□ 論　文 □

고속도로 연결로 접속부에서의 속도 추정 모형

Speed Prediction Models for Freeway Merging Area

申 治 玹
(韓國建設技術研究院 先任研究員)

목　차

ABSTRACT

가속차선이 교통류의 운영상태와 안전에 기여하는 바는 벌써부터 인식되어 왔으나 이 변속차선이 유입형 연결로 접속부 전체의 운영에 미치는 영향을 수치화하거나 체계적으로 평가하기 위해 현장 자료를 바탕으로한 실험적 연구는 진행되어 오지 못하였다. 현재 널리 참고되고 있는 1985년 USHCM의 접속부 운영상태 분석 방법론은 단지 차선 1의 교통량을 예측하는 데 주안점을 두고 있는데 가속차선의 길고 짧음에 따라 접속부 바로 전 차선 1의 교통량 분포가 크게 변화한다는 사실(많은 현장 관측을 통해 확인)은 고려하지 못하고 있다. 이는 접속부의 운영 상태가 같은 교통량 조건하에서도 크게 차이가 난다는 것을 뜻하며 가속차선의 존재를 무시한채 운영과 관련한 MOE를 도출하는 것이 서비스수준 산정 방법으로 충분한 것인가 하는 의문을 자연히 낳게 한다.

본 논문은 가속차선이 고속도로 연결로 접속부의 운영에 미치는 영향을 주로 다루고 있다. 가속차선의

독립적인 역할과 영향을 체계적으로 관찰하기 위해 미국내 여러 지역에서 8개의 고속도로 연결로 접속부를 선택하고 각 지점에 접속부의 상하류 지역을 포함하는 2,000ft 구간내에 다섯대의 카메라를 설치, 지점별로 약 3시간 동안 자료를 수집하였다. 총 193개 자료수의 분석을 통해서 다중 회귀 모형을 구성하는 독립변수로 가속차선의 길이를 사용하는 것이 타당하다고 결론지었으며, 접속부 운영의 질, 특히 속도를 추정하기 위한 모형을 수립하였다. 본 연구를 통해 얻어진 관점과 방법론은 1994 USHCM 고속도로 연결로 분석 방법론 설정에 일부분 반영되고 있으며, 특히 교통운영과 흐름의 방식에서 유사한 엇갈림 구간의 분석 방법과 일관성 있는 분석 체계 마련을 위해서 서비스수준 산정 절차 정립에 엇갈림 알고리듬을 활용하는 방안을 제시하였다.

1. INTRODUCTION

A ramp may be described as a length of roadway providing an exclusive connection between two roadway facilities. As the USHCM depicts, a ramp may consist of up to three geometric elements of interest: the ramp−freeway junction, the ramp proper, and the ramp−street junction.

Vehicles from a ramp terminal try to move into a freeway stream. To get into the stream, drivers need to decide whether gaps − void spaces created by vehicles in the closest lane to the ramp (lane 1) − are appropriate for them to accept in terms of size and speed. Based upon their evaluations, drivers accept gaps or reject them.

In this process, it is desired to lessen characteristic differences, primarily in speed, between two traffic demands to allow safe and fast merging maneuvers. When undue acceleration by entering ramp traffic takes place directly on the traveled highway, it disrupts the flow of through traffic and often is hazardous. To minimize these undesirable aspects of operation at the junction, acceleration lanes are put into place.

An acceleration lane is a length of roadway in which heavy interactions between freeway drivers and ramp vehicles originates. From this area where two traffic demands compete for space, the resulting turbulence of interaction propagates to the freeway mainline. Therefore, an acceleration lane should have sufficient length to enable a driver to make the necessary change between the speed of his approach and the speed of the highway traffic in a safe and comfortable manner.

To date, however, no analysis method exists which takes into account the impact of acceleration lanes on operational quality of ramp−freeway junctions although the length and type of acceleration lane produces some operational changes in the junction area. Furthermore, there is no firm guideline, based on empirical studies, which helps simulation studies, geometric design, and road safety.

Some mathematical models for calculating merging delay experienced by ramp vehicles existed, but many of the assumptions made in these heavy probability models were basically unrealistic. Several empirical studies were conducted in the late 60's as well. They were only concerned with spatial use of acceleration lane and by no means focused on its impact on operational aspects in the junction area.

It is therefore strongly suggested that well−established empirical studies be implemented that makes the impact of the acceleration lane on the operation of the junctions clear and compensates for the deficiencies in the previous studies. By doing so, more complete models to describe the operational state of the junctions can be constructed.

This study mainly deals with the effect of acceleration lanes on entrance ramp operation and investigates its effect on the operational quality in the junction area. Its primary concern resides in manifesting the role of acceleration lanes in operation of merge junction area. Particularly, an attempt to utilize the length of acceleration lane for various predictive models was consistently made.

The pricipal objective of this research is to identify responsive indicators of operational changes, reflective of the length of acceleration lanes, which lead to the development of level of service models. Observation of speed−related measures describing the state of operation, was made within an area of approximately 2,250 ft long. A typical section being observed covers from a point 750 ft upstream of the physical gore to 1,500 ft downstream from the gore.

