969), in their investigation of gap acceptance, have found different distribution of acceptance and rejection for night than for day.

Worrall, Bullen and Gur (1970) conducted a statistical analysis of the data collected at thirty freeway locations in Chicago area. The study results indicate that there is a tendency for the intensity of lane changing to increase in the vicinity of ramp area. Furthermore, gap acceptance behavior displays only a very weak, and as yet largely undefined, relationship to traffic flow. As a result of this study, a gap acceptance function based on lane density was formulated for lane changing vehicles.

In another study of gap acceptance, Moshe (1970) and Tian *et al.* (1999, 2000) applied the critical gap concept by using a distribution of angular velocities at merges. The drivers' critical gaps were then evaluated based on the 50th percentile value of the angular velocity.

Drew (1971), in a study of gap acceptance, evaluated the effect of relative speed on the critical gap for merging ramp vehicles at the Dumble entrance ramp of the Gulf freeway. Drew, as a result of his study, suggests that the critical gap be directly related to the speed. The critical gap for moving vehicles (based on the first gap evaluated) is

about 20% less than that of stopped vehicles. Drew developed a set of gap acceptance functions, and a merging delay model.

In an analysis of aerial data, Paul (1970) investigated gap acceptance characteristics for exiting vehicles close to their intended off ramps and for through vehicles as a function of distance from the off ramp. In this study Paul has shown that the mean and median gap that was accepted decreases as drivers move closer to the exit ramp. Paul supported these findings by referring to the fact that as the driver felt more psychological pressure to move toward the exit lane, he was willing to change the threshold requirements of the gap size that was needed. This result, which tends to agree with Worrall and Bullen (1969) findings, has some implication in development of the lane-changing algorithm of WEAVSIM. In fact, development of the lane-changing factor is based on the fact that as drivers move closer to the gore area of diverge, they become willing to accept greater risk and use more severe deceleration. Paul also shows that the size of the accepted gap decreases as flow levels increase.

Miller (1972), in his study of driver's gap estimation capabilities, explored the threshold gap acceptance functions under both static and dynamic conditions. One major finding of this study was that when gaps of equal length were compared, the lead gaps were shorter than the lag gaps.

2.5 Capacity Estimation Models

There is no method available for estimating freeway weaving capacity (Lertworawanich and Elefteriadou, 2003) and analyzing to establish the models of capacity on weaving sections, more than all the analysis of capacity of the separated connecting C-D roadway weaving sections are very deficient recently.

The study of estimating roadway traffic capacity have Hyde and Wright (1986) and Chang and Kim (2000) using two Statistical theories, to estimate the roadway traffic capacity, one is straight-forward probability theory to predict the largest flows likely to be observed during a given period, assuming an idealized traffic stream with a known flow counting distribution, and the other is using asymptotic methods of the kind which are frequently used in connection with meteorological and flood defense problems. Minderhoud, Hein and Bavy (1998) used headway distribution approaches and the bimodal distribution method that based on traffic volume counts to estimate the roadway capacity. Michael, Godbole, Lygeros and Sengupta (1998) calculated bounds on per-lane Automated Highway System (AHS) capacity as a function of vehicle capabilities and control system information structure (Carbaugh *et al.*, 1998). Capacity is constrained by the minimum inter-vehicle separation necessary for safety operation. Hall and Chen (1999), develop methods to calculate an upper limit on per-lane throughput for an automated highway system with mixed vehicle classes and platoon operational. The models are analytical and based on independent arrivals among the

classes. The results indicate that a mixture of vehicle classes or categories can have a significant detrimental effect on vehicle capacity. Zhang (2001) noted that on a homogeneous, uninterrupted flow highway there could be three kinds of capacity for any location, one for acceleration flow, the other of deceleration flow and still another for stationary flow. It is the stationary (equilibrium) flow capacity that one should adopt as the ideal capacity for a roadway, as defined by the highway capacity manual (Kittelson, 2000).

Development of weaving area analysis procedures and evaluation weaving capacity have Roess (1986), Fazio and Roupail (1990), Cassidy and May (1991), Wang, Cassidy, Chan and May (1993), Vermijs (1998), Velan and Aerde (1998), Aerde (1999), Kwon, Lau and Aswegan (2000, 2001), Roess and Ulerio (2001) and Cassidy and Rudjanakanoknad (2004) applying the merging of the weaving and non-weaving speed prediction algorithm developed by JHK and Associates, dependent on weaving area configuration and traffic flow conditions, that included the traffic speed and flow rate, or direct function of the interaction between the prevailing car-following and lane changing behaviors. It uses the regression method or other statistic analysis or INTRAS and INTEGRATION microscopic simulation models to estimate the weaving capacity to compare in year 1985 and 2000 Highway Capacity Manual (HCM).

The objective of developing a method for estimating weaving capacity of type A

and B weaving areas is based on gap acceptance and linear optimization by Lertworawanich and Elefteriadou (2003 and 2000), they applied the gap acceptance theory to compute the maximum lane-changes, and the critical lag is estimated by renewal processes. The results obtained by propositional method are compared to the HCM (2000) capacity values.

2.6 Modeling of Ramp Metering Control

Entrance ramp metering control is the most widely used form of freeway traffic control. It has been applied in numerous cities in the United States as well as in other countries. Its objective is the elimination, or at least the reduction, of the operational problems resulting from freeway congestion (Wilshire *et al.*, 1985 and Lumenbritt *et al.*, 1981). While performing a ramp-metering control study, linear programming method has been applied to achieve the control optimum (Ramp meter design guidelines, 1989, 1991; Wattleworth and Berry, 1965), and has been evaluated with total travel time, that the minimization of total travel time is equivalent to the minimization of total delay for a given travel demand pattern (Wu and Chang, 1999).

Existing ramp-metering research could be generally classified into two categories: i) fixed-time metering, and ii) traffic-responsive metering (Zhang and Ritchie, 1997). Fixed-time metering operates according to a predetermined metering rate; the traffic-response method adjusts the metering rate depending on current traffic

information. Existing fixed time metering methods can be referred to in Wattleworth and Berry (1965); Yuan and Kreer (1971); and May (1974). Several traffic-responsive ramp-metering methods have been developed over recent years, from trial-and-error approaches to systematic approaches: demand-capacity, occupancy control and ALINEA (Wilshire *et al.*, 1985; Papageorgiou *et al.*, 1991; Golstein and Kumar, 1982; Wei, 2001; Papageorgiou, 1984; Chien *et al.*, 1997; and Kachroo and Krishen, 2000). These ramp-metering algorithms, although traffic responsive, do not focus on separated connecting collector-distributor (C-D) roadways. This study tries to solve this problem by suggesting a new demand-capacity ramp-metering model that suits not only ordinary ramps but also separated connecting C-D roadways (Cho and Tsai, 2003, 2006).

The separated connecting C-D roadway aims to:

1. Reduce weaving and lane-changing on the main lanes of freeways,
2. Increase traffic safety on the main lanes of freeways, and
3. Enhance traffic speeds and traffic flows on the main lanes of freeways.

The definition of separated connecting C-D roadways can be found in the 1965 Highway Capacity Manual (HCM). Cho and Tsai discussed the capacity, which is critical in ramp metering, of separated connecting C-D roadways, and found it much different from that of other types of C-D roadways (Cho and Tsai, 2005).

Microscopic traffic flow simulator (MITSIM) laboratory has been applied to the

evaluation of freeway traffic control (Ben-Akiva, Cunea, Hasan, Jha and Yang, 2003). A model is developed using SIMUL8 micro-simulation model to simulate two conditions for entry of vehicles into highway namely, with and without metering (Hooi *et al.*, 2005). Evaluation study of two ramp metering algorithms, one is local control algorithm: ALINEA and the other is area wide coordinated algorithm: FLOW. These algorithms are to use microscopic simulation to evaluate systematically how the level of traffic demand, queue spillback handling policy and downstream bottleneck conditions affect the performance of the algorithms. These algorithms are also comparative evaluation of several performance indices in the cases with and without ramp metering (Hasan, Jha, and Ben-Akiva, 2002; Habib and Papageorgiou, 1995; Papageorgiou, 2002; Smaragdis and Papageorgiou, 2003; Bellemans *et al.*, 2006; Kosnalopoulos, and Papageorgiou, web.; Kachroo and Ozbay, 2003). Other study of ramp metering control applied artificial neural network models, these models belong to mesoscopic model that is developed to simulate typical time series traffic data and then expanded to capture the inherent time-space interrelations (Wei, 2001), and applied a coordinated traffic responsive ramp control strategy based on feedback control and artificial neural networks. Traffic simulation show that the proposed nonlinear ramp control strategy compares the linear quadratic (LQ) control strategy in reducing total travel times, particularly at situations where drastic changes in traffic demand and road capacity

occur (Allen *et al.*, 1976; Barwell, 1983; Zhang *et al.* 1997 and 2001; Papageorgiou, 1984; Kachroo and Krishen, 2000; and Li, Shen and Dai, 2004).

Based on Payne's (1971) continuum traffic stream model that belongs to macroscopic simulation model, a linear dynamic model with a quadratic objective function constructed for integrated-responsive ramp-metering control. It is incorporating on-line origin-destination (O-D) estimation of coordinated interchanges into the proposed model increase efficiency of the control (Chang *et al.*, 1994; Chang and Li, 2002). Freeway traffic control by means of speed signaling and ramp metering has been addressed, by following a sub-optimal approach. Starting from the estimation of the traffic flow, standard parameterized closed-loop regulators for speed signaling and ramp metering has been tuned using an optimization procedure based on Powell's method. The regulators perform with an estimate of a macroscopic model based state vector given by an extended Kalman filter (Alessandri *et al.*, 1998; Jiang, 2003). Using the CORFLO and TRANSYT models, those models are macroscopic simulation models of traffic on signalized intersections and freeways (Yang and Yagar, 1995). That model calculates the performance of the signal timing plan, and a minimization procedure (Findler *et al.*, 1997). The purpose of study for freeway ramp metering control is beneficial to the system in terms of total timesaving, e.g., the ramp metering control applied dynamic optimal process to minimize the total spent in the system (Zhang,

Ritchie, and Recker, 1996, 1999).

Another study is using metering entry to highways to reduce commuting costs. Those commuting costs include the cost of free-flow travel time, right-of-way capital cost and maintenance cost, cost of time spent in queue, cost of schedule delay etc. The major cost of metering is providing a holding area for vehicles waiting to enter (O'Dea, 1999). The study develops AD-ALINEA strategy that is successfully tested in a stochastic macroscopic simulation environment under various scenarios (Smaragdis, and Papageorgiou, 2003).

2.7 Summaries and Discussions

Dynamic optimal ramp metering controls model and dynamic optimal C-D roadway weaving capacity estimation model are closely related in dynamic traffic management and control. Essentially, a freeway and C-D roadway capacity estimation model needs information of freeway and C-D roadway traffic flow variables, such as traffic flow rates, speeds, densities, lane changing rates, and weaving rates etc. (Leisch, 1974 and Lo and Tung, 2003). The relationship between dynamic traffic flow models and dynamic optimal ramp metering control models are illustrated as Figure 1.1. Dynamic traffic flow variables can be estimated directly by a flow/speed/density function (Drew, 1975; Yoo, 1987; May 1990; Cassidy, 1991; Smulders, 1996; Carvell *et al.*, 1997; HCM, 2000; Cho, 2002; and Wu, 2002) or by traffic flow models. Therefore,

traffic flow models are the basic part of traffic management and control applications.

The conclusions and comments of traffic flow and ramp metering control researches can be illustrated as follows.

1. As the LWR-like model is employed, one has to give externally the relation between speed and flow rate and density. This is unsatisfying in terms of development of a theory (Lighthill and Witham, 1955; Richard, 1956; Newell, 1993; Zhang and Recker, 1999; Cho and Lo, 2002).
2. Macroscopic simulation models may be selected for high-density, large-scale systems in which a study of the behavior of groups of units is sufficient. It provides continuous properties for analyses. However, lack of behavioral discussion has been the drawback of macroscopic models. (Skabardonis, Cassidy, May, Cohen, 1989; Chronopoulos and Johnston, 1998 ; Cho and Lo, 2002, and Wu, McDonald and Chatterjee, 2007).
3. Microscopic simulation models have usually been adopted for the simulation of relatively small or simple system and macroscopic simulation can be used to simulate the large data or complex system. Numerous runs of microscopic simulations may be necessary to achieve such a long time. This limitation makes applications of these simulation models problematic for the provision of real-time traffic information (Zhang and Ritchie, 1997; Ran, Leight, and

Chang, 1999; Hasan *et al.*, 2002; Czogalla, Hoyer, and Jumar, 2002; and Chu *et al.*, 2002).

4. Using the development simulation equation models of local section on the freeway and separated connecting C-D roadway to estimate the capacity and to get to the optimal ramp metering control signal timing is highly beneficial. The dynamic identification and prediction of the network demand pattern should be incorporated into the future coordinated algorithms. The integrated control that incorporates merging and diversion, signal and ramp control should be modeled to study the impact of the route merging and diversion on the performance of ramp metering (Webster and Cobbe, 1966; Zhang and Ritchie, 1997; Zhang and Recker, 1999; Chien, 2001; Zhang, Ritchie and Jayakrishnan, 2001; Lei and Steward, 2002; Chu *et al.*, 2002; Papageorgiou *et al.*, 2003; Smaragdis *et al.*, 2004; Zhang and Levinson, 2004; Zhang and Levinson, 2004).

CHAPTER 3

Data Collection and Analysis

Data for this study were collected to formulate the regression models and applied to estimate the capacity and the ramp metering control models.

3.1 Data Requirements

Data collection activities for this research included traffic flow rate, vehicle classification, lane changing activity, traffic speed, density, and weaving section geometry. Personnel from the Taiwan National Freeway Bureau using video-recording equipment of ATMS collected all operational data. The weaving section geometry was obtained from freeway plans and field measurements.

3.2 Study Site Selection

The field data of operation in the separated connecting collector-distributor (C-D) roadway weaving sections were collected on May 20, and November 22, 2002, at one site. These data were collected through surveillance video cameras during the morning rush period (07:00 to 09:00). During data collection, the ramp metering controls were disabled to make sure the traffic conditions represented real demand.

The study site belongs to a cloverleaf type interchange with separated connecting Collector-Distributor (C-D) roadway (HCM, 1965). It is level grade and straight segment. The configuration of this site includes two loop-ramps (one on loop-ramp and

one off loop-ramp), two ramps (one on-ramp and one off-ramp), one separated connecting collector-distributor roadway (weaving section) and one mainline of freeway, respectively. The configuration of this site as indicated in Figure 3.1 and Table 3.1 shows the general conditions of this site.

These video cameras are used in the ATMS system component to do data collection. These cameras should be at a high enough position to record the whole situation and maneuver of the weaving section.

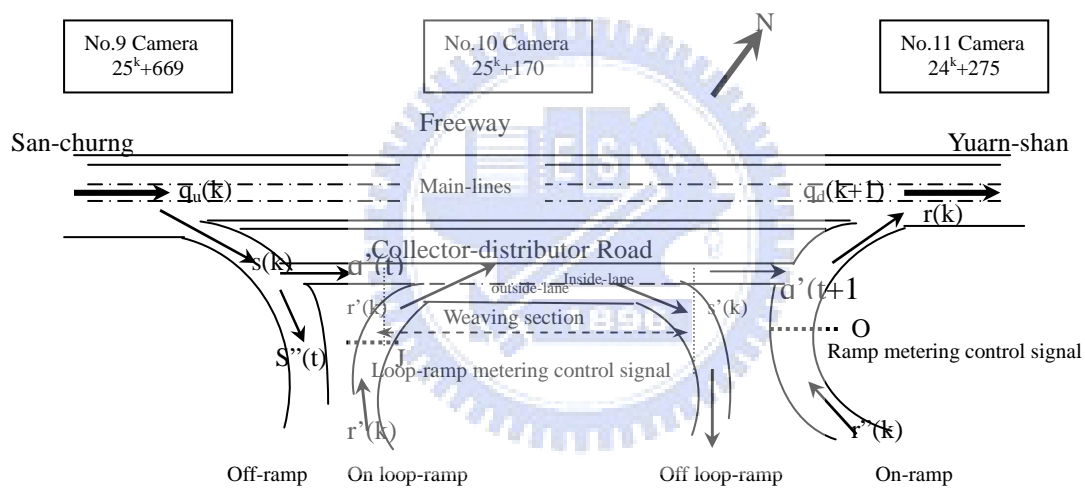


Figure 3.1 The configuration of C-D roadway weaving section on the Taipei interchange.

Table 3.1 General conditions of the study weaving section

Study section	Length (m)	Number and width (m) of Lanes				Curve (m)	Super-elevation (%)	Grade (%)	Affected by Signal	
		Main-Stream	Entry/ Exit	Entry	Exit					
Main-line	≈1015	3	3.75	-	-	R=0	2	0	No	No
C-D Roadway	≈806	1	3.75	1	3.75	R=0	2	0	No	No
Weaving Section	≈72	2	3.75	-	-	R=0	2	0	No	No
On-Ramp	≈402	-	-	1	4.5	R=300	10	+6.0	Yes	-
Off-Ramp	≈534	-	-	2	3.75	R=500	10	-1.32	-	No
On Loop-ramp	≈259	-	-	1	4.5	R=35	10	+5.52	Yes	-
Off Loop-ramp	≈270	-	-	1	4.5	R=45	10	-5.96	-	No

3.3 Analysis Results of Data Collection

Once the required traffic data were collected, the appropriate operational data were extracted directly from the videotape documentary. These data were summarized in 1-minute intervals. This time interval was used to increase the sample size.

Flows of traffic entering the weaving section and flows of weaving vehicles were measured from the videotaped data. Densities were also obtained directly from the videotapes by counting the number of vehicles in a weaving section at a given time. Pausing the videotape every 5-second, recording the densities for each lane, and averaging the reading to obtain a density value for each 1-minute periods did this. Television timer was used to determine average speeds and the time it takes for a vehicle to travel a known distance. All lane changes within the entire weaving section were counted and summed for each 1-minute period; these values were then converted to lane changes per hour per kilometer per lane. Weaving section length was measured between the painted gore points showed as Figure 3.2.

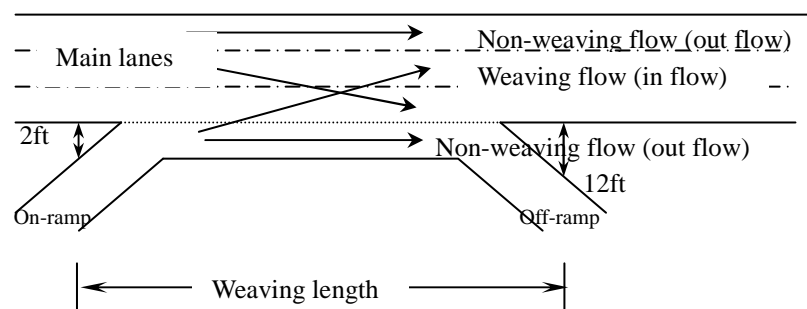


Figure 3.2 Weaving section lengths was measured between the painted gore points

3.3.1 Traffic operational characteristics

Based on the field collecting data, this paper presents the driver behavior of traffic operation with the lane-changing rate and numbers on the weaving section, that driver behavior of lane changing is dependent on traffic density, traffic speed and traffic flow rate etc.

3.3.1.1 Traffic flow rate

By analyzing of the field data from 120 minutes of camera videotape, we can conclude the composition of traffic flow rate on the weaving section as follows and as in table 3.2.

1. The total flow is 4988 vehicles from 07.01.20AM to 09.01.19AM on the weaving section. Average flow rate is 42 vehicles per minute (2520 vehicles per hour).
2. The lane changing rates are 100% of those vehicles to made the lane changing driving. The lane-changing rate that vehicles from the inside-lane change to the outside-lane is 23.54% and the lane-changing rate that vehicles from the outside-lane change to the inside-lane is 76.46%. The number of weaving vehicle is 2459 vehicles and the weaving rate is 49.29%, e.g., the remainder vehicle of 2529 vehicles that vehicles execute lane changing only, and the non-weaving rate of 50.71%.

Table 3.2 Traffic flow rate on the weaving section of Taipei interchange

Class of vehicles	Traffic flows (Vehicles)		
	Inside-lane	Outside-lane	Subtotal
Passenger car	894	3242	4136
Light Truck	109	226	335
Bus	51	157	208
Heavy Truck	120	189	309
Total	1174	3814	4988

3.3.1.2 Traffic speed and acceleration and deceleration

3.3.1.2.1 Traffic speed

Figure 3.3 and 3.4 illustrate the traffic speed distribution on the inside-lane and the outside-lane in the weaving section on Taipei Interchange of Taiwan National Freeway Systems. From these figures that are to divide into two parts for discussion, it may be concluded that:

1. The average speed on the weaving section is 24.47 kilometers per hour.
2. On the inside-lane that the average speed being 23.40 kilometers per hour, the range of speed is on 5.95 ~ 48.62 kilometers per hour. The range of speed concentrated on 8.81~13.76 kilometers per hour, that the rate is roughly 29.17%, and on 28.23~35.94 kilometers per hour, that the rate is roughly 30.00%. The total rate is 59.17%. The speed distributed on the inside-lane shows as Figure 3.3.

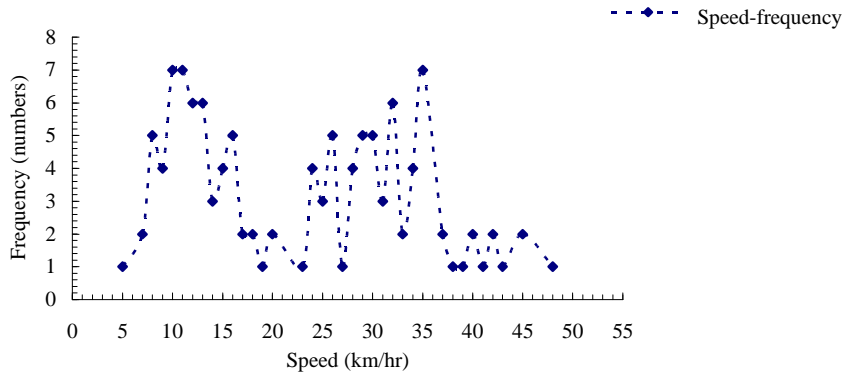


Figure 3.3 The diagram of traffic speed-frequency on the inside-lane

3. On the outside-lane that the average speed being 25.58 kilometers per hour, the range of speed is on 8.60 ~ 44.18 kilometers per hour. The range of speed concentrated on 12.07~17.87 kilometers per hour, that the rate is roughly 35.00%, on 23.17~29.11 kilometers per hour, that the rate is roughly 18.33%, and on 36.02~40.72 kilometers per hour, that the rate is roughly 15.83%. The total rate is 69.16%. The speed distributed on the inside-lane shows as Figure

3.4.

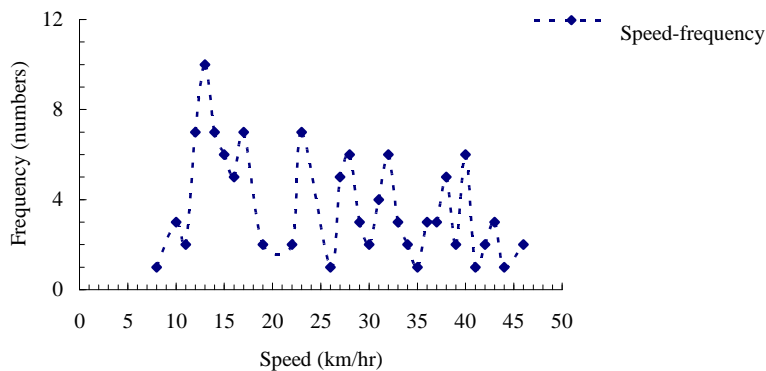


Figure 3.4 The diagram of traffic speed-frequency on the outside-lane

The traffic speeds of the vehicle classification are demonstrated as in Table 3.3.

Table 3.3 Traffic speeds on the weaving section of Taipei interchange

Class of vehicles	Average speed (km/hr)		
	Inside-lane	Outside-lane	Subtotal average
Passenger car	20.56	23.45	22.01
Light Truck	23.79	24.79	24.29
Bus	26.50	31.37	28.93
Heavy Truck	22.76	22.75	22.65
Total average	23.40	25.58	24.49

Comparison of the average speed differentials of correlation with each vehicle traveling in the weaving section indicated that this differential are not significant, that range concentrated on 20.56~31.37 kilometers per hour. The average speed of buses is faster than other vehicles on the weaving section.

3.3.1.2.2 Acceleration and deceleration

A following vehicle is able to avoid collision even when a lead vehicle undergoes the most extreme deceleration. This constraint is, however, relaxed during lane-changing maneuvers. But vehicles driven on the weaving section to make lane-changing activities may accept potentially unsafe positions for a short period of time when engaged in the weaving maneuvers that their speed accelerates or decelerates significantly. The vehicle average acceleration on the inside-lane is about 0.53 km per hr² and the average deceleration is about 0.57 kilometers per hr². The differential acceleration or deceleration of each classification of vehicle shows as passenger cars accelerate 0.03 kilometers per hr², light trucks accelerate 0.01 kilometers per hr², buses accelerate 0.15 kilometers per hr², and heavy trucks decelerate 0.21 kilometers per hr²,

respectively. This is because the heavy trucks decelerate apparently at high density on the weaving section. On the outside-lane vehicle accelerates about 0.86 kilometers per hr^2 , and decelerates about 0.63 kilometers per hr^2 . The differential acceleration or deceleration of each classification of vehicles shows that passenger cars accelerate 0.12 kilometers per hr^2 , light trucks accelerate 0.24 kilometers per hr^2 , buses accelerate 0.15 kilometers per hr^2 , and heavy trucks accelerate 0.15 kilometers per hr^2 , respectively.

3.3.1.3 Traffic density

Density is defined as the number of vehicles occupying a length of roadway usually in a single lane over a length of 1 mile (or 1 kilometer). Another method of computing the density uses the distance headway and the distance gap, but direct measurement of those distances in the field is difficult (May, 1990, and HCM, 2000). Therefore, this study is using the number of vehicles occupying a given length of 72 meters on the weaving section. And the weaving vehicles driving in each lane on weaving section under the situation including the density and lane-changing behaviors, etc. are counted and analyzed. When the vehicle driven begins to execute the weaving activities on the inside-lane change to the outside-lane, the density on the outside-lane is one of main factor to impact the weaving activities. The density on the inside-lane is one of main factor to impact vehicle on the outside-lane change to the inside-lane weaving activities. The results of analysis are as follow.

1. Vehicles on the inside-lane take weaving activities to the outside-lane, and the average density on the outside-lane is 75.51 vehicles per kilometer.
2. Vehicles on the outside-lane take weaving activities to the inside-lane, and the average density on the inside-lane is 59.13 vehicles per kilometer.
3. The density of the beginning point executed by the weaving activities of each vehicle on the inside-lane concentrates on 36 vehicles per kilometer that the rate is 1.61%. On 43 and 58 vehicles per kilometer the rate is 1.57%. On 54 vehicles per kilometer the rate is 1.53%. On 33 vehicles per kilometer the rate is 1.41%. On 41 vehicles per kilometer the rate is 1.34%, and on 47 vehicles per kilometer the rate is 1.30%etc. The total rate is 10.33%. The density distributed on the inside-lane shows as Figure 3.5.
4. The density of the beginning point executed by the weaving activities of each vehicle on the outside-lane concentrates on 50 vehicles per kilometer that the rate is 1.41%. On 53 and 45 vehicles per kilometer the rate is 1.39%. On 43 and 60 vehicles per kilometer the rate is 1.36%. On 47 vehicles per kilometer the rate is 1.34%. On 55 vehicles per kilometer the rate is 1.31%. On 48 vehicles per kilometer the rate is 1.27%, and on 62 vehicles per kilometer the rate is 1.24% etc. The total rate is 12.07%. The density distributed on the outside-lane shows as Figure 3.6.

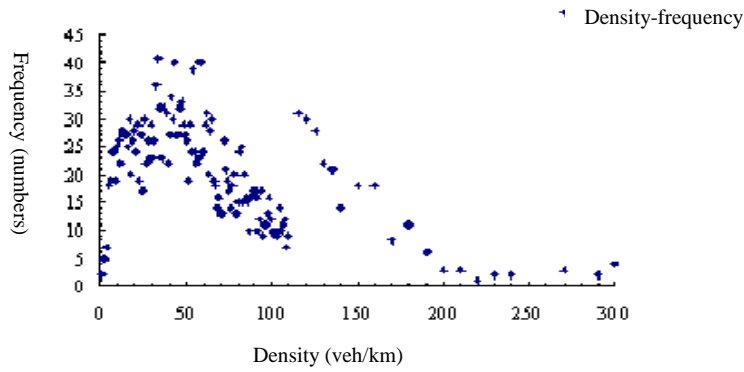


Figure 3.5 The diagram of traffic density-frequency on the inside-lane

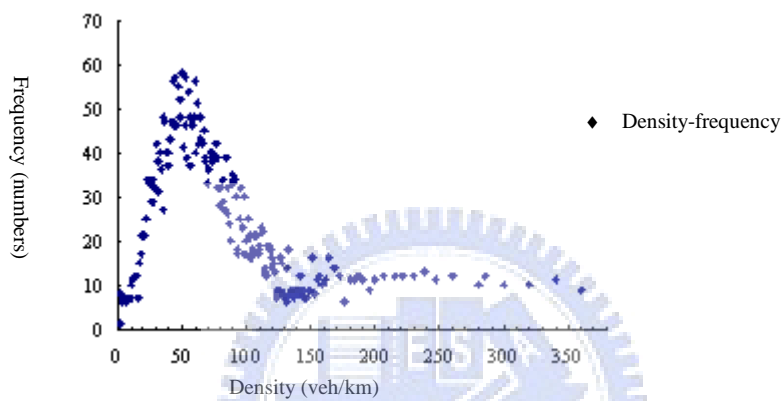


Figure 3.6 The diagram of traffic density-frequency on the outside-lane

Results of the previous analysis indicates when the vehicles take the weaving activities on the weaving section in the morning on Taipei interchange while the traffic operational situation are in the higher density, there would be about 67.32 vehicles per kilometer, belonging to level of service E (HCM. 1985).

3.3.2 Driver's behaviors

The data analyzed of the driver's behaviors in this study focuses on the lane changing behaviors.

3.3.2.1 Lane changing behaviors

Lane changing is the radical factor in weaving sections and affects the operational

characteristics essentially (Chen, Liu, and Ren, 2000). When a vehicle travels down a particular link, it may make discretionary lane changes, mandatory lane changes, or both. Discretionary lane changes are a function of the prevailing traffic conditions, while mandatory lane changes are usually a function of the prevailing network geometry (Aerde, 1999). Analysis of driver behaviors of lane-changing on the weaving section requires that each weaving vehicle makes lane change to execute the weaving movement, including the lane-changing numbers and rates, and lane-changing location and timing distribution etc.

3.3.2.1.1 Location of lane-changing distribution

The location of the lane-changing distribution analysis with the field data is as follows.

1. Beginning the location of lane changes on the inside-lane that the minimal distance is -13 meters, and the maximal distance is 75 meters. A lot of vehicles making the lane-changing distance concentrated on 0~5 meters that the rate is 25.35%. On 10~15 meters the rate is 18.52%, and on 20~25 meters the rate is 9.26% etc. The total rate is 53.13%. 608 vehicles (the rate is 69.64%) will be making the lane-changing distance on the weaving section length (72 meters), but some vehicles (265 vehicles, about 30.36%) are beginning the lane-changing at location before the starting point.

2. Finishing the location of lane changes on the inside-lane that the minimal distance is -8 meters, and the maximal distance is 80 meters. A lot of vehicles making the lane-changing distance concentrated on 0~5 meters that the rate is 10.31%. On 10~15 meters the rate is 15.64%. On 20~25 meter the rate is 18.08%, and on 30~35 meters the rate is 16.92% etc. The total rate is 60.95%. All vehicles will be making the lane-changing distance on the weaving section length (72-meters), but 33 vehicles (about 3.82%) finish the lane changing at location before than the original point, and 7 vehicles (about 0.81%) finish the lane changing at location behind the closing point.

3. Beginning the location of lane changes on the outside-lane that the minimal distance is -15 meters, and the maximal distance is 110 meters. A lot of vehicles making the lane-changing distance concentrated on 0~5 meters that the rate is 11.54%. On 20~25 meters the rate is 12.35%. On 30~35 meters the rate is 11.91%, and on 75~80 meters the rate is 23.95% etc. The total rate is 59.75%. 663 vehicles (the rate is 23.04%) begin the lane-changing at location before the starting point, and about 715 vehicles make the lane-changing distance is longer than the weaving section length (the rate is 24.82%).

4. Finishing the location of lane changes on the outside-lane that the minimal distance is -10 meters, and the maximal distance is 110 meters. A lot of

vehicles making the lane-changing distance concentrated on 0~5 meters that the rate is 11.57%. On 10~15 meters the rate is 11.52%. On 20~25 meters the rate is 13.53%. On 30~35 meters the rate is 11.67%, and on 75~80 meters the rate is 11.32% etc. The total rate is 59.61%. About 97 vehicles (the rate is 4.75%) begin the lane changing before the starting point, and about 249 vehicles (the rate is 12.21%) is extra the weaving section length.

3.3.2.1.2. Timing of lane changing distribution

The timing of the lane-changing distribution analysis with the field data shows as follows.

1. Beginning the time of lane changes on the inside-lane that the minimal timing is 0.10 seconds, and the maximal timing is 24.12 seconds. A lot of vehicles taking the lane-changing time concentrated on 0.10~1.09 seconds that the rate is 25.29%. On 1.20~2.19 second the rate is 28.62%, and on 2.47~3.46 seconds the rate is 16.90% etc. The total rate is 70.81%, and 7 vehicles (about 0.80%) begin the lane-changing before than origin time point.
2. Finishing the time of lane changes on the inside-lane that the minimal timing is 0.19 seconds, and the maximal timing is 24.12 seconds. A lot of vehicles taking the lane-changing time concentrated on 0.80~1.79 seconds that the rate is 36.81%. On 1.80~2.79 seconds the rate is 25.23%, and on 3.00~3.99 second

the rate is 11.00% etc. The total rate is 73.04%.

3. Beginning the time of lane changes on the outside-lane that the minimal timing is 0.73 seconds, and the maximal timing is 52.95 seconds. 904 vehicles taking the lane-changing time concentrated on 1.00~1.99 seconds that the rate of 31.46%. On 2.10~3.09 seconds the rate is 15.40%, and on 3.40~4.39 seconds the rate is 6.36%, etc. The total rate is 53.22%, and 17 vehicles (about 0.59%) begin the lane-changing before the origin time point.

4. Finishing the time of lane changes on the outside-lane that the minimal timing is 0.24 seconds, and the maximal timing is 51.95 seconds. A lot of vehicles taking the lane-changing timing concentrated on 0.40~0.90 seconds that the rate is 12.65%. On 1.00~1.99 seconds the rate is 36.23%. On 2.10~3.09 seconds the rate is 16.91%, and on 4.00~4.99 seconds the rate is 9.17%, etc. The total rate is 74.96%. Those finished time of lane changes on the outside-lane that will be uniform to 0.40~3.09 seconds time interval that the rate is 74.46%.

3.3.2.1.3 Length of lane changing distribution

The length of the lane-changing distribution analysis with the field data shows as follows.

1. The length of lane changing concentrated on this weaving section at 10 meters

that the rate is 10.81%. At 12 meters the rate is 6.38%. At 15 meters the rate is 6.74%. At 20 meters the rate is 17.09%, and at 30 meters the rate is 6.01%. The total rate is 47.03%. The length of lane changing distributed on the weaving section shows as Fig. 3.7.

2. The number of vehicles changing lanes over than the weaving length on this weaving section is 940 vehicles (the rate is 23.69%). The length of lane changing concentrated on 80 meters that rate is 14.36%, and on 75 meters the rate is 71.60%, that the total rate is 85.96%.

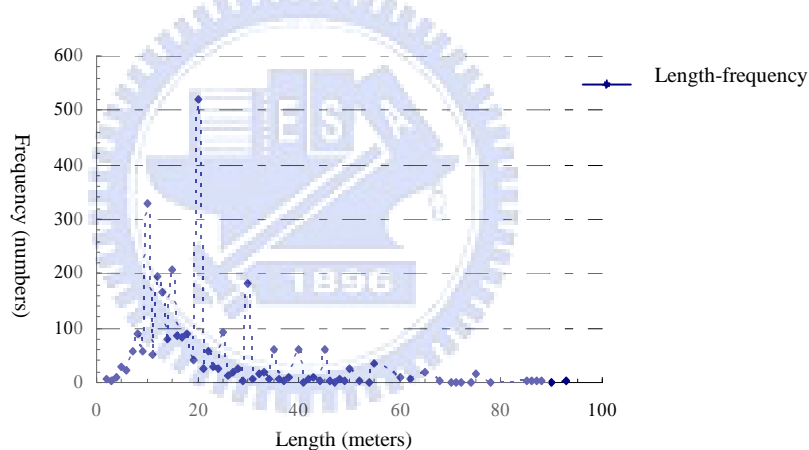


Figure 3.7 The diagram of change lane length –frequency on the weaving section

3.4 Summaries and Discussions

This section has presented a procedure for estimation of weaving speed and density over time for a separated connecting C-D roadway weaving section, which is the most common type of weaving area in the Taiwan National Freeway network. The field observations and the analysis of traffic density and traffic speed, and correlation of lane changing that can be summarize the lane changing behaviors on the weaving

section shows as in Table 3.4 and Table 3.5. This study presents some conclusions as follows:

1. The main purpose of the traffic operational on separated connecting C-D roadway is taking the traffic safety due to the vehicle making the lane changing and weaving activities on weaving sections. Vehicles driven on the weaving section of C-D roadway on Taipei interchange that the lane changing is 100%. The weaving activity is dependent on the distribution with the vehicle driven on each lane of weaving section, so that the weaving rate of 49.29%.
2. The lane-changing behavior may cause turbulence in weaving sections. To avoid the effect of weaving and to avoid the forced weaving, some vehicles begin the lane changing before the origin point on the weaving section, the changing rate about 30.36% on the inside-lane and 23.04% on the outside-lane. A lot of vehicles make the lane changing behind the closing point that rates about 24.82% on the outside-lane. A lot of vehicles finish the lane changing behind the closing point of weaving section, that rate about 0.81% on the inside-lane, and 12.21% on the outside-lane.
3. According to the relationship of LWR (May, 1990 and Wilhelm, 1988), density is a main factor to impact the traffic flow rate and traffic speed. Therefore, analysis of the density for weaving sections is very important to this study in

this paper. Results can be attained from the field data analysis, as the average density on the weaving section is 67.32 vehicles per kilometer per lane. The average density on the inside-lane is 59.13 vehicles per kilometer and on the outside-lane are 75.51 vehicles per kilometer, which concludes that the average density on the outside-lane is higher than the inside-lane. The average density in the morning on the C-D roadway weaving section of Taipei interchange is higher; it's 67.32 vehicles per kilometer per lane.

4. The major activity of vehicles driven on weaving sections is making the lane changing. Nevertheless, impact on the vehicle to make the lane-changing activity accepts the vehicle gaps and density. Furthermore, the vehicle speed and acceleration/deceleration are also impacted by the lane changing. Therefore, the vehicle speed and acceleration/deceleration are also analyzed in this study; the average vehicle speed on the weaving section is 24.47 kilometers per hour, slower than the HCM 1985. On the inside-lane the average vehicle speed is 23.40 kilometers per hour, on the outside-lane of 25.58 kilometers per hour, which indicated that the vehicle speed on the outside-lane is faster than the inside-lane. It is presented in this study that all vehicles making the lane-changing behavior on the weaving section make the acceleration activity from 0.01 to 0.24 kilometers per hour². But the heavy trucks driven on the

inside-lane only makes deceleration activity, moreover, the average deceleration attained to 0.21 kilometers per hr².

5. At very low density there is enough space in front and a typical vehicle very rarely feels the need for changing its lane and at very high densities most of the lane-changing attempts are unsuccessful. In this study the weaving section belongs to separate connecting C-D roadway that the vehicle must make the lane changing activities to the other lane. At the low density that vehicles may make the discretionary lane changes, their lane changing numbers and rates increase rapidly. But at the high density that vehicle makes the mandatory lane changes, their lane changing numbers and rates increase slowly.

Table 3.4 Driver behaviors with lane changing on the inside-lane of study site

Items Kinds of vehicle	Beginning location				Finishing location				Lane changing variables				Remark
	Distance	Time	Speed	Density	Distance	Time	Speed	Density	Length	Time	Speed	Acceleration Deceleration	
	m.	Sec.	Km/hr	Veh./km	m.	Sec.	Km/hr	Veh./km	m.	Sec.	Km/hr	Km/hr ²	
Passenger car	11.32	2.42	20.10	84.10	24.23	2.55	20.29	79.33	17.53	4.28	20.27	0.03	The all values in the table are average.
Light truck	12.43	2.04	23.80	72.42	24.18	2.21	23.78	71.40	16.92	3.36	23.15	0.01	
Bus	11.41	1.07	25.90	54.61	23.93	1.28	27.62	56.22	15.04	3.28	26.61	0.15	
Heavy truck	10.19	2.74	23.91	64.62	26.82	2.86	21.56	60.37	21.07	4.58	21.66	-0.21	
Average	11.34	2.07	23.43	68.94	24.79	2.23	23.31	66.83	17.64	3.88	22.92	-0.01	

Table 3.5 Driver behaviors with lane changing on the outside-lane of study site

Items Kinds of vehicle	Beginning location				Finishing location				Lane changing variables				Remark
	Distance	Time	Speed	Density	Distance	Time	Speed	Density	Length	Time	Speed	Acceleration Deceleration	
	m.	Sec.	Km/hr	Veh./km	m.	Sec.	Km/hr	Veh./km	m.	Sec.	Km/hr	Km/hr ²	
Passenger car	31.75	3.22	23.42	79.21	33.59	2.68	26.10	51.69	22.13	4.67	26.56	0.12	The all values in the table are average.
Light truck	36.46	4.10	22.36	83.80	34.38	3.06	27.10	74.11	20.13	3.97	28.53	0.24	
Bus	29.00	2.11	33.40	51.15	12.66	3.00	43.06	50.74	25.97	2.93	36.28	0.15	
Heavy truck	27.10	2.94	22.65	71.50	35.89	2.55	25.30	66.35	22.85	4.45	25.97	0.15	
Average	31.08	3.09	25.46	71.42	29.13	2.82	30.39	60.72	22.77	4.01	29.34	0.17	

CHAPTER 4

Fundamental Relationships of Traffic Flow Characteristics on the Weaving Sections

Before a regression model was built and developed for analyzing the separated connecting collector-distributor (C-D) roadway weaving section performance. The empirical relationships between traffic flow rate, traffic speed and traffic density were examined to have a better understanding of the operational characteristics of weaving sections (Cho and Tsai, 2005).

An important behavioral characteristic of traffic flow is the hypothesized relationship between the speed and density of traffic at equilibrium, i.e. the fundamental diagram (Sanwal *et al.*, 1996). The basic relationships among the three traffic measures are usually represented by typical curves plotted together (HCM, 1985). In order to ensure the suitability and the valid range of the data, the 1-minute interval flow rate, density, and speed for the weaving section of freeway are plotted together; that includes the flow rate versus speed, the flow rate versus density and density versus speed etc.

4.1 The Variables of Traffic Flow Characteristics

The variables of traffic flow characteristics include as traffic flow rate, traffic speed and traffic density, respectively.

Traffic flow rate is defined as “the equivalent hourly rate at which vehicles pass

over a given point or section of a lane or roadway during a given time interval less than one hour” (Yoo, 1987). A rate of flow is, therefore, computed by the following equation:

$$Q_{ik} = N_{ik} \left[\frac{3600}{i} \right] \quad (4.1)$$

The traffic speed used in this study is the average running speed, which can also be referred to as the space mean speed. The speed of each vehicle in the original data is a direct source for the calculation of the average running speed. Since the speed in the data is already in kilometers per hour, and average running for a given interval is obtained by computing the average of all speed observations in the given interval. The following equation is used to compute the average speed:

$$U_{ik} = \frac{\sum_{j=1}^i \sum_{m=1}^{n_j} U_{jm}}{\sum_{j=1}^i n_j} \quad (4.2)$$

Traffic density is defined as “the number of vehicles occupying a given length of a lane or roadway average over time, and in a single lane over a length of 1 mile (or 1 kilometer) is usually expressed as vehicles per mile (or kilometer)” (Yoo, 1987). Another method computed the density can use the distance headway and distance gap, but those distances direct measurement in the field is difficult (May, 1990, and HCM, 2000). Therefore, in this study is using the number of vehicles occupying a given length of 72 meters on the weaving section. And to count, to analysis the weaving vehicles driving in each lanes on the weaving section that situation included the density and lane-changing behaviors etc. Since the data contains the coordinate of each vehicle

within the study area, simply counting the number of vehicles within the weaving section and using the following equation can easily compute traffic density:

$$K_i = \sum_{k=1}^i N_{ik} \left(\frac{1000}{i \times L_w} \right) \quad (4.3)$$

4.2 Relationship between Speed and Flow Rate

Relationship between speed and flow rate was studied at the beginning. Average flow rate per lane was used to normalize the weaving section flows, and space-mean speed was obtained from the videotaped field data by calculating speeds from the travel space and travel time data (Smith, Hall and Montgomery, 1996). A scatter-plot of traffic speed versus traffic flow rate is illustrated in Figure 4.1. Aggregated 1-minute observation data from the study site was used to construct the scatter plot.

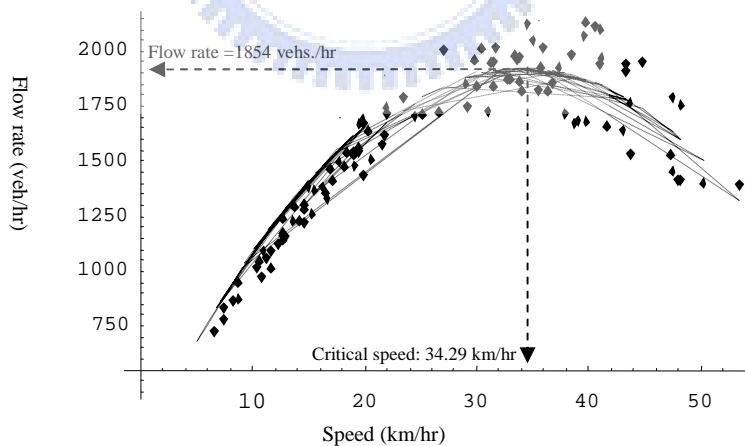


Figure 4.1 Regression results of traffic speed vs. traffic flow rate

Figure 4.1 illustrates a high degree of scatter among the data. Speed appears to be sensitive to the flow rates measured, e.g., fewer than 1854 vehicles per hour per lane.

More scatters were found in higher flow rate; indicating that speed is sensitive to flow as it approaches capacity. From the data collected, an obvious relationship between speed and flow rate was found. The critical speed is 34.29 kilometer per hour.

4.3 Relationship between Density and Flow Rate

Relationship between density and flow rate was also examined. Density was measured directly from the videotaped field data over the length of weaving section. Figure 4.2 illustrates the relationship between density and flow rate using average densities and average flow rate for 1-minute period. Flow appears to be sensitive to density, although the scatter increases at lower densities (e.g., fewer than 55.74 vehicles per kilometer), and the scatter decrease at higher densities (e.g., more than 55.74 vehicles per kilometer).

There is a conceptual flow in the relationship between density and traffic flow, however. For a given weaving section, the traffic speed is nearly constant until traffic flow approaches the capacity level. Fredericksen and Ogden (1994) and Benjamin (2003), found that traffic flow in the weaving section study did not approach capacity. This resulted in density values consisting of volumes divided by essentially constant speed. In this study, the plot of density versus flow rate is the same as speed versus flow rate, which would obviously be a strong linear relationship at lower traffic flow (e.g., lesser than 1854 vehicles per hour per lane). It was determined that a model for

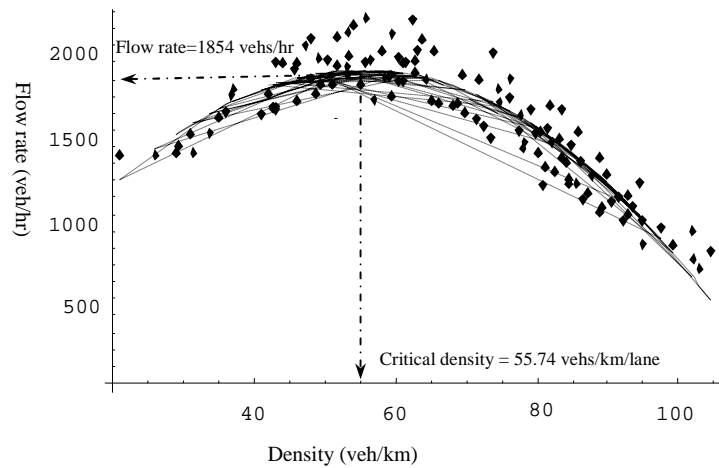


Figure 4.2 Regression results of traffic density vs. traffic flow rate

predicting densities on the basis of flow would be the most effective procedure for predicting traffic operations in the weaving sections on separated connecting C-D roadways.

4.4 Relationship between Speed and Density

Relationship between speed and density was also examined. Speed and density were measured directly from the videotaped field data over the length of the weaving section. Figure 4.3 illustrates the relationship between speed and density using average speed and average density for the 1-minute period. Density appears to be sensitive to speed, although the scatter decreases at higher speed.

Figure 4.3 presents the relationship between the traffic speed and traffic density over the observed period. This is significantly from most speed-density models in which the traffic speed generally decreases linearly or nonlinearly with the increase of density (Chang and Kao, 1991). However, in this case the traffic speed in these higher density

region or lowers density region appears to be a nonlinear relationship, which in median region would obviously be a strong linear relationship.

Those figures show reasonable agreements with the typical curves as far as the range of the data allows. The curves support that the overall level of service of the traffic from which this data was collected ranges from the level of service C to the critical point, i.e., the level of service E.

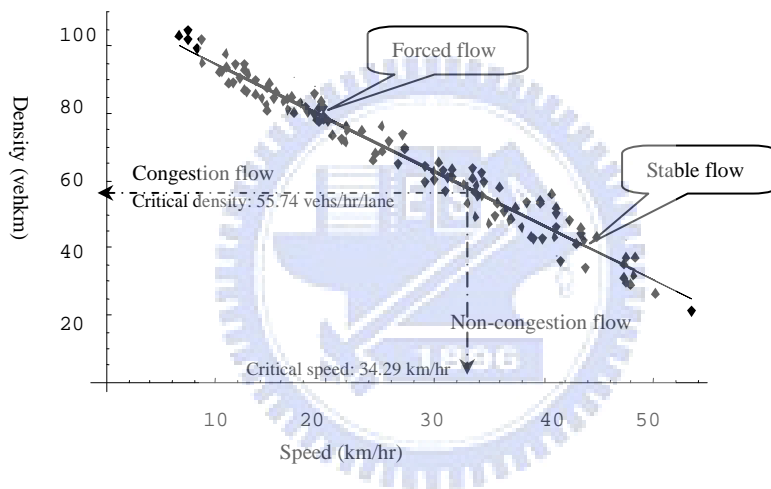


Figure 4.3 Regression results of traffic speed vs. traffic density

A detailed description of the data is presented and the time interval for the traffic conditions study is selected. In addition, procedures for computing traffic flow measures are explained. Finally, traffic conditions of the data and the relationships among the traffic condition parameters are studied.

Based on the 1-minute interval traffic conditions on the weaving section of the C-D roadway, the traffic flow rates ranged from 1357 to 4328 vehicles per hour ; the traffic densities ranged from 42 to 209 vehicles per kilometer; and the traffic speeds

ranged from 6.6 to 53.48 kilometers per hour. The flow rate and density increased with time during the study period while the speed decreased. The pair-wise relationship curves among the flow rate, density, and speed closely matches the basic relationship curves among these three parameters within the range of the data. The result supports the adequacy of the data. The level of service of the traffic during the hour ranges from C to E.

4.5 Analysis Results

The purpose of this study and analysis for weaving traffic characteristics is to build up the regression model of the correlation of traffic speed, flow rate, and density, and predict the trend of traffic flow characteristics.

4.5.1 Density versus speed

The model of the result of regression for density versus speed is indicated as follows:

$$k = 110.879 - 1.60814 u \quad (4.4)$$

The coefficient of correlation ($R^2 = 0.965338$) and the t_d value of constant is 131.837 (greater than t_{test} value is 1.686 with 95% confidence), and of the variable of speed is -57.3264 (absolute value greater than t_{test} value), the speed variable is the significance of the density. In this case, it is explained by the variability of the dependent variable, the traffic density in this case. The adjusted R_a^2 value for Equation

(4.4) is 0.965044. The result of regression for traffic density versus traffic speed equation is shown as Figure 4.3. This model indicates when the critical density at maximum flow (capacity) equals to 55.74 vehicles per kilometer per lane. Then coincides with the critical speed 34.29 kilometers per hour at that point of capacity of bottleneck, and the flow rate is 1854 vehicles per hour per lane. The jam density equals to 110.879 vehicles per kilometer per lane and then the traffic flow rate and speed equals to zero. The results are shown as Figure 4.3 and Figure 4.2.

4.5.2 Density versus flow rate

The model of the result of regression for the density versus flow rate is indicated as follows:

$$q = 149.567 + 61.7085 k - 0.558381 k^2 \quad (4.5)$$

The coefficient of correlation ($R^2 = 0.845873$) and the t_d value of constant are 1.7393 (the value greater than t_{test} value is 1.686 with 95% confidence); of the variable of density are 15.1115 (greater than t_{test} value); and of the variable of density square is -18.09 (absolute value greater than t_{test} value). The density variable is significant to the flow rate. In this case, it is explained by the variability of the dependent variable, the traffic flow rate in this case. The adjusted R_a^2 value for Equation (4.5) is 0.843239. The result of regression for the traffic density versus the traffic flow rate equation is shown as Figure 4.2. The model indicated that the critical density is 55.74 vehicles per

kilometer per lane, which coincides with the flow rate at 1854 vehicles per hour per lane (capacity).

4.5.3 Speed versus flow rate

The model of the result of regression for the speed versus flow rate is indicated as follows:

$$q = 108.87 u - 1.58715 u^2 \quad (4.6)$$

The coefficient of correlation ($R^2 = 0.904596$) and the t_d value of variable of speed are 24.8847 (greater than t_{test} value is 1.685 with 95% confidence), and of the variable of speed square is -20.7316 (absolute value greater than t_{test} value). The speed variable is significant to the flow rate. In this case, it is explained by the variability of the dependent variable, the traffic flow rate in this case. The adjusted R_a^2 value for Equation (4.6) is 0.902779. The result of regression for the traffic speed versus the traffic flow rate equation is shown as Figure 4.1. The model indicated the critical speed is 34.29 kilometers per hour, which coincides with the flow rate at 1854 vehicles per hour per lane (capacity).

4.6 Discussions

Summary, the linear speed-density regression model is $k = 110.879 - 1.60814 u$ (it also equal to $u = 68.948 - 0.62184 k$), where the coefficient of the correlation $R^2 = 0.965338$. The regression model form and trend are similar to that of

Greenshield's Linear Speed-Concentration model (Gerlough and Huber, 1975). Our model is simple to use and several investigators have found good correlation between the model and field data.

Second, the flow rate-density regression model is $q = 61.7085 k - 0.55838 k^2 + 149.567$, where the coefficient of correlation $R^2 = 0.845873$. The regression model form and trend are similar to the special flow-density model for freeway (Drew, 1975). Whereas most stream flow models are used to describe one-lane flows, it is possible to develop models describing the total flow on one roadway of a freeway.

Third, the flow rate -speed regression model is $q = 108.87 u - 1.58715 u^2$, where the coefficient of correlation $R^2 = 0.904596$. The regression model form and trend are similar to the Greenshield's speed-flow function that was fitted to the Chicago Data (Drake *et al.* 1967).

4.7 Summaries

The general objective of this research is to study the traffic flow characteristics of weaving vehicles in a freeway weaving section. Efforts are spent on analyzing the interrelation of traffic flow characteristics, as traffic speed, density, and traffic flow rate.

The traffic flow characteristics of weaving traffic on congested traffic flow conditions in a separated connecting C-D roadway on the freeway weaving section is

studied by analyzing the three basic factors as traffic density, traffic speed, and traffic flow rate characteristics of weaving vehicles. On the basis of the statistical analysis results, the following major conclusions were drawn:

1. Form and trend of regression models in the fundamental traffic flow characteristics (includes traffic flow rate, density, and speed) of weaving vehicles appear to fit in Greenshield model. The results are showing the basic traffic flow pattern that pattern is no influence on increasing lane changing and weaving activities, that is to be similar to the between of traffic in basic freeway sections. However, specific values obtained for weaving traffic such as the critical density, which is 55.74 vehicles per kilometer per lane; the result is higher than the Taiwan HCM (THCM, 2001) and HCM (2000). The critical speed is 34.29 kilometers per hour; and the maximum flow rate (capacity) is 1854 vehicles per hour per lane. These results showed that the values of traffic variables including the traffic flow rate and traffic speed are lesser than the Highway Capacity Manual (HCM, 2000) and the traffic density is higher than that of the HCM.
2. The traffic flow characteristics in the weaving section were focusing on the frontage roadway facilities with ramp onto non-congestion traffic conditions. This study investigated the macroscopic models of traffic flow characteristics

on the weaving section of a separated connecting C-D roadway in the Taiwan National Freeway Systems, and found a comprehensive relationship among the traffic speed, density, and flow rate under congestion traffic condition.

3. Traffic density and traffic speed appeared to be better parameters for describing the behavior of weaving traffic than traffic flow rate. The regression analysis appeared to be the constant coefficients of regression model that coefficients of constant indicate significant difference, with the 95% confidence, between the traffic flow rate and the traffic density and traffic speed. However, the coefficients of the first variables and secondary variables (density or speed) appear higher, which is significant with the 95% confidence. Generally, the R^2 values for these regression models are very high, which ranges from 0.845873 to 0.965338. It implies that a considerable portion of the correlation of traffic flow characteristics such as traffic density, traffic speed, and traffic flow rate are excessively influenced by these factors considered in the regression variables.

CHAPTER 5

Relationships between Lane-changing and Traffic Flow

Characteristics

Weaving flow on the separated connecting collector-distributor (C-D) roadway and freeway weaving sections have many different characteristics, including four main strategies with driven vehicles, that are dynamic traffic flow situation. These main strategies include car following, gap-acceptance, lane-changing, merging and diverging control, etc. This study focuses on analysis for the relationships of traffic speed, traffic flow rate, traffic density and lane changes.

All lane changing rules consist of two parts, a reason or trigger criterion (“Do I want to change the lane ?”) and a safety criterion (“Is it safe to change the lane ?”). Only if both criteria are fulfilled, the lane is changed (Chowdhurt, Wolf, and Schreckenberg, 1997). This section discusses the correlation of the lane-changing characteristics (lane-changing frequency) vs. traffic speed and traffic density, respectively (Cho and Tsai, 2003).

5.1 Lane-Changing Frequency and Traffic Speed

This section first discusses the relationships among the traffic speed and lane-changing frequency observed in the study location—separated connecting C-D roadway, followed by an estimation of the correlation between each basic speed variable

and the microscopic lane-changing characteristics (lane-changing frequency). Note that the traffic speed and lane-changing frequency for each time period (it's 1-minute) is measured and calculated directly from field data. Mainly for illustration Figure 5.1 presents the relationship among the traffic speeds over the observed periods. As commonly seen in high weaving traffic, the traffic speed appears to increase before in 33.56 kilometers per hour with lane-changing frequency is 22 numbers. But exceeding the point then the speed appears to decrease.

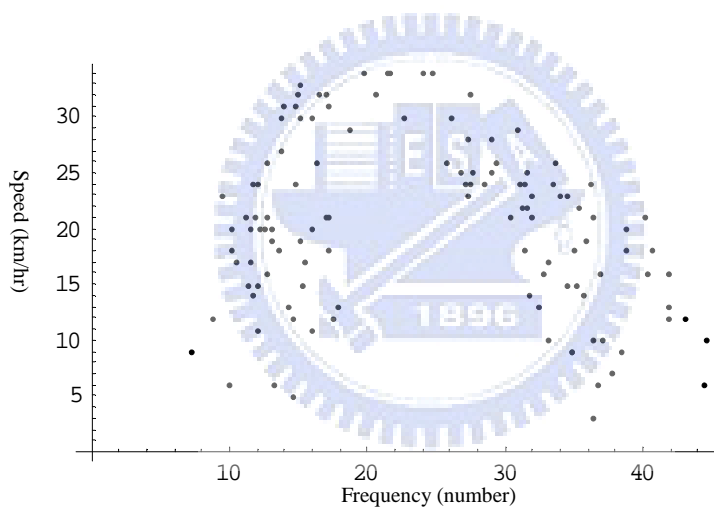


Figure 5.1 The diagram of traffic speeds vs. frequency of lane changes

With respect to the relations between the lane-changing characteristics and traffic speed variables, the regression results of lane-changing frequency versus each of the speed variable shows as Table 5.1 and Figure 5.2.

$$u = -250.259 + 198.0637 x_1^{0.15} - 0.808035 x_1^{1.25}$$

The R^2 value for this regression model is 0.349038 and the t_d test value is -6.65855 for first constant variable, and second variable is 7.288767, and third variable

is -7.73793 (the absolute t_d value greater than $t_{test} = 1.9819$ with 95% confidence belongs to significant level). Unfortunately, it can be seen that despite the relatively low R^2 for this case, the coefficients of determination are still rather low.

Table 5.1 The regression results of traffic speeds vs. frequency of lane-changes

Description	Variables	Estimated	SE	Ttest
Parameter table	Constant	-250.259	37.5846	-6.65855
	$x_l^{0.15}$	198.0637	27.1738	7.288767
	$x_l^{1.25}$	-0.808035	0.10442	-7.73793
Rsquared	R^2	0.349038		
Adjusted Rsquared	R_a^2	0.337911		
EstimatedVariance		38.8417		

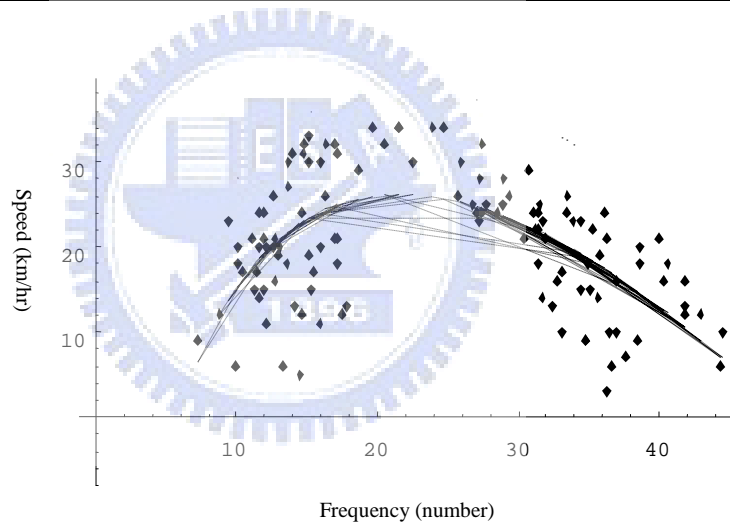


Figure 5.2 Regression results of traffic speeds vs. frequency of lane changes

5.2 Lane-Changing Frequency and Traffic Density

The second part discusses the relationships among the traffic density and lane-changing frequency observed in the study location, followed by an estimation of correlation between each basic density variable and the lane-changing characteristics (lane-changing frequency). Mainly for illustration Figure 5.3 present the relationship

among the traffic density over the observed periods. As seen in high weaving traffic, the relationship of the traffic density and the lane-changing frequency appears to increase in low densities, that range is from 23.74 vehicles per kilometer to 131.60 vehicles per kilometer, the lane-changing frequency is from 3 numbers to 34 numbers. The maximum lane-changing rate or frequency is at small densities. Another one, the relationship of the traffic density and the lane-changing frequency appears to decrease in high densities that range is from 149.71 vehicles per kilometer to 270.57 vehicles per kilometer; the lane-changing frequency is from 34 numbers to 5 numbers.

With respect to the relations between the lane-changing characteristics and traffic density variables, the regression results of lane-changing frequency versus each of the traffic density variable in low-density group show as the Table 5.2.

$$k = 5.684 + 0.0285 x_l^{1.45}$$

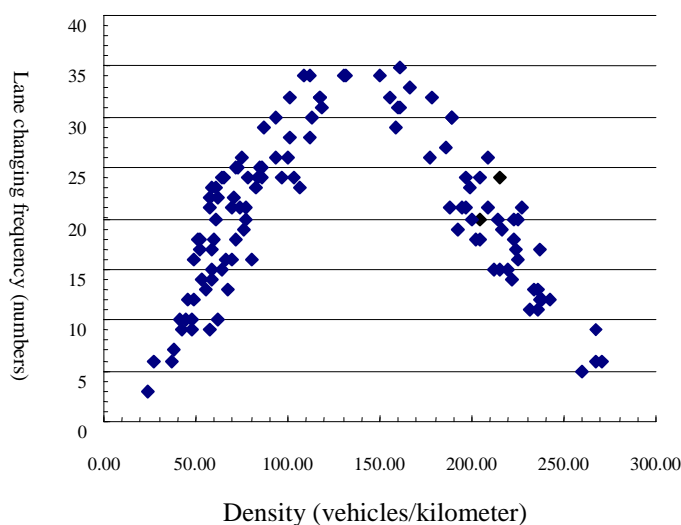


Figure 5.3 The diagram of traffic densities vs. frequency of lane changes

Table 5.2 The regression results of traffic densities vs. frequency of lane changes in low-densities group

Description	Variables	Estimated	SE	Ttest
Parameter table	Constant	5.68415	0.996005	5.70695
	$x_l^{1.45}$	0.02849	0.001755	16.2328
Rsquared	R^2	0.794875		
Adjusted Rsquared	R_a^2	0.791859		
EstimatedVariance		12.5822		

The $R^2 = 0.7948$ and adjusted $R_a^2 = 0.7918$. The $t_d = 5.70695$ and 16.2328 for the constant and x_l variable (the t_d value greater than $t_{test} = 1.9976$ with 95% confidence). It can be seen that despite the relatively higher R^2 and coefficient of determination for this case, the t_d value at the 95% significant level is significantly with the constant and x_l variable.

The regression results of lane-changing frequency versus each of the traffic density variable in high-densities group show as the Table 5.3

Table 5.3 The regression results of traffic densities vs. frequency of lane-changes in high-density group

Description	Variables	Estimated	SE	Ttest
Parameter table	Constant	70.301	3.22045	21.8295
	x_l	-0.241	0.01532	-15.751
Rsquared	R^2	0.837897		
Adjusted Rsquared	R_a^2	0.834520		
EstimatedVariance		9.48271		

$$k = 70.301 - 0.241 x_l$$

The $R^2 = 0.8379$ and adjusted $R_a^2 = 0.8345$. The $t_d = 21.8295$ and -15.751 for constant and x_l variable (the absolute t_d value greater than $t_{test} = 2.0128$ with 95% confidence). It can be seen that despite the relatively high R^2 and coefficient of

determination for this case, the t_d value at the 95% significant level is significantly with the constant and x_l variable. Same as the relationship on the low-density group in this case.

How frequently do the vehicles change lane in this study, The average frequency of lane changing per vehicle per unit time (1-minute) has been plotted against the total density of vehicles in Figure 5.3. In this case study, the average frequency of lane changing of vehicles per unit time exhibits a maximum at a moderate density. The reasons for this type of variation with density are as follows. At very low densities there is enough space in front and a typical vehicle very rarely feels the need for a lane changing. And at very high densities most of the attempts of lane changing are unsuccessful either because of close proximity of other vehicles in front or behind the empty neighboring site on the other lane.

5.3 Lane-Changing Frequency and other Traffic Flow Variables

5.3.1 Exploratory analysis

This section first reports the relationships among traffic speed, density, and flow rate observed in the test site, followed by an estimation of the correlations between each basic flow variable and the macroscopic lane-changing characteristics as lane-changing frequency.

Note that the average flow ratio (AFR) for each time period j is calculated by

dividing the lane flow of the source lane with the lane flow of the target lane. Because vehicles have only one choice from the inside lane to the outside lane or from the outside lane to the inside lane, the flow ratios are estimated by proportioning the individual flow ratios for the two possible lane changes with the relative number of lane changes in each direction, and are defined as follows (Chang and Kao, 1991):

$$\text{Source lane} = \text{inside lane} : AFR = \frac{F_{ij}}{F_{oj}} \times NL_{ioj}$$

$$\text{Source lane} = \text{outside lane} : AFR = \frac{F_{oj}}{F_{ij}} \times NL_{oij}$$

Where F_{nj} denotes the flow rate in lane n (including inside lane i and outside lane o) observed during time period j , and NL_{nmj} represents the number of lane changes from lanes n to m during the time interval j . The source and target lanes used hereafter refer to the origin and destination of a successful lane-changing maneuver. The average density ratios (ADR) and average speed ratios (ASR) are calculated in the same manner as AFR. Note that the average density and density ratio were both measured directly from videotapes, instead of estimating from speed and flow.

To further analyze the relations among all variables used in the exploratory analysis, a Pearson correlation matrix is computed and reported in Table 5.4.

Table 5.4 Pearson correlation coefficient matrixes for key traffic flow variables

Variables	AF	AD	AS	AHD	VHD	AFR	ADR	ASR
AF	1.0000	0.8516	-0.3896	-0.0648	-0.0443	0.4038	0.3780	0.8279
AD		1.0000	-0.8055	-0.2083	-0.0170	0.1056	0.1162	0.6776
AS			1.0000	0.1959	0.0042	0.1723	0.2190	-0.4337
AHD				1.0000	0.0751	0.3226	0.2643	0.0639
VHD					1.0000	-0.0125	-0.0018	0.0328
AFR						1.0000	0.9479	0.3426
ADR							1.0000	0.2044
ASR								1.0000

As expected, the average density exhibits a significant positive correlation (0.8516) with the average flow rate, but correlate negatively (-0.2083) with the average headway. A similar level of correlation also exists between the average density and average speed ratio (0.6776), and between the average flow ratio and average density ratio (0.9479), and between average flow rate and average speed ratio (0.8279). Besides, the variance of headway, as expected, is nearly independent from all variables.

With respect to the relations between the lane-changing characteristics and key traffic flow variables, Table 5.5 summarizes the regression results of lane-changing frequency (LCF) versus each of the variables defined previously. Among those, variables such as average density, average flow rate, average flow ratio, and average speed ratio, exhibit especially high correlation with both the frequencies of lane changes. For example, both the average density and the average flow rate yield linear relations with the number of lane changes (see Figure 5.4 and 5.5).

Table 5.5 Summaries of the relations between the frequency of lane changes (LCF) and key traffic flow variables

Relationship	R ²
LCF = -5.151 + 1.0185AD	0.6108
LCF = 154.824 + 26.343AF	0.9059
LCF = 54.142 - 0.520AS	0.3460
LCF = 1.8877-0.01341AHD	0.3306
LCF = -4.1646+0.0625VHD	0.1667
LCF = -126.67 + 4.6187AFR	0.5174
LCF = -57.575 + 2.955ADR	0.3716
LCF = -0.393 + 0.523ASR	0.8401

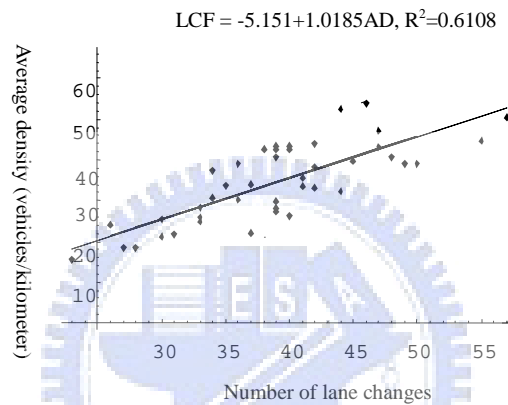


Figure 5.4 Regression results of average densities vs. number of lane changes

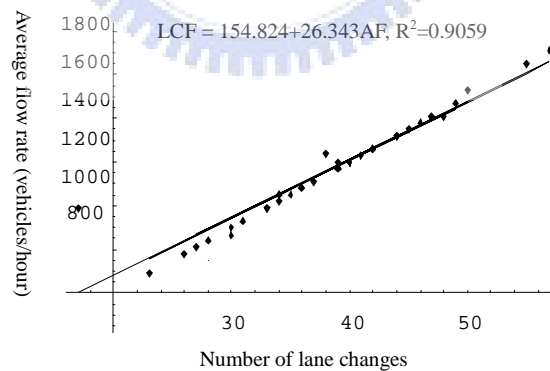


Figure 5.5 Regression results of average flow rates vs. number of lane changes

It is interesting to note that in all exploratory analysis, the correlations between the average speed and LCF are relatively lower than those computed against the average flow or density (see Table 5.5). This may be attributable to the unique feature of

congested traffic flows because the average prevailing lane speed, except for inside lane, during the observed periods was significantly affected by the variation in the average lane density, and was mostly below the speed limit. Hence, from a macroscopic view, the average lane speed may not be a critical variable associated with the lane-changing characteristics in congested traffic flows, even though an increase in the cruising speed is the main incentive for an individual to change lanes.

A further exploration of such interrelations was conducted, focusing on the difference and the ratio of variables in each lane versus the lane-changing frequency. It was found that using the difference in basic traffic flow characteristics as explanatory variables yielded the R^2 values is from 0.9069 to 0.3460 (see Table 5.5) and significant parameters for all cases. In contrast, the use of variable ratios between each lane mostly achieved significant level of fit, where the R^2 value for AFR and ASR against LCF at test site is 0.5174 and 0.8401.

In summary, among the variables being studied, basic traffic variables such as the average lane flow and density seems to be the two best explanatory variables for the lane-changing frequency. However, these two variables are highly correlated (see Table 5.4) and, as such, will not be used simultaneously in the model building. Similarly, the headway mean and variance are most likely candidates for use in constructing lane-changing prediction models, and will be considered only one at a time.

5.3.2 Lane-changing frequency models

This section further explores the relation between the lane-changing frequency and key traffic flow variables based on the exploratory analysis. A brief description of the modeling methodology is presented first, followed by the discussion of several model formulations and estimation results.

5.3.2.1 Estimation methodology

Let n_i denote the number of vehicles observed during the i th time interval for a given lane, and y_i represent those out of n_i changing lanes. The observations thus become the form $\frac{y_1}{n_1}, \dots, \frac{y_N}{n_N}$, a set of variables representing the lane-changing frequency of vehicles during each interval. As noted in the preliminary analysis, these variables vary with the traffic conditions within and between lanes. Their interrelation can thus be stated as:

$$\frac{y_i}{n_i} = p_i = \sum_{j=1}^P x_{ij} \beta_j + \varepsilon_i \quad i = 1, \dots, N \quad (5.1)$$

Note that the application of commonly used linear models in the above specification will result in a biased estimation because the error term ε_i is generally assumed to follow a normal distribution, and, as such, the probability p_i will lie between $(-\infty, \infty)$, which is inconsistent with the laws of probability. Hence, a generalized binary model is used to transform the model structure, and map the unit interval $(0, 1)$ for p_i onto the real line $(-\infty, \infty)$ (Chang and Kao, 1991).

Despite a wide choice of transform functions available in the literature (McCullagh and Nelder, 1983), the logit function appears to be the most convenient, and is employed in this study. Equation (5.1) is thus transformed as:

$$\ln\left(\frac{p_i}{(1-p_i)}\right) = Y_i = \sum_{j=1}^P x_{ij}\beta_j + \varepsilon_i \quad (5.2)$$

5.3.2.2 Model application

Consider a given weaving section lane segment of separated connecting C-D roadway on which the frequency of lane changes in every 1-minute interval varies with the traffic conditions within and between the inside and outside lane segments. Because each individual driver based on his own desired and perceived surrounding traffic conditions takes the action of changing lanes independently, the occurrence of such events from a macroscopic perspective is stochastic in nature. A proper model to capture such a process thus needs to have both the stochastic and non-negative properties (because the observed lane-changing frequency cannot be negative). Among several commonly recognized statistical models, the Poisson process seems to fit the lane-changing behavior phenomenon reasonably well, because the sample space of a Poisson distribution is the set of non-negative integers. Conceivably, the mean of such a Poisson process shall not be a constant, but evolve with the traffic flow characteristics (Chang and Kao, 1991). It can thus be expressed as:

$$E(LCF_i) = \mu_i = f(z_{ij}) \quad i = 1, \dots, N \quad (5.3)$$

$$J = 1, \dots, P$$

With the generally presumed multiplicative relationship among explanatory variables,

Equation (5.3) can be restated as the following logarithmic form:

$$\mu_i = \prod_{j=1}^P (z_{ij}) \alpha_j + \nu_i$$

and $\ln(\mu_i) = \sum_{j=1}^P c_j x_{ij} + \delta_i \quad i = 1, \dots, N \quad (5.4)$

5.3.2.3 Parameter estimation

Given the assumed Poisson nature, the probability of having y_i lane changes over

the i th interval is given by:

$$P_r(LCF_i = y_i) = \frac{(\mu_i)^{y_i} \times (\mu_i)^{y_i}}{(y_i)!} \quad (5.5)$$

The likelihood of having y_1, y_2, \dots, y_N lane changes over N observed intervals can thus be represented as follows:

$$L(Y) = \prod_{i=1}^N \frac{(\mu_i)^{y_i} \times (\mu_i)^{y_i}}{(y_i)!}, \quad i = 1, \dots, N \quad (5.6)$$

and the log transformation of equation (5-6) is:

$$\ln(Y) = \sum_{i=1}^N [\ln(\mu_i) + y_i \ln(\mu_i) - \ln(y_i!)] \quad (5.7)$$

where $Y = (y_1, y_2, \dots, y_N)$. Because $\ln(\mu_i) = \sum_{j=1}^P c_j x_{ij}$, $i = 1, \dots, N$, the log

likelihood function can further be stated as:

$$\ln(C) = \sum_{i=1}^N [(\sum_{j=1}^P c_j x_{ij}) + y_i \sum_{j=1}^P c_j x_{ij} - \ln(y_i!)] \quad (5.8)$$

where $C = (c_1, c_2, \dots, c_p)$, representing unknown parameters to be estimated.

The unknown parameters in the log likelihood function, as discussed in the previous studies, can be obtained by the standard maximum likelihood method.

5.3.2.4 Model formulation and estimation

Following the similar procedures stated in the previous section, four linear combinations, which yield relatively high goodness of fit, are presented below:

Model A1: $\ln \mu_i = a'_1 + b'_1 \ln(\text{AHD}_i) + c'_1 \ln(\text{ADR}_i) + d'_1 \ln(\text{ASR}_i)$

Parameter: (1.6150) (-0.0523) (0.1248) (0.5834)

t_d value: (10.3059) (-5.9550) (9.1499) (13.3025)

$R^2 = 0.7631$

Model A2: $\ln \mu_i = a'_2 + b'_2 \ln(\text{AD}_i) + c'_2 \ln(\text{AFR}_i) + d'_2 \ln(\text{AHD}_i)$

Parameter: (2.9113) (0.1672) (0.0944) (-0.0949)

t_d value: (23.3916) (7.1305) (4.3701) (-8.1384)

$R^2 = 0.5811$

Model A3: $\ln \mu_i = a'_3 + b'_3 \ln(\text{AS}_i) + c'_3 \ln(\text{AFR}_i) + d'_3 \ln(\text{AHD}_i)$

Parameter: (4.0623) (-0.1040) (0.1229) (-0.3951)

t_d value: (20.1169) (-3.2999) (5.4008) (-6.4362)

$R^2 = 0.4407$

Model A4: $\ln \mu_i = a'_4 + b'_4 \ln(\text{AF}_i) + c'_4 \ln(\text{ADR}_i) + d'_4 \ln(\text{AFR}_i)$

Parameter: (-4.0943) (1.0000) (-2.39E-13) (1.14E-13)

$$t_d \text{ value: } (-4.53E+12) (8.13E+12) (-2.4573) \quad (1.0436)$$

$$R^2 = 1.0000$$

Model A1 includes the average headway, the average density ratio, and the average speed ratio, where model parameter for these three variables is all statistically significant at the 5% level (see the t_d value). The sign of its parameter of average headway (AHD) shows an expected negative relationship between the LCF. This is reasonable because a large average density ratio generally exists only in heavy traffic where vehicles are closely distributed and move in-groups (or platoons). A higher ADR value thus implies a higher level of concentration (or congestion), and more restraint for drivers to travel restrainedly at their lower speed. Consequently, more lane changes may take place (see Table 5.4). Such a relation is further confirmed by a positive sign for the average density ratio (ADR) where the LCF increases as the ADR does, indicating that more lane changes may occur at a higher level of traffic density.

Model A2 is similar to model A1 expect that the variable AFR replace ADR and AD replace ASR. These two variables have the high correlation in between (see Table 5.4). Model A2 expectedly yields these parameter values as 0.1672, 0.0944, -0.0949 respectively, and a slightly higher level of fit (0.7631 vs. 0.5811).

Model A3 includes the average speed, average flow ratio, and average headway, where model parameter for these three variables are all statistically significant at the 5%

level (see t_d values), that result is similar to Model A1 and Model A2. The sign of its parameters of AFR showing an expected positive relationship between the numbers of lane changes (LCF) and AD shows an expected negative relationship between the numbers of lane changes (LCF). It indicates that on congested freeway weaving sections the decrease in average headway and traffic speed is the result of an increase in traffic flow ratio.

Model A4 is similar to Model A3 expect that the variable becomes the average density ratio (ADR) instead of the average flow ratio (AFR). Due to the high correlation between these two variables ($R=0.9479$, see Table 5.4), which is relatively easier to measure, it is used in model A4. As can be seen from the sign of the parameter for AFR, the lane-changing frequency exhibits a positive correlation with the average flow ratio in congested traffic conditions. In conjunction with AFR and ADR, this model seems to yield as good a fit as those using average traffic flow and speed variable, and a slight higher level of fit (0.4407 vs. 1.0000).

Regarding the predictability of the above LCF model, it is necessary to note that those, as stated earlier, are exploratory in nature due to the availability of empirical data. Such a limitation can be seen from the residual distributions for Model A1 and Model A3 illustrated in Figure 5.6, and Figure 5.7, respectively. In model A1, approximately 80% of the total observations have more than 20% prediction errors. In model A3,

approximately 90% of the total observations have more than 10% prediction errors.

However, considering the availability of so few data and the result that more than 90% of the observations fall within the 20% boundaries, this approach of using econometric models to characterize macroscopic lane changing behavior seems quite promising.

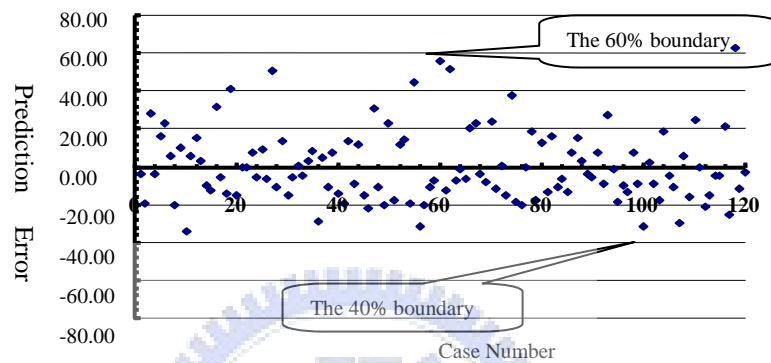


Figure 5.6 The distribution of prediction errors for Model A1.

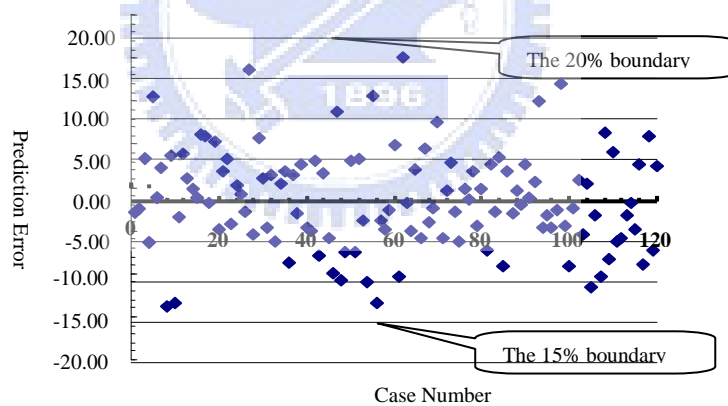


Figure 5.7 The distribution of prediction errors for Model A3.

5.4 Summaries and Discussions

This study presented the empirical results of the lane-changing frequency in congested separated connecting C-D roadway weaving traffic flows. An exploratory analysis was conducted first to investigate the interrelations between the lane-changing characteristics and key traffic flow variables. Although only limited observations were

available in the study, the results clearly indicated that the distribution of headways and the density and flow ratios between neighboring lanes is principal factors affecting the lane-changing behavior. Such a relation was further confirmed with a generalized linear model.

The model focused on estimating the frequency of lane changes, where occurrences of lane changing were assumed to follow a Poisson process with a time-varying mean. The model with the standard maximum likelihood estimation yielded expected signs for all parameters and achieved reasonable level of fit.

It is fully recognized that the reported results are based only on limited observations from one test site, which may not be sufficient to represent the general lane-changing characteristics. Further studies to collect more data and identify some additional critical factors such as the geometric conditions will be necessary. Besides, the macroscopic lane-changing characteristics may differ significantly on heavily congested weaving sections because a driver's desire to change lanes may vary with the traffic conditions. This subject along with the mechanisms governing an individual driver's lane-changing decisions are vital to the understanding of weaving sections of freeway traffic characteristics, and will also be addressed in our future research.

CHAPTER 6

Capacity Analysis on the Weaving Sections

This section focuses on a typical separated connecting collector-distributor (C-D) roadway weaving section that is part of a cloverleaf interchange. While focusing mainly on establishing a capacity estimation model for separated connecting C-D roadway weaves, this study applies the modified renewal processes of gap acceptance (Banks, 2003; Ross, 1996; and Cho and Tsai, 2009) and uses the linear optimization methodology used in the capacity analysis for the separated connecting C-D roadway weaving sections. Applying the renewal process to the gap acceptance method can model the high level of lane-changing activity typically present in weaving sections, significantly influencing capacity.

6.1 Proposed Methodologies and Models

The first part of this section presents definitions and assumptions used in this study. After clarifying the terminology and assumptions the models themselves are presented and discussed.

6.1.1 Collector-distributor roadway weave capacity linear programming model

This section focuses on the loop-ramp weave segments of the separated connecting collector-distributor (C-D) roadway. Figure 6.1 shows a C-D roadway weave segment, together with the symbols indicating various traffic streams. Traffic demands

at a C-D roadway weaves section can be separated into two traffic streams, as shows in Figure 6.1.

HCM (2000) defines capacity as the maximum vehicle flow that can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given period based on existing roadway, traffic, and control conditions. This study assumes the capacity of C-D roadway weaves to be the sum of the following two parts.

1. The capacity of the inside weaving lane (lane 1), and
2. The capacity of the outside weaving lane (Auxiliary lane (lane 0)).

An analysis of traffic behavior based on 1-minute intervals that few vehicles make more than two lane changes within the length of the weaving segment, and attempting to model lane-changing behavior involving more than two changes would unnecessarily complicate the methodology while providing minimal benefits in terms of improved accuracy. Generally, traffic demand from the loop-ramp and C-D roadway lanes of a C-D roadway weaving section can be divided into several components, as shown in Figure 6.1.

The capacity of C-D roadway areas comprises the maximum flow rate of vehicles in the two lanes. Furthermore, the capacity of the weaving segment is estimated using the following procedure. Based on consideration of the C-D roadway lanes of the weaving section alone, the maximum traffic flows are divided into four components, as

shown on Figure 6.1.

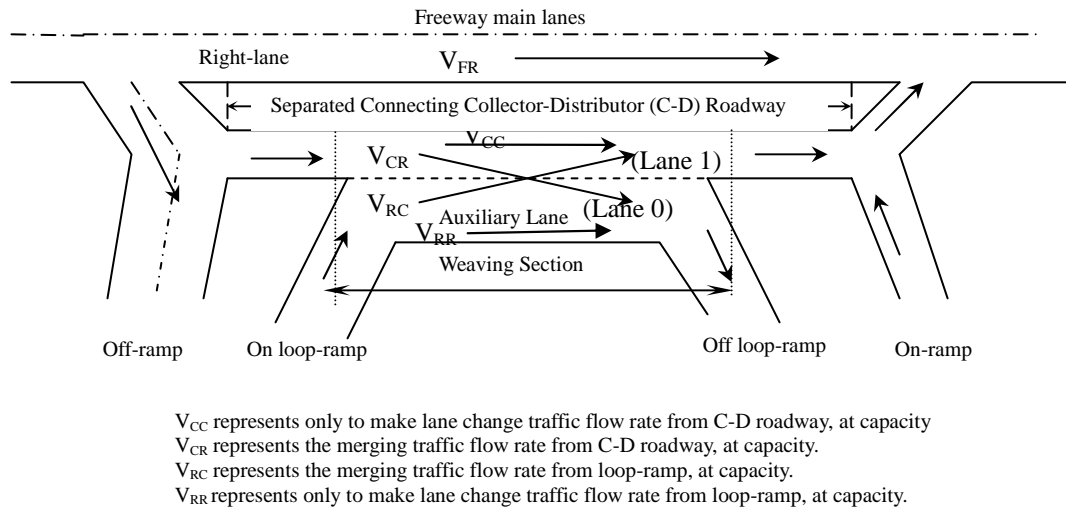


Figure 6.1 Sketches of the weaving flows on a separated connecting C-D roadway

Based on the configuration shown in Figure 6.1, the capacity of the two lanes can be mathematically expressed as

$$c(\text{Capacity}) = \text{Max}(V_{CC} + V_{RC} + V_{RR} + V_{CR}) \quad (6.1)$$

The capacity of the two lanes influenced by weaving can be estimated via Equation (6.1), using a linear optimization technique, together with some constraints.

The previous notation requires imposing the following constraints:

1. The capacity of the weaving section cannot exceed that of the equivalent basic C-D roadway segments. These constraints can be expressed as

$$V_{RR} + V_{RC} \leq BAC \quad (\text{Basic Auxiliary lane segment Capacity})$$

$$V_{CC} + V_{CR} \leq BCC \quad (\text{Basic C-D roadway segment Capacity})$$

2. Capacities depends heavily on the proportion of weaving traffic; W_0 and

W_1 are the ratios of weaving vehicles to total vehicles in the lane 0 and lane 1, respectively. Those ratios must remain the same as the respective ratio of the demands throughout the (iteration) process of capacity estimation. This constraint can be expressed as

$$\frac{V_{RC}}{(V_{RC} + V_{RR})} = W_0$$

$$\frac{V_{CR}}{(V_{CR} + V_{CC})} = W_1$$

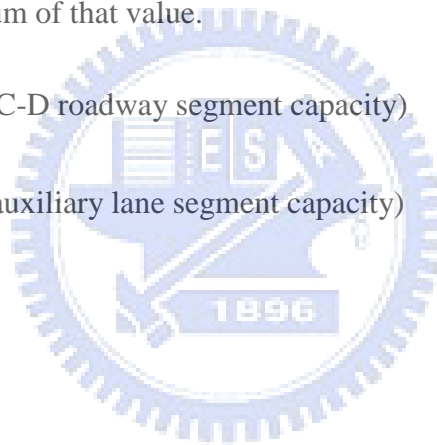
3. The value of each component specified in Equation (6.1) should not exceed their potential maximum of that value.

$$V_{CC} \leq BCC \text{ (Basic C-D roadway segment capacity)}$$

$$V_{RR} \leq BAC \text{ (Basic auxiliary lane segment capacity)}$$

$$V_{RC} \leq \text{Max}(V_{RC})$$

$$V_{CR} \leq \text{Max}(V_{CR})$$



The last two constraint values, $\text{Max}(V_{RC})$ and $\text{Max}(V_{CR})$, are estimated using gap acceptance theory. These variables represent the “potential capacity” for each of the traffic streams, and are related to gap availability, size and acceptance (Lertworawanich and Elefteriadou, 2003). Estimating these gap characteristics requires advance knowledge of the time headway distributions of traffic flows arriving from the C-D roadway, the loop-ramp, the critical lag, etc.

Based on the capacity equation and constraints, the final linear programming

optimization equations are expressed as follows:

$$\text{Objective function: } c(\text{Capacity}) = \text{Max}(V_{CC} + V_{RC} + V_{RR} + V_{CR})$$

$$\text{Subject to: Capacity constraint 1: } V_{RR} + V_{RC} \leq BAC$$

$$\text{Capacity constraint 2: } V_{CC} + V_{CR} \leq BCC$$

$$\text{Lane 1 volume ratio constraint: } \frac{V_{CR}}{(V_{CR} + V_{CC})} = W_1$$

$$\text{Loop-ramp lane (lane 0) volume ratio constraint: } \frac{V_{RC}}{(V_{RC} + V_{RR})} = W_0$$

$$\text{Bound: } V_{CC} \leq BCC$$

$$\text{Bound: } V_{RR} \leq BAC$$

$$\text{Bound: } V_{RC} \leq \text{Max}(V_{RC})$$

$$\text{Bound: } V_{CR} \leq \text{Max}(V_{CR}) \quad (6.2)$$

The proposed methodology is thus based on using gap acceptance parameter estimation to calculate the maximum lane changes for each traffic stream, and calculating capacity using the above linear programming optimization equations.

The first step in the problem solving procedure involves obtaining traffic-related parameters, traffic origin-destination demands, and volume estimates. Meanwhile, the second step involves estimating traffic speeds in each lane. The speeds of weaving vehicles in each lane are crucial parameters. Furthermore, the third step involves calculating the critical lag for the lane-changing processes using the equations of designed by Drew *et al.* (1967) and the value of parameters findings of Raff and Hart

(1950) (see Lertworawanich and Elefteriadou, 2003). Furthermore, the fourth step involves estimating the maximum lane changes using gap acceptance theory. All the parameters required for optimization Equation (6.2) can thus be obtained. The final step is calculating the capacity of the C-D roadway weaves using the linear programming. The remainder of this section explains the critical lag estimations, the development of the rear-lag distributions and, finally, the estimation of maximum lane changes.

6.1.2 Critical lag estimations

This section develops a methodology for estimating the ideal safe gap for merging processes on C-D roadway. The ideal safe gap is defined as the value within which merging vehicle can enter the C-D roadway stream without causing C-D roadway vehicle to reduce speed or change lanes. This study assumes the critical gap to be equivalent to the ideal safe gap, as estimated by Drew *et al.* (1967), Brilon *et al.* (1999) and Hagrind (2000), and the value of critical gap was finding by Raff and Hart (1950) that value is approximately 0.2 seconds less than the average critical gap (Lertworawanich and Elefteriadou, 2003).

The average acceleration of a typical vehicle, when approaching at a speed below that of the main traffic or in situations where the weaving vehicle must accelerate, is assumed to be a linear function of the speed of the merging vehicle. Using equations of motion, the ideal minimum gap for merging was based on the time required for safe

stopping and the time loss from acceleration, which can be expressed as:

2. Time loss from acceleration

$$\text{Time loss from acceleration} = \frac{u+v}{b \cdot v} + \frac{1}{b \cdot v} \left[\frac{a}{b} - v \cdot \ln \left(\frac{a-b \cdot v}{a-b \cdot u} \right) \right]$$

3. Time required for safe stopping

$$\text{Time required for safe stopping} = \frac{(L_c + L_r)}{v} + 2RT$$

The equation for estimating the ideal safe gap was expressed as (Drew *et al*, 1967):

$$TT = \frac{L_c + L_r}{v} + 2RT + \frac{u+v}{b \cdot v} + \frac{1}{b \cdot v} \left(\frac{a}{b} - v \cdot \ln \left(\frac{a-b \cdot v}{a-b \cdot u} \right) \right) \text{ with } u < v$$

The ideal minimum lag required for merging, which according to Raff and Hart (1950)

is 0.2 seconds less than the critical gap, can be expressed as

$$TT = \frac{L_c + L_r}{v} + 2RT + \frac{u+v}{b \cdot v} + \frac{1}{b \cdot v} \left(\frac{a}{b} - v \cdot \ln \left(\frac{a-b \cdot v}{a-b \cdot u} \right) \right) - 0.2 \text{ with } u < v \quad (6.3)$$

Parameters a and b must be estimated before determining the minimum safe lag using Equation (6.3). Based on experiences in Australia (Knox, 1964), the recommended values of a and $\frac{a}{b}$ are 4.8 mph/sec and 80 mph, respectively, and the recommended reaction time is 1 second. Notably, that these recommended values do not influence the overall structure of the methodology presented here.

6.1.3 Development of rear lags distributions

This section describes the method used to estimate the rear lag distribution. The rear lag distributions are then used to calculate the maximum lane-change according to gap acceptance theory. To estimate the maximum lane-changes from loop-ramp to the

C-D roadway, it is necessary to calculate the distribution of the rear lag $RL01$, as illustrated in Figure 6.2. Conversely, to calculate the maximum number of lane-changes from the C-D roadway to the loop-ramp, the distribution of the rear lag $RL10$, as shown in Figure 6.3, must be calculated.

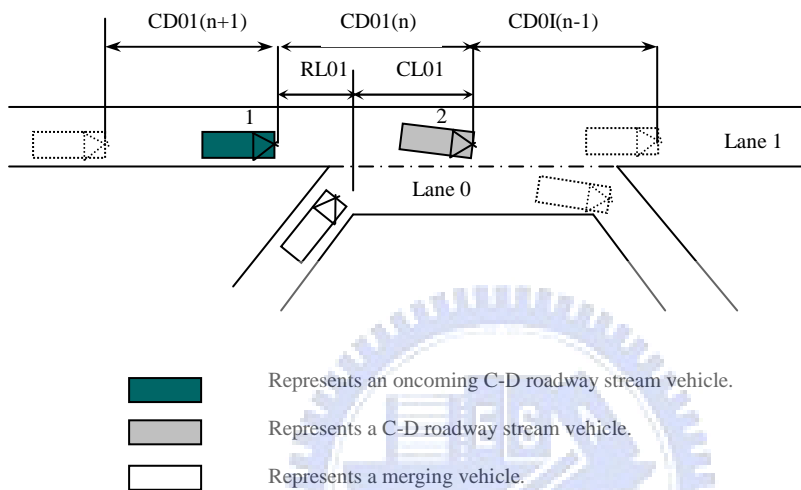


Figure 6.2 Rear lag $RL01$ of C-D roadway lane 1

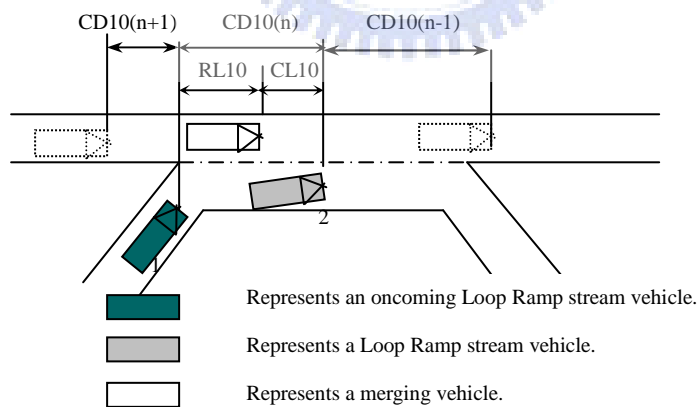


Figure 6.3 Rear lag $RL10$ of loop-ramp lane 0

The time headway distributions for the oncoming traffic stream in lanes 0 and 1 are represented as $f_{CD10}(x)$ and $f_{CD01}(x)$ respectively.

6.1.3.1 Determination of rear lags $RLOI$ for lane 1

Based on Figure 6.2, it is assumed that the arrival process of vehicles in lane 0 is independent of that of vehicles in lane 1 of the C-D roadway. Consequently, the rear lag distribution $RLOI$ depends on the intended destinations and actions of vehicles 1 and 2. This study considers four possible situations at the merge gore which are specified as follows:

1. Vehicle 1 is weaving vehicle that wishes to weave from lane 1 to lane 0 and vehicle 2 is non-weaving vehicle. In this case, vehicle 1 weaves from lane 1 to lane 0 depending on gap availability in lane 0.
2. Both vehicles 1 and 2 are weaving vehicles that wish to weave from lane 1 to lane 0. In this case, vehicles 1 and 2 weave from lane 1 to lane 0 depending on gap availability in lane 0.
3. Vehicle 1 is non-weaving vehicle and vehicle 2 is a weave vehicle that wishes to weave from lane 1 to lane 0. In this case, vehicle 2 weaves from lane 1 to lane 0 depending on gap availability in lane 0.
4. Both vehicles 1 and 2 are non-weave vehicles.

The probability of vehicle 1 weaving (i.e., desiring to weave from lane 1 to lane 0) is W_1 (defined as the ratio of weaving to non-weaving flows in lane 1). Thus, the probability of vehicle 1 being a non-weaving vehicle is $(1-W_1)$. Moreover, vehicle 2

wishes to weave from lane 1 to lane 0 if the following conditions are fulfilled: (1) it is a weaving vehicle with a probability of W_1 , and (2) it is a non-weaving vehicle with a probability of Ω . Therefore, the corresponding probability that vehicle 2 wishes to weave from lane 1 to lane 0 is ΩW_1 , and the probability of vehicle 2 being a non-weaving vehicle is $(1-\Omega W_1)$.

Following the calculations, the distributions of rear lag $RL01$ for each of the four cases are combined with the corresponding probabilities, thus yielding the final distribution used in the next step of the methodology.

Case 1: Vehicle 1 is a weaving vehicle that wishes to weave from lane 1 to lane 0, while vehicle 2 is a non-weave vehicle.

In Figure 6.2, vehicle 1 weaves from lane 1 if an appropriate lag is available in lane 0. Two sub-cases thus exist: (1) vehicle 1 can identify a lag and weave from lane 1 to lane 0, and (2) vehicle 1 can be forced to weave from lane 1 to lane 0. These two sub-cases are described as follows.

Based on renewal, the distribution of the rear lag $RL01$ is expressed as

$$f_{RL01,1}(x) = \frac{(1 - F_{CD01}(x))}{E(x)}$$

where $f_{RL01,1}(x)$ denotes the distribution of $RL01$ for case 1, $F_{CD01}(x)$ represents the cumulative distribution function (cdf) for the time headways of traffic in lane 1, and $E(x)$ is the expected time headway of the traffic in lane 1.

Case I-1: Vehicle 1 has a lag available to weave from lane 1 to lane 0 and vehicle 2 is forced to weave from lane 1 to lane 0.

The probability of the driver of vehicle 2 attempting to weave from lane 1 to lane 0 is assumed to be η , while the probability of vehicle 2 identifying a suitable lag is denoted as P_x and estimated as follows:

$$P_x = P(RL01 \geq \tau) = \int_{\tau}^{\infty} f_{RL01}(x) dx$$

where $f_{RL01}(x)$ denotes the distribution of the rear lag $RL01$ defined as $f_{RL01}(x) = \frac{(1 - F_{CD01}(x))}{E(x)}$, and τ represents the critical lag.

The probability that vehicle 1 can weave from lane 1 to lane 0 is defined as follows:

$$P(CL01 \geq \tau) = \int_{\tau}^{\infty} f_{CL01}(x) dx = a$$

Where $f_{CL01}(x)$ is the distribution of front lag $CL01$, which is identical to $f_{RL01}(x)$, and τ represents the critical lag.

This situation thus occurs with a probability of $\eta P_x a$. Figure 6.2 shows that the gap available to the merging vehicle from the loop-ramp is the sum of the three consecutive time headways, $CD01(n-1)$, $CD01(n)$, and $CD01(n+1)$, each of which possesses the same distribution ($f_{CL01}(x)$). Since available rear lag is a function of this gap, it is necessary to first calculate the gap distribution:

$$F_{CD01(n+1)+CD01(n)+CD01(n-1)}(u) = \iiint_{x+y+z \leq u} f_{CD01}(x) f_{CD01}(y) f_{CD01}(z) dx dy dz$$

$$\rho_{RL01}(x) = \frac{(1 - F_{CD01(n+1)+CD01(n)+CD01(n-1)}(x))}{E(x)}$$

where $E(x)$ denotes the expected value of the gap ($CD01(n+1)+CD01(n)+CD01(n-1)$), and $\rho_{RL01}(x)$ represents the distribution of the rear lag $RL01$ for the case under consideration.

Case 1-2: Although vehicle 1 forced to weave from lane 1 to lane 0, vehicle 2 can weave lane, since vehicle 1 cannot identify a sufficiently large lag.

The probability that the available lag for vehicle 1 is smaller than the critical lag must be calculated.

$$P(CL01 \leq \tau) = \int_{\tau}^{\infty} f_{CL01}(x) dx = 1 - a$$

where $f_{CL01}(x)$ is the distribution of the front lag $CL01$ and is identical to $f_{RL01,1}(x)$, and τ represents the critical lag.

The gap available to the merging vehicle denotes the sum of the two consecutive time headways, $CD01(n+1)$ and $CD01(n)$, each of which possesses the which share the same distribution, $f_{CL01}(x)$. The probability of this case occurring is $\eta P_x(1-a)$. The distribution of $CD01(n+1) + CD01(n)$ can be estimated as follows:

$$F_{CD01(n+1)+CD01(n)}(u) = \int_0^u \int_0^{u-x} f_{CD01}(x) f_{CD01}(y) dx dy$$

$$\alpha_{RL01}(x) = \frac{(1 - F_{CD01(n+1)+CD01(n)}(x))}{E(x)}$$

where $F_{CD01(n+1)+CD01(n)}(x)$ denotes the cdf of the $CD01(n+1)+CD01(n)$, and $E(x)$ represents the expected value of the gap ($CD01(n+1)+CD01(n)$), and $\alpha_{RL01}(x)$ is the distribution of the rear lag $RL01$ for this case.

The gap available to the merging vehicle is the sum of the two consecutive time headways, $CD01(n)$ and $CD01(n-1)$, each both of which share the same distribution ($f_{CD01}(x)$).

To determine the distribution of the rear lag for case 1, the distribution of the rear lag $RL01$ for the two sub-cases must be combined with the corresponding occurrence probabilities, as follows:

$$F_{RL01,1}(x) = \eta P_x (a \int_0^x \rho_{RL01}(u) du + (1-a) \int_0^x \alpha_{RL01}(u) du)$$

$$f_{RL01,1}(x) = \frac{(dF_{RL01,1}(x))}{dx}$$

where $F_{RL01,1}(x)$ denotes the cdf of the rear lag $RL01$ of case 1, and $f_{RL01,1}(x)$ represents the distribution of the rear lag $RL01$ of this case.

Case 2: Vehicles 1 and 2 both wish to weave from lane 1 to lane 0.

In Figure 6.2, vehicle 2 attempts to weave from lane 1 to lane 0 or merge into the loop-ramp traffic stream to proceed to the off-loop ramp. The probability of vehicle 2 being able to perform this maneuver is a function of the gap availability in lane 0. Two sub-cases thus are considered.

Case 2-1: Vehicle 2 perceives a sufficient lag and can weave from lane 1 to lane 0.

The probability that a lag equals or exceeds the critical lag is calculated as follows:

$$P(CL01 \geq \tau) = \int_{\tau}^{\infty} f_{CD01}(x) dx = a$$

Where $f_{CL01}(x)$ denotes the distribution of front lag $CL01$ and is identical to $f_{RL01,1}(x)$ based on the renewal process, and τ represents the critical lag.

From Figure 6.2, the gap available to the merging vehicle from lane 0 is the sum of the two consecutive time headway, $CD01(n)$ and $CD01(n-1)$, each of which have the distribution ($f_{CD01}(x)$). The distribution of the gap, $CD01(n) + CD01(n-1)$, must be calculated before calculating the distribution of $RL01$. The equation for estimating the distribution of $CD01(n) + CD01(n-1)$ is as follows:

$$F_{CD01(n)+CD01(n-1)}(u) = \int_0^u \int_0^{u-x} f_{CD01}(x) f_{CD01}(z) dx dz$$

$$\alpha_{RL01}(x) = \frac{(1 - F_{CD01(n)+CD01(n-1)}(x))}{E(x)}$$

where $E(x)$ denotes the expected value of gap ($CD01(n) + CD01(n-1)$), and $\alpha_{RL01}(x)$ represents the distribution of the rear lag $RL01$ of this case.

Case 2-2: Vehicle 2 is unable to find a sufficient large lag; therefore, it is forced to weave from lane 1 to lane 0. In this case, the probability that the available lag for vehicle 2 is below the critical lag is calculated as

$$P(CL01 \leq \tau) = \int_0^{\tau} f_{CD01}(x) dx = 1 - a$$

where $f_{CL01}(x)$ denotes the distribution of front lag $CL01$ and is identical to $f_{RL01,1}(x)$ based on the renewal process, and τ represents the critical lag.

From Figure 6.2, the gap available to the merging vehicle from the loop-ramp is the random variable $CD01(n)$. Consequently, the distribution of the rear lag for this case

resembles that in case 1 or $f_{RL01,1}(x)$.

The distribution of the rear lag $RL01$ of each sub-case must be combined with the corresponding occurrence probability, as follows:

$$F_{RL01,2}(x) = (a)\left(\int_0^x \alpha_{RL01}(u)du\right) + (1-a)\left(\int_0^x f_{RL01,2}(u)du\right)$$

$$f_{RL01,2}(x) = \frac{dF_{RL01,2}(x)}{dx}$$

where $f_{RL01,2}(x)$ denotes the distribution of the rear lag $RL01$ for this case.

Case 3: Vehicle 1 wishes to weave lanes, and vehicle 2 wishes to weave from lane 1 to lane 0.

Figure 6.2 show that vehicle 2 weaves from lane 1 to lane 0 provided an appropriate lag exists in lane 0. Again, two possible sub-cases exist: (1) vehicle 2 can find an adequate lag and weave to lane 0, and (2) vehicle 2 may be force to weave from lane 1 to lane 0. Vehicle 1 is willing to weave lanes. The two sub-cases are as follows.

Case 3-1: Vehicle 1 can weave lane and vehicle 2 can weave from lane 1 to lane 0.

The probability that the driver of vehicle 1 will weave lane is assumed to be η , and the probability that vehicle 1 will find a sufficient lag is denoted as P_x and estimated as follows:

$$P_x = P(RL01 \geq \tau) = \int_{\tau}^{\infty} f_{RL01}(x)dx$$

where $f_{RL01}(x)$ denotes the distribution of the rear lag $RL01$ defined as $f_{RL01}(x) = \frac{(1 - F_{CD01}(x))}{E(x)}$, and τ represents the critical lag.

The probability that vehicle 2 can weave from lane 1 to lane 0 is defined as follows:

$$P(CL01 \geq \tau) = \int_{\tau}^{\infty} f_{CL01}(x)dx = a$$

where $f_{CL01}(x)$ denotes the distribution of front lag $CL01$, which is identical to $f_{RL01,1}(x)$, and τ represents the critical lag.

Therefore, the probability this situation occurring is $\eta P_x a$. From Figure 6.2, the gap available to the merging vehicle from the loop-ramp is calculated by summing the three consecutive time headways, $CD01(n-1)$, $CD01(n)$, and $CD01(n+1)$, each of which has the same distribution ($f_{CD01}(x)$). Since an availability of a rear lag is a function of this gap, the distribution of the gap must first be calculated:

$$F_{CD01(n+1)+CD01(n)+CD01(n-1)}(u) = \iiint_{x+y+z \leq u} f_{CD01}(x)f_{CD01}(y)f_{cd01}(z)dx dy dz$$

$$\rho_{RL01}(x) = \frac{(1 - F_{CD01(n+1)+CD01(n)+CD01(n-1)}(x))}{E(x)}$$

where $E(x)$ represents the expected value of the gap ($CD01(n+1) + CD01(n) + CD01(n-1)$), and $\rho_{RL01}(x)$ is the distribution of the rear lag $RL01$ for this case.

Case 3-2: Although vehicle 1 is able to weave lane, vehicle 2 is forced to weave to lane 0 because it cannot identify a sufficiently large lag.

The probability that the available lag for vehicle 2 is smaller than the critical lag is calculated as:

$$P(CL01 \leq \tau) = \int_0^{\tau} f_{CD01}(x)dx = 1 - a$$

where $f_{CL01}(x)$ denotes the distribution of the front lag $CL01$ and is identical to $f_{RL01,1}(x)$

based on the renewal process, and τ represents the critical lag.

The gap available to the merging vehicle is calculated by summing of the two consecutive time headways, $CD01(n+1)$ and $CD01(n)$, each of which shares the same distribution, $f_{CD01}(x)$. This case has an occurring probability of (ηP_x) $(1-a)$. The distribution of the $CD01(n+1) + CD01(n)$ can be estimated as follows:

$$F_{CD01(n+1)+CD01(n)}(u) = \int_0^u \int_0^{u-x} f_{CD01}(x) f_{CD01}(y) dx dy$$

$$\alpha_{RL01}(x) = \frac{(1 - F_{CD01(n+1)+CD01(n)}(x))}{E(x)}$$

where $F_{CD01(n+1)+CD01(n)}(u)$ denotes the cdf of the $CD01(n+1) + CD01(n)$, $E(x)$ represents the expected value of the gap ($CD01(n+1) + CD01(n)$), and $\alpha_{RL01}(x)$ is the distribution of the rear lag $RL01$ for this case.

The gap available to the merging vehicle is the sum of the two consecutive time headways $CD01(n)$ and $CD01(n-1)$, which shares the distribution ($f_{CD01}(x)$).

To determine the distribution of the rear lag for case 3, the distribution of the rear lag $RL01$ of the two sub-cases must be combined with the corresponding occurrence probabilities, as follows:

$$F_{RL01,3}(x) = (\eta P_x) \left(a \left(\int_0^x \rho_{RL01}(u) du \right) + (1-a) \left(\int_0^x \alpha_{RL01}(u) du \right) \right)$$

$$f_{RL01,3}(x) = \frac{(dF_{RL01,3}(x))}{dx}$$

where $F_{RL01,3}(x)$ denotes the cdf of the rear lag $RL01$ and $f_{RL01,3}(x)$ is the distribution of the rear lag $RL01$ for this case.

Case 4: Vehicles 1 and 2 are non-weave vehicles that changes lane.

Vehicles 1 and 2 are willing to make lane changes and are able to find an adequate lag for doing so. The probability of the driver of vehicles 1 and 2 performing a lane weave is assumed to be $(1-W_1)$, while the distribution of the rear lags $RL01$ is expressed as

$$f_{RL01,4}(x) = \frac{(1 - F_{CD01}(x))}{E(x)}$$

where $f_{RL01,4}(x)$ denotes the distribution of $RL01$ for case 4, $F_{CD01}(x)$ represents the cdf time headways of traffic in lane 1, and $E(x)$ is the expected value of time headway of the traffic in lane 1.

Final distribution of the rear lag $RL01$: The final distribution of the rear lag $RL01$ can be calculated by combining the distributions of the rear lag $RL01$ for all the above cases with their corresponding probabilities. The final distribution of the rear lag $RL01$ is as follows:

$$\begin{aligned} P(RL01 < x) &= W_1(1 - \Omega W_1) \int_0^x f_{RL01,1}(u) du + \Omega W_1^2 \int_0^x f_{RL01,2}(u) du \\ &\quad + (1 - W_1)(\Omega W_1) \int_0^x f_{RL01,3}(u) du + (1 - W_1)(1 - \Omega W_1) \int_0^x f_{RL01,4}(u) du \\ &= F_{RL01}(x) \\ f_{RL01}(x) &= \frac{dF_{RL01}(x)}{dx} \end{aligned} \tag{5.4}$$

where $f_{RL01}(x)$ denotes the final distribution of the available rear lag $RL01$. This distribution is used to determine the maximum probability of lane-changes from lane 1 to lane 0.

6.1.3.2 Determination of rear lags $RLI0$ for lane 0

Regarding Figure 6.3, the arrival process of vehicles in lane 1 is assumed to be independent of that of vehicles in lane 0 of the loop-ramp, but is influenced by the size of the rear lag $RL01$. In this case, four independent possible cases occur at the merge gore on this weaving section are specified as potentially occurring prior to the rear lag $RL01$ for lane 1.

The probability of vehicle 1 weaving is W_2 . Consequently, the probability of vehicle 1 not weaving is $(1-W_2)$. Vehicle 2 wishes to weave into lane 1 provided the following conditions are fulfilled: (1) vehicle 2 is a weaving vehicle with a probability of W_2 , and (2) vehicle 2 is a non-weaving vehicle that weaves lane with a probability of Φ . This corresponding probability that it wishes to weave into lane 1 is ΦW_2 , and vehicle 2 only has a probability of $(1-\Phi W_2)$ of weaves lanes into lane 1. After calculating the distributions of rear lag $RLI0$ for each of the four cases these distribution are combined with the corresponding probabilities to yield the final distribution used in the procedural step of the methodology to determine rear lag $RL01$ for lane 1.

Final distribution of the rear lag $RLI0$: The final distribution of the rear lag $RLI0$ can be calculated by combining the distributions of the rear lag $RLI0$ for all of the possible cases above, together with their corresponding probabilities. The final distribution of the rear lag $RLI0$ is as follows:

$$\begin{aligned}
P(RL10 < x) &= W_2(1 - \Phi W_2) \int_0^x f_{RL10,1}(u) du + \Phi W_2^2 \int_0^x f_{RL10,2}(u) du \\
&\quad + (1 - W_2) \Phi W_2 \int_0^x f_{RL10,3}(u) du + (1 - W_2)(1 - \Phi W_2) \int_0^x f_{RL10,4}(u) du \\
&= F_{RL10}(x) \\
f_{RL10}(x) &= \frac{dF_{RL10}(x)}{dx} \tag{6.5}
\end{aligned}$$

where $f_{RL10}(x)$ denotes the final distribution of the available rear lag $RL10$.

6.2 Maximum Lane Change Estimations

As mentioned at the start of this section, the maximum capacity depends on the maximum lane-changes that can be estimated with gap acceptance theory. This study uses the general gap acceptance equation provided in the revised traffic flow theory monograph (1992) to estimate the maximum number of lane-changes, which can be expressed as

$$Q_m = Q_p \int_0^\infty f(t) g(t) dt \tag{6.6}$$

6.2.1 Estimation of maximum number of lane-changes from loop-ramp lane 0 to C-D roadway lane 1

The maximum numbers of lane-changes from loop-ramp lane 0 to C-D Roadway lane 1 can be determined by plugging the rear lag $RL01$ in Equation (6.4) into Equation (6.6).

$$\begin{aligned}
Max.(V_{RC}) &= Q_p \int_0^\infty f(t) g(t) dt \\
&= V_{CR} \int_{t_0}^{\infty} \frac{x_{01} - t_{TT01}}{t_{av01}} f_{RL01}(x) dx \tag{6.7}
\end{aligned}$$

6.2.2 Estimation of maximum number of lane-changes from C-D roadway lane 1 to

loop-ramp lane 0

The maximum number of lane-changes from C-D roadway lane 1 to loop-ramp lane 0 can be determined by adding the rear lag $RL10$ in Equation (5.5) into Equation (5.6).

$$\begin{aligned} \text{Max.}(V_{CR}) &= Q_p \int_0^{\infty} f(t)g(t)dt \\ &= V_{RC} \int_{t_0}^{\infty} \frac{y_{10} - t_{TT10}}{t_{av10}} f_{RL10}(y)dy \end{aligned} \quad (6.8)$$

Once the maximum lane-changes become available, the linear programming is applied to calculate the capacity of two weaving lanes in the weaving section. The solution to Equation (6.2) is the required capacity of the weaving section (specifically two lanes, lanes 0 and 1). The total capacity of the weaving section is denoted as the capacity of these two weaving lanes.

6.3 Model Application and Sensitivity Analysis

The first part of this section applies the proposed method to a sample loop-ramp and C-D roadway weaves, and then estimates their capacity using the HCM (2000) methodology. The results of the two methods are compared and discussed. Sensitivity analysis is then performed to examine the influence of changes in the segment capacity value of the basic separated connecting collector-distributor roadway.

The application presented in this study describes the time headways of all traffic streams using exponential distribution, to facilitate calculation and presentation. This

assumption does not influence the methodological framework (Lertworawanich and Elefteriadou, 2003). Furthermore, for a loop-ramp weave to reach capacity, it is not necessary that all movements have high demand, and thus users must select an appropriate distribution for each movement.

6.3.1 Field data collection and analysis results

This study chose a site along the Taipei freeway systems for data collection to compare the capacity observations with estimates made using the proposed methodology. Data collection stations are located as shown in Figure 3.1 and Table 3.1 in Chapter 3 lists the general site conditions. Complete field data sets of traffic operation in weaving sections were collected during peak period (7:00 a.m. to 9:00 a.m.) over two days (May 20, and November 22, 2002). The posted speed limit on the separated connecting collector-distributor (C-D) roadway was 60 km/h (THCM, 2001), and the study site comprised both level and straight road segments. The video cameras used for the data collection were component of the ATMS system and were positioned at sufficient elevation to record the overall traffic situation and characteristics of the weaving section. The required operational data were derived directly from the videotape documentary. The data were broken down into 1-minute intervals.

Both traffic flows entering the weaving section and weaving vehicles were measured using the videotaped data. Densities were also obtained directly from the

videotapes by counting the number of vehicles in a given weaving section length. Specifically, the data was obtained by pausing the videotape using a television timer at 3 seconds intervals, recording the traffic densities for each lane, and averaging the reading to obtain a density value for each 1-minute period. Traffic speeds (space mean speed) were calculated and used to determine the time taken for a vehicle to travel a known distance. Lane changes within the weaving section were summed for each 1-minute period, and these values were then converted into lane-changes per hour per kilometer per lane. The station can continuously provide information regarding the speeds, flow rates, and densities for the oncoming traffic from both the C-D roadway and the loop-ramp for different event intervals.

6.3.2 Analytical results

Table 6.1 lists the basic traffic characteristics for the study site, including the estimation of critical time headway, average time headway, average speed, average density, average traffic flow rate, weaving rate, and non-weaving rate for lanes 0 and 1.

Table 6.1 Basic traffic characteristics analysis of the separated connecting collector-distributor roadways

Position	Items	Lane 0 (outside lane)	Lane 1 (inside lane)
	Critical time headway	1.55sec	1.31sec
	Average time headway	2.93 sec	4.82 sec
	Average speed	25.58 km/hr	23.40 km/hr
	Average density	59.13 veh/km	75.51 veh/km
	Average traffic flow rate	1854.56 veh/hr	1214.91 veh./hr
	Weaving rate (w)	0.4929	
	Non-weaving rate	0.5071	

6.3.3 Comparison with the Highway Capacity Manual

This study stresses that the weaving types in the HCM (2000) do not cover the separated connecting C-D road discussed here. In HCM, since weaving type A is the only type that resembles the separated connecting C-D road, it is adopted in the current comparison. Table 6.2 lists the capacities calculated using the proposed methodology and the method proposed by the HCM (2000), for the above site condition.

Given a weaving rate of 0.4929, the capacity estimated using the proposed methodology is 878 pcphpl, compared to 889 pcphpl using the HCM (type A), meaning a 1.25% difference the estimates using the two methods. Accordingly, it can be concluded that the HCM type-A approach overestimates separated connecting C-D roadway capacity.

Table 6.2 Summaries of the case study results

Case study	Capacity estimate (pcphpl)	
	Proposed method	HCM (2000)
Capacity (w=0.4929)	878	889
Capacity (w=0)	1750	2200

Noted: pcphpl=passenger car per hour per lane

6.3.4 Sensitivity analysis of loop-ramp weaves capacity for flow ratios w_1 and w_2

To gain more insight into the proposed capacity prediction method, sensitivity analysis of capacity versus flow ratios of traffic streams from lanes 1 and 0 is conducted to examine their influence on capacity. As discussed above, the capacity of the

loop-ramp weave can reach a maximum, depending on prevailing flow ratios.

Figure 6.1 shows the sample weave configuration, which is used for the sensitivity analysis. This study assumes a traffic speed for the C-D roadway of 60 km/h, and that the C-D roadway traffic is equally distributed between the two lanes. The speeds of weaving and non-weaving vehicles are calculated based on analysis of field data. For the sensitivity analysis the flow ratios of the traffic streams in lanes 1 and 0 vary from 0.0 to 1.0, with increments of 0.1.

Table 6.3 and Figure 6.4 list the results of the sensitivity analysis. The capacity of the loop-ramp weave varies from 700 to 3450 pc/h, equivalent to the capacities of basic C-D roadway segments. Although this finding corresponds with the HCM (2000), the capacity value is less than for the HCM, as listed in Table 6.4. This phenomenon occurs because on separated connecting collector-distributor roadway such as those vehicles must be using to undertake weaving or lane-change activities, do not allow for merging in the other lane. Furthermore, this phenomenon occurs because weaving vehicles from the lane of the loop-ramp or C-D roadway create additional gaps following the completing of weaving and lane changing. Nevertheless, too many weaving vehicles in the weaving lanes can create conflict and reduce capacity.

Table 6.3 Sensitivity analyses of capacity with various weaving ratios

Capacity	w2=0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
w1=0.0	3450	3250	3100	2900	2750	2650	2550	2500	2450	2400	2350
0.1	3250	3050	2900	2700	2550	2450	2350	2300	2250	2200	2150
0.2	3100	2900	2750	2550	2400	2300	2200	2150	2100	2050	2000
0.3	2950	2750	2600	2400	2250	2150	2050	2000	1950	1900	1850
0.4	2700	2500	2350	2150	2000	1900	1800	1750	1700	1650	1600
0.5	2550	2350	2200	2000	1850	1750	1650	1600	1550	1500	1450
0.6	2350	2150	2000	1800	1650	1550	1450	1400	1350	1300	1250
0.7	2200	2000	1850	1650	1500	1400	1300	1250	1200	1150	1100
0.8	2050	1850	1700	1500	1350	1250	1150	1100	1050	1000	950
0.9	1900	1700	1550	1350	1200	1100	1000	950	900	850	800
1.0	1800	1600	1450	1250	1100	1000	900	850	800	750	700

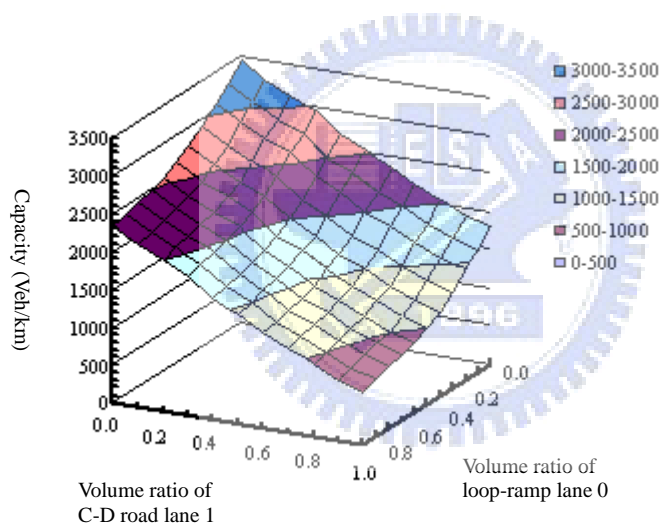


Figure 6.4 Capacity estimates for the weaving section examined of this study

Table 6.4 Comparisons of the capacity results between HCM (2000) and proposed method

Weaving ratio Capacity	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
HCM type A	2200	1770	1460	1210	1030	880	760	660	580	515	450
Proposed method	1750	1520	1370	1180	1000	850	720	615	525	430	350
Difference capacity	450	250	90	30	30	30	40	45	55	85	100
Percent (%)	25.71	16.45	6.60	2.54	3.00	3.53	5.56	7.32	10.48	19.77	28.57

6.4 Summaries and Discussions

This study demonstrates that the weaving flow ratio for each lane of the separated connecting collector-distributor roadway of the weaves with the loop-ramp entrance flow that flow is to impact the capacity. Ramp weave capacity reduces with increasing weaving flow. This phenomenon occurs because of the characteristics of the weaving traffic flow on the separated connecting collector-distributor roadway where vehicles weave or change lanes. Sensitivity analysis demonstrates that the capacity in the loop-ramp weave model is a reducing function of the weaving rates on the separated connecting collector-distributor roadway and loop-ramp lanes. The study results further demonstrate that using the HCM type-A capacity as the separated connecting C-D roadway\ capacity results in overestimation, reaching 25.71% in the subject example. However, the rate of increase in the capacity of the loop-ramp weaves or C-D roadway weaves is less than for the type A weaving section of freeway segment capacity (Lertworawanich and Elefteriadou, 2003), owing to the larger presence of weaving vehicles in the weaving section. Based on the proposed methodological approach, the proposed procedure is suitable for application to other C-D roadway types with time headway distributions, which can be obtained, provided field data are available. In the future gathering additional data to validate the proposed model for various separated connecting collector-distributor roadway-weaving segments could perform a further

study.



CHAPTER 7

A Control Model for Freeway Ramp Metering

Traffic congestion causes great economic loss all over the world. Therefore, control methods should be introduced to improve efficiency and reduce congestion in subpart traffic networks. Ramp metering aims to regulate the amount of traffic entering a freeway from a specific entry ramp. This enables the freeway to operate at a desired level of service. This control strategy is effective if it is able to ensure that the traffic volume downstream from the ramp does not create a freeway bottleneck (where the traffic demand exceeds capacity).

This study proposes a demand-capacity ramp-metering model with a discrete-time traffic relationship (flow-density-speed). The objective of the model is to minimize the traffic queues on entry ramps while keeping the flow on the freeway mainline at certain level of service. The metering control model is formulated as a mathematical program that minimizes the difference between actual and desired density. A solution framework is proposed for applying the control system (Cho and Tsai, 2009).

The next section begins with a configuration of a separated connecting collector-distributor road that is utilized throughout this study. The third section describes the ramp-metering control model, traffic flow relationships, and signal timing. Two numerical examples with real data are discussed in section four. Conclusions are

made in section five.

7.1 Separated Connecting Collector-Distributor Roadways

A typical separated connecting collector-distributor (C-D) roadway is shown as Figure 7.1. Notations that describe the traffic flow rates are indicated on the figure to give a clear explanation.

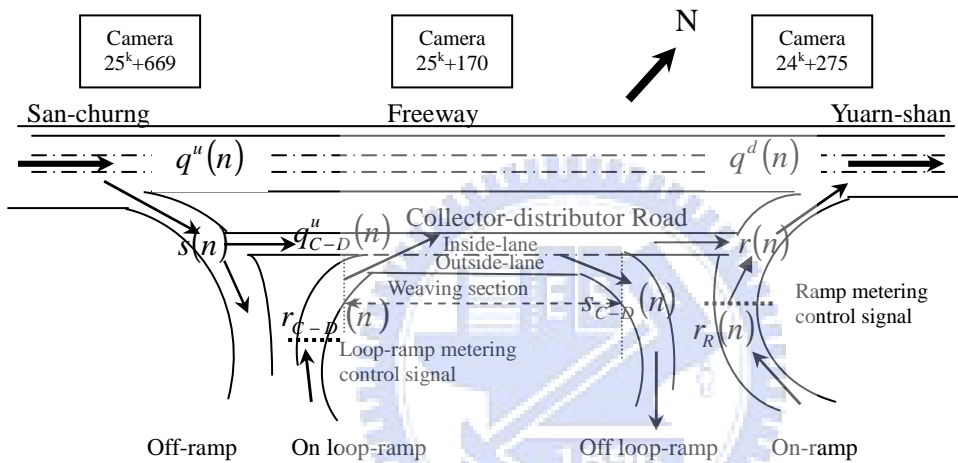


Figure 7.1 The configuration of separated connecting collector-distributor roadway.

7.2 Ramp Metering Control Model

Ramp metering aims to regulate the amount of traffic entering freeway from a specific entry ramp; ensuring the traffic volume downstream of the ramp does not exceed capacity. This section begins by describing traffic flow relationships on each road segment in section 7.2.1, and the ramp control objectives in section 7.2.2. The ramp-metering rate is determined in section 7.2.3. The determination of ramp control signal timing is discussed in section 7.2.4, and the solution framework is illustrated in

section 7.2.5.

7.2.1 Traffic flow relationships

To develop a ramp-metering model, it is necessary to describe the traffic flow relationships with mathematical models. In this study, we assume traffic flow relationships for each road segment conform to widely accepted norms: $flow = density \times speed$. This study utilizes a traffic relationship shown in (Greenshields *et al.*, 1935).

Consider a discrete-time representation of the traffic relationships on a multi-lane freeway section with a single entry and exit ramp. It is assumed that at time slice n , upstream traffic flows into a certain section at a rate of $q^u(n)$ per lane from upstream boundary, and entry-ramp traffic flows into the same section at a rate of $r(n)$ per lane. Moreover, traffic flow in the road section discharges at the downstream boundary at a flow rate of $q^d(n)$ per lane, and exiting traffic diverts to the exit ramp at a flow rate of $s(n)$ per lane. Therefore, according to the law of conservation, the number of vehicles on the freeway section at time $n+1$, denoted as $Q(n+1)$, can be represented as:

$$Q(n+1) = Q(n) + \Delta t (\lambda_N q^u(n) - \lambda_N q^d(n) + r(n) - s(n)) \quad (7.1)$$

Derived from traffic relationship, define $k(n) = \frac{Q(n)}{\lambda_N \Delta x^F}$. Equation (7.1) can now be written as:

$$k(n+1) = k(n) + \frac{\Delta t}{\Delta x^F} \left(q^u(n) - q^d(n) + \frac{r(n)}{\lambda_N} - \frac{s(n)}{\lambda_N} \right). \quad (7.2)$$

We assume in this relationship between traffic flow and density is $q = f(k)$, which is generally referred to as the fundamental diagram of traffic flow. If we substitute $q = f(k)$ into equation (7.2), and define $\alpha = \frac{\Delta t}{\Delta x^F}$, we obtain:

$$k(n+1) = k(n) - \alpha \times f(k(n)) + \alpha \times q''(n) + \frac{\alpha}{\lambda_N} (r(n) - s(n)). \quad (7.3)$$

Applying control system terminology to the traffic flow relationships, traffic density $k(n)$ is the state variable, ramp metering rate $r(n)$ is a control variable, and freeway inflow $q''(n)$ is the disturbance input.

It is assumed that the traffic flow relationships on separated connecting C-D roadway are similar to those of freeway mainline. Therefore, define $\alpha_{C-D} = \frac{\Delta t}{\Delta x^{C-D}}$ and the traffic flow relationships can be represented as:

$$Q_{C-D}(n+1) = Q_{C-D}(n) + \Delta t (\lambda_N^{C-D} q_{C-D}''(n) - \lambda_N^{C-D} q_{C-D}^d(n) + r_{C-D}(n) - s_{C-D}(n)) \quad (7.4)$$

$$k_{C-D}(n+1) = k_{C-D}(n) + \frac{\Delta t}{\Delta x^{C-D}} \left(q_{C-D}''(n) - q_{C-D}^d(n) + \frac{r_{C-D}(n)}{\lambda_N^{C-D}} - \frac{s_{C-D}(n)}{\lambda_N^{C-D}} \right) \quad (7.5)$$

and

$$k_{C-D}(n+1) = k_{C-D}(n) - \alpha_{C-D} \times f_{C-D}(k_{C-D}(n)) + \alpha_{C-D} \times q_{C-D}''(n) + \frac{\alpha_{C-D}}{\lambda_N^{C-D}} (r_{C-D}(n) - s_{C-D}(n)). \quad (7.6)$$

7.2.2 Ramp control objectives

The objective of ramp control is to keep the level of service on the controlled freeway segment at a desired level. From the fundamental traffic diagram, high traffic density implies low traffic speed. Moreover, when high traffic demand surges into a

specific section, the density will increase, causing congestion. Therefore, vehicles approaching the section should be controlled to ensure that freeway segment operates at its optimal condition (at capacity). In order to do this, density should be kept within a small range around the critical density (the density when traffic flow is at capacity). Hence, the objective of ramp metering control is to have traffic density $k(n)$ follow a target of $k_d \in [k_c - \varepsilon, k_c + \varepsilon]$, where k_c is the critical density and ε is a user-defined parameter (Zhang and Ritchie, 1997). ε is chosen to be one standard deviation of the samples in this study.

If the inflow of a section is larger than its outflow, $r(n) > \lambda_N f(k(n))$, traffic density will increase until it becomes completely blocked. Applying the following ramp metering law then the congestion can be prevented (Zhang and Ritchie, 1997):

$$r(n) = \frac{\lambda_N}{\alpha} \times (k_d - k(n) + \alpha \times f(k(n))). \quad (7.7)$$

If inflows are known on both freeway mainline and ramp, the following metering law can keep the density close to target density,

$$r(n) = \frac{\lambda_N}{\alpha} \times (k_d - k(n)) + \alpha \times (f(k(n)) - q^u(n)). \quad (7.8)$$

As the objective of ramp control in this study is to maintain traffic density close to critical density it is equivalent to minimize the difference between the observed and critical density for both freeway mainline and C-D roadway. The ramp control objective

is formulated as following nonlinear optimization problem,

$$\text{Min } \frac{1}{2} \sum_n (k_d - k(n))^2 \quad (7.9)$$

Subject to

Equation (7.3), (7.6), (7.8)

7.2.3 Determining the ramp metering rate

The Demand control is based on the real-time calculation of comparing upstream demand and downstream capacity. The upstream demand flow is measured in real-time and compared with downstream capacity, which is determined by downstream flows and gap measurements. The difference between upstream demand and downstream capacity is the allowable entrance flow rate. Theoretically, if the upstream demand is greater than the downstream capacity, a zero metering rate (or ramp closure) is used to prevent congestion on freeway mainline. However, this is rare in the real world. Practically, if the upstream demand is larger than the downstream capacity, a minimum metering rate (which is predetermined) is selected.

Therefore, after solving the densities on both the freeway mainline and the C-D roadway, the ramp-metering rate should be determined to satisfy those densities. This study determines the ramp-metering rate by minimizing total queue length on both entry ramp and loop-ramp. The problem is formulated as follows:

$$\text{Min} \sum_{n=1}^N \left((d_R(n) - r_R(n))^2 + (d_L(n) - r_{C-D}(n))^2 \right) \quad (7.10)$$

subject to

$$r_R(n) = \begin{cases} c_R, & \text{if } \theta(n) \times r(n) \geq c_R \\ \theta(n) \times r(n), & \text{if } \theta(n) \times r(n) < c_R \end{cases}, \quad n = 1, 2, \dots, N \quad (7.11)$$

$$r_{C-D}(n) = \begin{cases} c_L, & \text{if } (1 - \theta(n)) \times r(n) \geq c_L \\ (1 - \theta(n)) \times r(n), & \text{if } (1 - \theta(n)) \times r(n) < c_L \end{cases}, \quad n = 1, 2, \dots, N \quad (7.12)$$

$$\theta(n) = \frac{r_R(n)}{r_R(n) + r_{C-D}(n)}, \quad n = 1, 2, \dots, N \quad (7.13)$$

$$d_R(n+1) = \begin{cases} 0, & \text{if } r_R(n) \geq d_R(n) \\ d_R(n) + q_R(n+1) - r_R(n), & \text{if } r_R(n) < d_R(n) \end{cases}, \quad n = 1, 2, \dots, N-1 \quad (7.14)$$

$$d_L(n+1) = \begin{cases} 0, & \text{if } r_{C-D}(n) \geq d_L(n) \\ d_L(n) + q_L(n+1) - r_{C-D}(n), & \text{if } r_{C-D}(n) < d_L(n) \end{cases}, \quad n = 1, 2, \dots, N-1 \quad (7.15)$$

$$d_R(1) = q_R(1) \quad (7.16)$$

$$d_L(1) = q_L(1) \quad (7.17)$$

7.2.4 Determining the control-signal timing

The above models are capable to obtaining optimal ramp metering rates, but they should be implemented by a timing signal that actually controls the traffic flow. However, actual implementation of ramp-metering rates should be accomplished by signal timing. This study utilizes the model develop in (Webster and Cobbe, 1966) to calculate loop-ramp signal timing, including green time and cycle length:

$$C_L(n) = \frac{1.5 \times L_{lost} + 5}{1 - \frac{r_{C-D}(n)}{c_{C-D} - q_{C-D}^u(n)}} \quad (7.18)$$

and

$$g_L(n) = \frac{r_{C-D}(n)}{c_{C-D} - q_{C-D}^u(n)} \times C_L(n) \quad (7.19)$$

The cycle length and green time for ramp are also calculated by the same method, with the following equations,

$$C_R(n) = \frac{1.5 \times L_{lost} + 5}{1 - \frac{r_R(n)}{c_{Freeway} - q^u(n) - r_{C-D}(n)}} \quad (7.20)$$

and

$$g_R(n) = \frac{r_R(n)}{c_{Freeway} - q^u(n) - r_{C-D}(n)} \times C_R(n). \quad (7.21)$$

If the capacity were less than traffic demand, the above equation would not be suitable for calculating cycle length and green time. In that case, the metering rate is set to be available capacity. If the ramp entrance demand is less than or equal to available freeway mainline capacity, the demand will be satisfied; else, the entrance rate will be controlled in order to make sure the capacity won't be exceeded.

7.2.5 Solution framework

This section illustrates the solution framework of combining ramp-metering control, queue-length minimizing, ramp metering rates and signal timing. The solution

framework is demonstrated in Figure 7.2 (The system diagram of ramp metering control). It begins with i) deriving traffic demands from surveillance system, ii) calculating the available capacities of the road segments, iii) determining the optimal ramp metering rates, iv) minimizing queue length with respect to demands from ramp and loop-ramp, and v) determining signal timings.

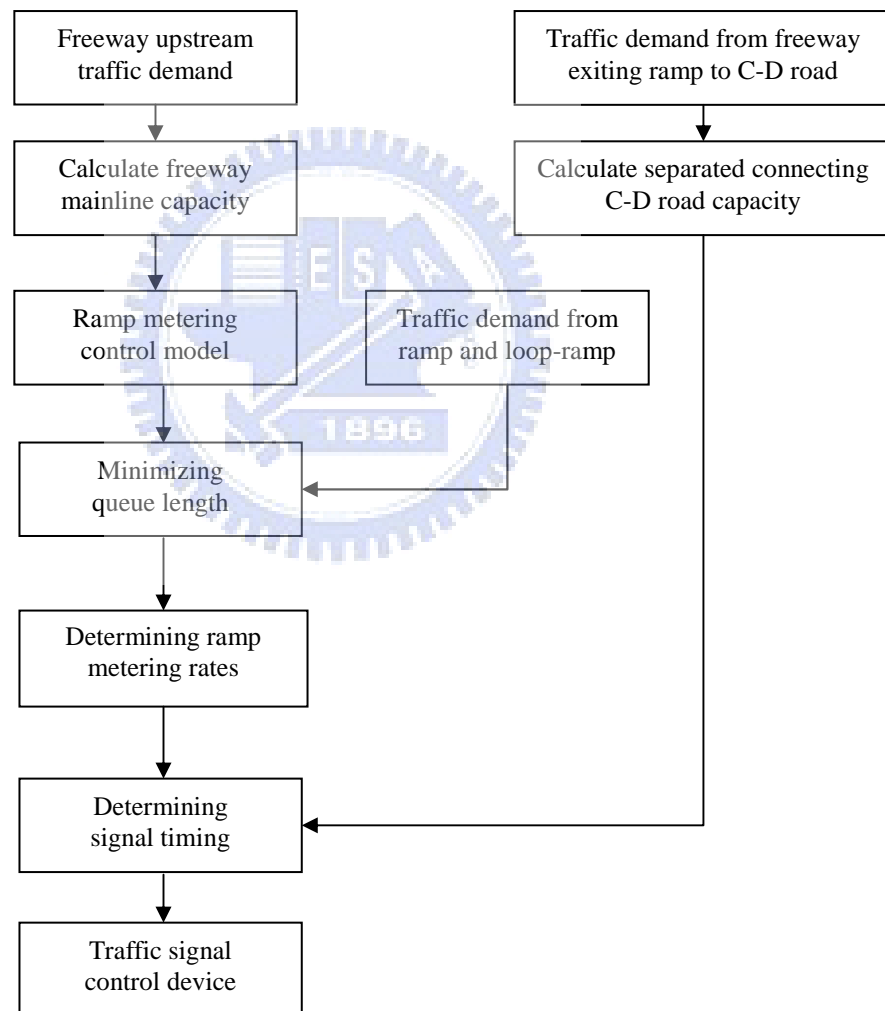


Figure 7.2 The solution framework of ramp metering control problem.

7.3 Numerical Examples

Two numerical examples are discussed in this section: (i) the Taipei interchange and (ii) the Shihdin interchange. The first example demonstrates the effectiveness of the proposed model by evaluating present pre-timed ramp metering control and proportion distribution method. The second example discusses the performance of proposed model by comparing it with ALINEA (Papagoergiou, *et al.*, 1991, 1998; Smaragdis, *et al.*, 2003; Kosnalopoulos, *et al.*, web.).

7.3.1 Taipei interchange

An actual freeway interchange is tested in this section. The configuration of the Taipei interchange is illustrated in Figure 7.1. Field data of this test include traffic flow rates, speeds, densities, vehicle classes, lane changing activity, and weaving section geometry. These data were collected through surveillance video cameras during the morning rush period (07:00 to 09:00) on November 22, 2002. During data collection, the ramp metering controls were disabled to make sure the traffic conditions represented real demand. The collected data are summarized in Table 3.1, in Chapter 3.

During data collection, there were a total of 4,988 vehicles: 1,174 vehicles in the inside lane and 3,814 vehicles in the outside lane. The average traffic speed was 24.47 km/hr (23.40km/hr in the inside lane and 25.58 km/hr in the outside lance). Average traffic density was 67.32 vehicle/km; 59.13 vehicle/km in the inside lane and 75.51 vehicle/km in the outside lane. The lane changing activity rate was 50.71%, and the

weaving activity rate was 49.29%.

The parameters (on freeway main line and C-D roadway) for this numerical example are described in Table 7.1. The free-flow speed on ramp v_{ramp} and loop-ramp $v_{loop-ramp}$ are set to be 40 kilometers per hour; the capacities are set to 1,800 vehicles per hour per lane. The minimum ramp-metering rate r_{min} is 180 vehicles per hour (Lumentritt *et al*, 1981). The minimum value of green time is 2 second, and the maximum value of green time is 15 seconds (Payne, 1971).

Table 7.1 Parameters of the ramp metering control model on the Taipei interchange

Parameter	Freeway	C-D road
Free-speed (km per hour)	100	60
Jam density (vehicles per km per lane)	102.08	110.88
Critical density (vehicles per km per lane)	54.85	65.58
Target density (vehicle per km per lane)	45.40	59.65
Capacity (vehicle per hour per lane)	2350	1750
Number of lanes	3	2

The results of the proposed model, the proportion method, and the fixed signal timing are plotted in black lines, grey lines, and dash lines, respectively. Figure 7.3.a and 7.3.b illustrate the calculated cycle lengths on the ramp and the loop-ramp, respectively. Figure 7.3.c and 7.3.d demonstrate the computed green times on the ramp and the loop-ramp, respectively. The results of cycle length and green time are demonstrated in Figure 7.3. It is clear to see in Figure 7.3 that the cycle length and green time are quite different from those of the fixed signal timing method and proportion method. Figure 7.4.a demonstrates the queue length on the ramp, Figure 7.4.b illustrates the queue

length on the loop-ramp, and Figure 7.4.c shows the total queue length (number of vehicles) on both the ramp and the loop-ramp. Applying the calculated signal timing to the ramp metering afforded a total queue volume of 19,131 vehicles. The queue lengths of the fixed-cycle time and proportion-distribution methods were 24,103 and 21,629, respectively. The total queue volumes were reduced 4,972 vehicles derived from the proposed model (more than 20%). The total queue length results are shown in Figure 7.4.

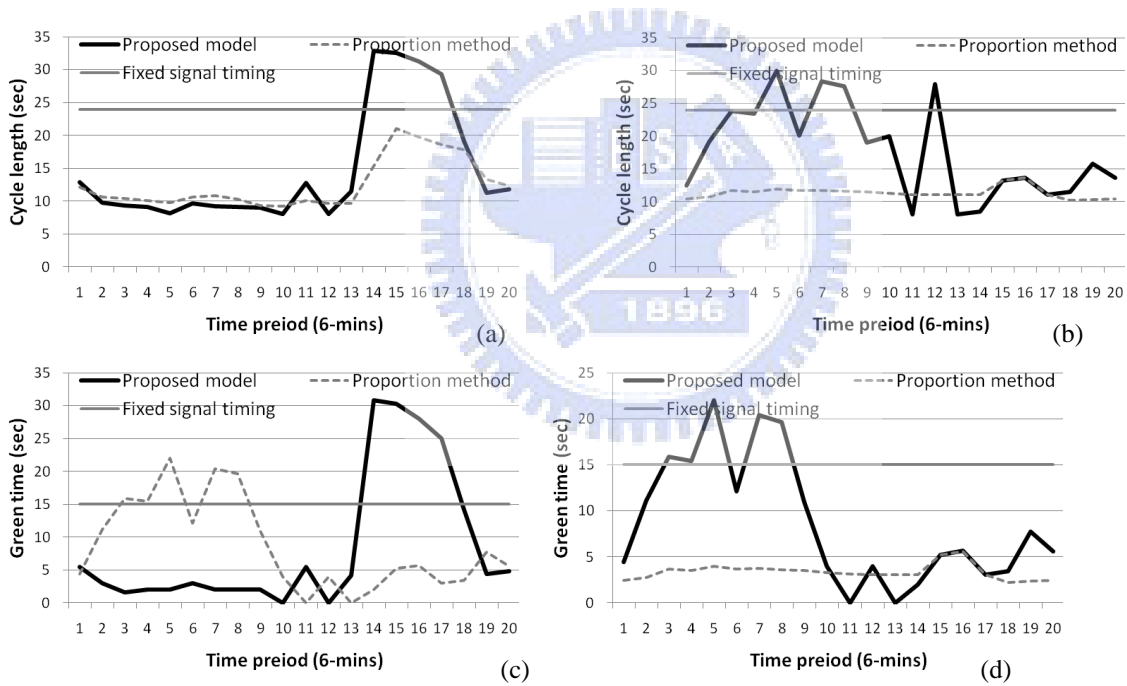
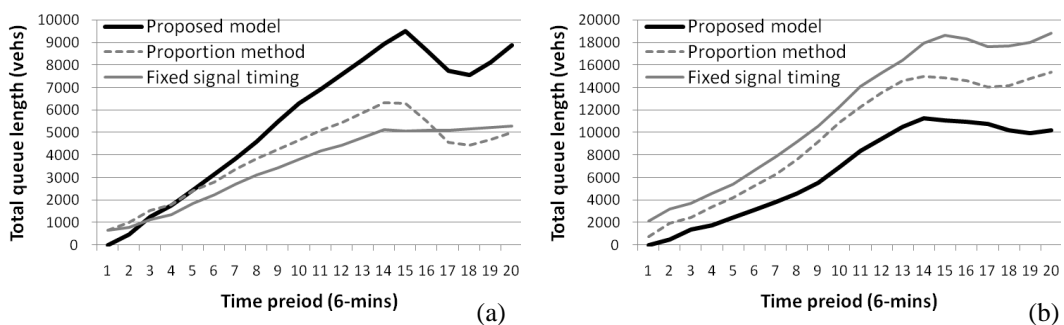


Figure 7.3 The comparison of cycle lengths and green times among the proposed model, the proportion method, and the fixed signal timing method



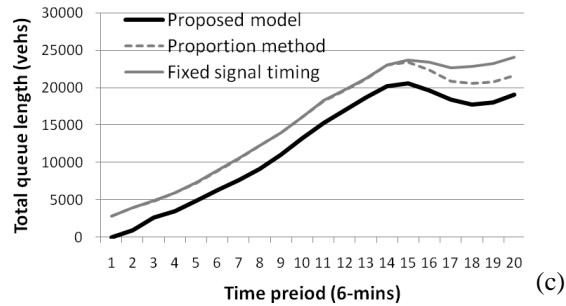


Figure 7.4 The comparison of total queue length among the proposed model, the proportion method, and the fixed signal timing method

7.3.2 Shihding interchange

This section compares the performance of the proposed model to ALIENA on a freeway ramp. The configuration of test site is illustrated in Figure 7.5 and Table 7.2.s.

The freeway mainline at the test site has two lanes; the entry ramp has one lane. Field data of this test include traffic flow rates, speeds, densities, vehicle classes, and occupancy rates. These data were collected using the surveillance system during the morning rush period (9:00AM to 11:00AM) on August 12, 2006. The collected data were based on fixed-timing ramp metering control: 26 seconds of cycle length, 10 seconds of green time, 3 seconds of yellow time, and 13 seconds of red time.

Table 7.2 Data on the Shihding interchange configuration

Road section	Length (m)	Number and width (m) of Lanes				Curve (m)	Grade (%)	Affected by Signal		Super--elevation (%)
		Main-Stream	Entry		Entry			Exit		
Main-line	500	2	3.50	-	-	R=0	0	No	No	2
On-Ramp	350	-	-	1	4.5	R=800	+6.0	Yes	-	6

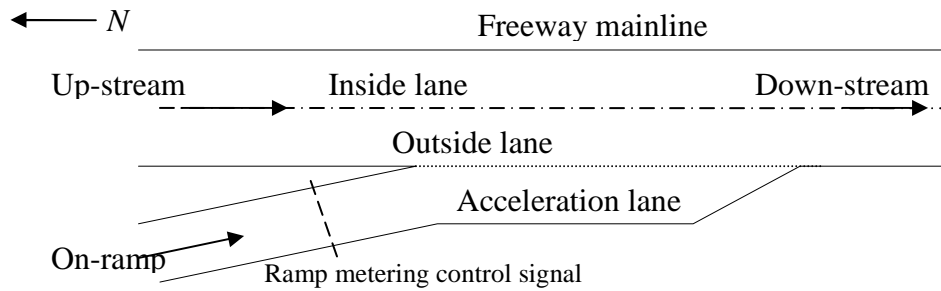


Figure 7.5 The Shihding interchange test site.

During the data collection, there was an average of 1,428 vehicles per hour using the freeway mainline: 708 vehicles in the inside lane and 720 vehicles in the outside lane. The average traffic speed was 59.95 kilometers per hour (61.06 kilometers per hour in the inside lane and 59.41 kilometers per hour in the outside lane). The average traffic density was 33.72 vehicles per kilometer (16.67 vehicles per kilometer in the inside lane and 17.05 vehicles per kilometer in the outside lane). The parameters utilized in the ramp-metering example are listed in Table 7.3.

A comparison among the proposed ramp-metering model, the fixed-timing metering control method, and ALINEA is demonstrated in Figure 7.6. The results of the proposed model, fixed signal timing method, and ALINEA are plotted in black lines, grey lines, and dash lines, respectively. Figures 7.6.a, 7.6.b, 7.6.c demonstrate the flow rates, densities, and speeds on freeway, respectively. The average traffic characteristics of these control methods, including flow rate, density, and speed, are listed in Table 7.4. When compared to the fixed-timing signal method, the proposed ramp-metering model was able to improve the flow rate by 12.5%; however, that of the ALINEA remains

nearly the same.

Table 7.3 Parameters of the ramp metering control model on the Shihding interchange

Parameter	Freeway	Ramp
Free-speed (km per hour)	70	40
Jam density (vehicles per km per lane)	152.05	
Critical density (vehicles per km per lane)	71.34	
Target density (vehicle per km per lane)	56.62	
Capacity (vehicle per hour per lane)	2200	1800
Critical occupancy (%)	38.00	
Number of lanes	2	1

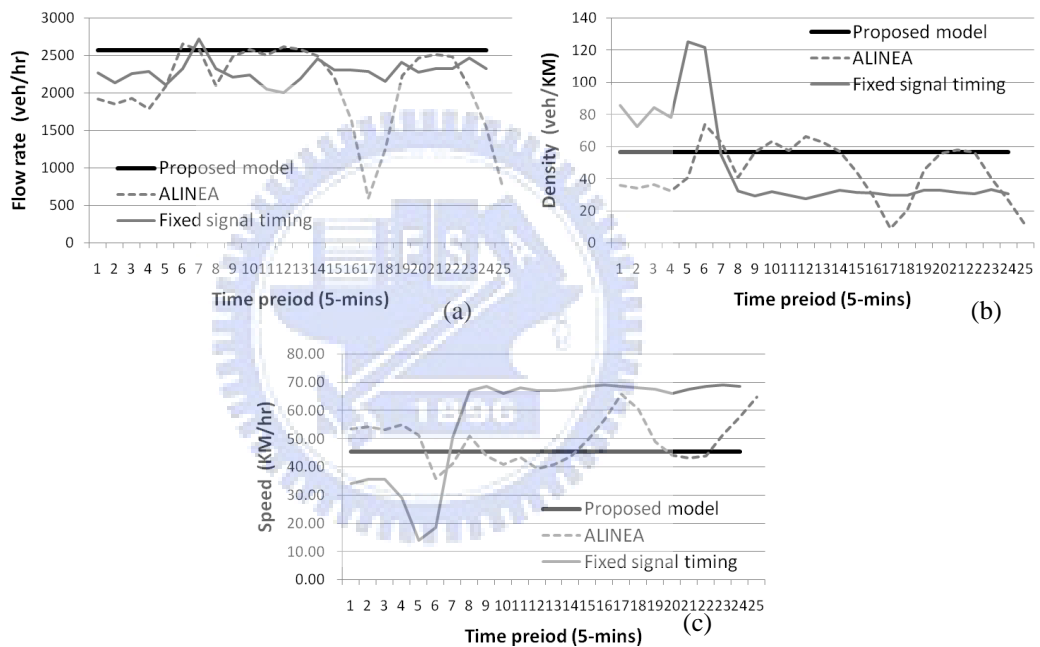


Figure 7.6 Shows the control results of the proposed model, the fixed signal timing method, and ALINEA

Table 7.4 Traffic characteristics under different control methods

Traffic Characteristics	Units	Proposed model	Fixed timing method	ALINEA
Flows	Vehicle per hour	2,570	2,284	2,280
Densities	Vehicle per hour per lane	56.62	47.86	51.35
Speed	KM/hr	45.39	57.04	46.36

7.4 Conclusions

This research presents an open-loop control model to address a ramp-metering problem. It is formulated as a mathematical programming model that optimizes traffic conditions on freeway system. A queue minimization model that enhances metering rates on both loop-ramps and ramps of separated connecting C-D roadway has been presented. Two numerical examples with real network data demonstrate the effectiveness of the proposed model. Preliminary results show that the proposed model has the potential to solve ramp-metering problems.



CHAPTER 8

Conclusions and Perspectives

8.1 Conclusions

In this chapter, we reach some conclusions, perspectives and contributions of this research. The complete content is to present in the final edition of this dissertation.

The general objective of this research is to study the traffic flow characteristics of weaving vehicles and optimal ramp metering control in a freeway and separated connecting collector-distributor (C-D) roadway weaving sections. Efforts are focused on analyzing the interrelation of traffic flow characteristics, as traffic speeds, traffic densities, and traffic flow rates.

In this research, we can conclude three parts as following.

First, the traffic flow characteristics of weaving traffic in a separated connecting C-D roadway on the freeway weaving section and on congested traffic flow conditions is studied by analyzing the three basic factors as traffic density, traffic speed and traffic flow rate characteristic of weaving vehicles. On the basis of the statistical analysis results the following major conclusions were drawn:

1. The general trends in the traffic flow characteristics of weaving vehicles appear to be similar to the lane changing and weaving activities of traffic in basic freeway sections. The general trends include traffic density, traffic speed

and traffic flow rate. However, specific values obtained for weaving traffic, such as the critical density being 55.74 vehicles per kilometer per lane, critical speed being 34.29 kilometers per hour and maximum flow rate (capacity) being 1854 vehicles per hour per lane, are different from the values for the traffic in other types of freeway sections.

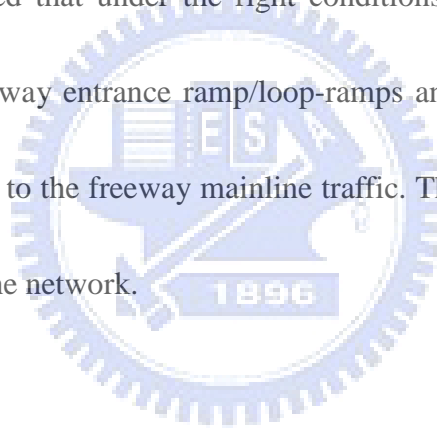
2. Traffic density and traffic speed appear to be better parameters for describing the behavior of weaving traffic than traffic flow rate. The results of regression analysis appear that the constant coefficients of regression models coefficient of constant indicate no significant with 95% confidence difference is observed between the traffic flow rate versus the traffic density and traffic speed. But the coefficients of first (x) and secondary (x^2) variables appear higher significant with 95% confidence.
3. Generally, the R^2 values for these regression models are very high, that ranges from 0.733409 to 0.949083. It implies that a considerable portion of the correlation of traffic flow characteristics as traffic density, traffic speed and flow rate are excessively influenced by those factors considered in the regression variables.

Second, the developed capacity estimation model shows that the ratio of weaving flow on each lane of the separated connecting C-D roadway of the loop-ramp weaves

affects its capacity. As weaving flow percentage increases, the capacity of the ramp weaves is to decrease. This happens because the characteristics of the weaving traffic flow on the separated connecting C-D roadway that these vehicles must be to take weaves or lane change activities. The sensitivity analysis shows that in the model the capacity of the loop-ramp weaves is a decreasing function of the weaving rates on the separated connecting C-D roadway and loop-ramp lanes. However, the rate of increase on capacity of the loop-ramp weaves or C-D roadway weaves is less than that of the basic freeway segment capacity, due to the presence of weaving vehicles. Based on the theoretical concept of the methodology, the procedure is enough to be applied to other weaving types of time headway distributions, which can be obtained whenever field data are available. Then the results of this study are realistic than the HCM (2000) method.

Third, the ramp metering has been an effective tool for combating freeway congestion. One particular approach is based on linear equation and optimal solution theory—demand-capacity metering simulation equation model. However, this approach is based on the linear equations of a nonlinear traffic flow model, and its performance is limited when traffic conditions deviate far from the state that the controller was designed to maintain. While an efficient control model allows for a high performance operation, an inefficient one does not. Owing to available techniques, a dynamic

operation is required. The primary objective of this research has been to present some general properties underlying the effectiveness of ramp metering along a congested freeway and separated connecting C-D roadway, taking queuing, traffic entrance traffic dynamics into account. It is shown that optimal ramp/loop-ramp control policies depend both on traffic entrance propensities and on differential between freeway and C-D roadway weaving traffic conditions. The development simulation equation model may be considered a highly efficient local ramp metering strategy according to the reported field results. The model showed that under the right conditions, with a freeway and separated connecting C-D roadway entrance ramp/loop-ramps and heavy traffic; ramp metering can be very beneficial to the freeway mainline traffic. This model can actually promote more efficient use of the network.



8.2 Perspectives

The developed methodology built a new paradigm of dynamic weaving section capacity estimation and optimal ramp metering control problems. However, there are still some promising issues that should be further examined to enrich this research field.

They are outlined in the following.

1. Encapsulating dynamic weaving section capacity estimation and optimal ramp metering control will make the proposed framework more complete. For example, weaving section capacity estimation model could be replaced with a

previous state estimation model; variant types of weaving section could also be considered in future analysis.

2. The parameter calibration for the sensitivity of dynamic weaving section capacity estimation model will make the proposed models more powerful in practical applications. It is also a valuable issue in the research field of roadway weaving sections capacity analysis.
3. To formulate some problems of operational and planning applications as a dynamic weaving section capacity estimation and optimal ramp metering control design problem in this style, will help traffic operator to understand the perturbations caused by a new management alternative. It provides a different viewpoint from conventional dynamic weaving section capacity estimation and optimal ramp metering control approaches.
4. With the further treatments mentioned above, a good potential could be expected to deploy a dynamic weaving section capacity estimation and optimal ramp metering control simulator based on the proposed methodology.



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Vita



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Publications:

A. Referred Journal Papers.

1. Cho, Hsun-Jung and George C. P. Tsai (2005), “The Behavior Analysis on a New Type Weaving Section,” *Journal of the Eastern Asia Society for Transportation Studies*, Vol. 6, pp1570~1081, 2005.
2. Cho, Hsun-Jung and George C.P. Tsai (2006), “Ramp Metering Control on the Junction of Freeway and Separated Connecting Collector-Distributor Roadway,” *Journal of the WSEAS TRANSPORTATIONS on MATHEMATICS Issue 1, Vol. 5, ISSN 1109-2769*, pp. 111~116.
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4. Cho, Hsun-Jung and George C.P. Tsai (2009), “Capacity Analysis for a New Weaving Section Type of Freeway-Separated Connecting Collector-Distributor Roadway,” *Submitted to the Journal of Transportmetrica*.

B. Referred Conference Papers.

1. Cho, Hsun-Jung and George C. P. Tsai (2005), “The Behavior Analysis on a New Type Weaving Section,” *Presentation paper in The 6th International Conference of Eastern Asia Society for Transportation Studies 2005*, 2005.09.21~24.

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