2. BACKGROUND AND LITERATURE REVIEW

2.1 Mathematical Models for Merging Delay

A few mathematical formulations of merging delay which incorporate the length of acceleration lanes are originated from a series of many "gap−acceptance" models[5]−−[9]. Although many calculation methods of delays experienced by vehicles in a minor street during a gap−acceptance process had been revised and improved by several researchers[10]−−[13], the existence of acceleration lanes in the merge areas was still ignored in the delay calculations.

A gap−acceptance process from acceleration lanes was first treated by Haight and Bisbee[12] in the late 1960's. A complete mathematical model was not suggested, but many problems associated with a model formulation were pointed out. Weiss and Maradudin[13] also pioneered in this field and formulated many mathematical models for various gap−acceptance processes.

Mine and Mimura[14], extended the Weiss and Maradudin's study, were able to derive the probability density function (PDF) of delay to merging vehicles but assumed acceleration−lane length to be infinite. Weiss and Blumenfeld[15] later revised the previous model[13] and formulated the merging process from an acceleration lane with a finite length. It is assumed that vehicles on the acceleration lane travel at a constant speed v while the traffic on the main road travels at a constant speed V greater than v. Explicit distinction between merging delays while moving and stopped delays at the end of acceleration lane was mathematically treated in this model.

Michaels and Fazio[18] supported the idea that the angular velocity of on-coming traffic is a critical factor in merging vehicles' decision process while travelling on the acceleration lane and were able to estimate the length of acceleration lane necessary for the ramp driver to find an acceptable gap in 85 percent of the time.

Drew, et. al.[10] investigated the effect of many geometric factors on the gap-acceptance process; the effect of acceleration-lane length on the speed of ramp vehicles were studied; speed profiles of ramp vehicles at the nose were developed. Percentage of acceptance of certain gap sizes by ramp vehicles who travel on the acceleration lanes with different lengths was derived in the form of a regression formula.

Mathematical models are usually found to be far from real-world phenomena which have been observed through empirical studies. Their concern is focused only on prediction of merging delays to minor street vehicles. This fact actually disqualifies the mathematical formulations as level of service models because they simply ignore overall operational state of the merging area. Some critical assumptions made in these models are not realistic enough to describe the operational phenomena usually faced.

2.2 Some Empirical Analyses

A study by Fukutome and Moskowitz [16] is of interest due to its analytical approach that traces paths on acceleration lanes taken by merging vehicles. Their findings are such that regardless of ramp design features, most ramp vehicles took similar paths under various freeway flow conditions; Somewhat more length of the acceleration lane was used at low volumes than at high volumes; Merging distance required at high turning speed is as great as that required at low speed; Natural path of nearly all vehicles is within a 50:1 taper, and this design provides sufficient acceleration distance for all turning speeds of ramp vehicles.

Polus and Livneh[17] first suggested that drivers consist of two groups: drivers who perform the merging maneuver during the first half of the acceleration lane and drivers who merge during the second part of the acceleration lane, regardless of whether an appropriate gap or headways was available to them previously. They observed that a number of ramp vehicles (almost 50 percent) use nearly the full portion of an acceleration lane even under very light freeway traffic conditions.

The outcome of the above two studies make an interesting point. It implies that the merging process from acceleration lanes seldom operates to minimize merging times, which is exactly contrary to a conventional assumption made in mathematical models. Unfortunately however, all the empirical studies have not related the effect of the acceleration lanes to overall operation of the merge area by limiting their focuses only to the behavior of ramp vehicles on the acceleration lanes.

2.3 Treatment of Acceleration Lanes in Current Technical Manuals

The 1985 HCM[2] maintains 13 regression formulas in its chapter 5 for analyses of various features of ramp–freeway junctions. Each regression equation predicts volumes in freeway outerlane using freeway volume, ramp volume, and geometric and volume conditions in adjacent ramps. The relevant level of service is decided by calculating a checkpoint volume which is the lane 1 volume plus the ramp volume.

However, the core part in the computational procedure of ramp–freeway analysis, the prediction of lane 1 volume, does not reflect the use of acceleration lanes. The lane 1 volume is computed regardless of what the length of acceleration lane is and if the acceleration lane exists. The effect of acceleration lanes on vehicular distribution among freeway lanes has not been discussed, nor has any possible reason been stated.

The chapter 5 of 1994 USHCM which adopted some key ideas from this study utilizes the length of acceleration lane as one of important independent variable in the models for density/speed prediction. The format of the models, however, seems to be little obscure, and general fitting results are considered unsatisfactory. Moreover density was derived from flow–speed relationship and never measured directly.

"A Policy on Geometric Design of Highways and Streets", widely known as the AASHTO Green book[21], discusses findings accumulated over time and recommends design standards in selecting the length of acceleration lanes. The minimum required length of acceleration lanes is found from a look–up table provided using highway design speed, curve design speed of ramp, and desired speed of ramp vehicles at the end of acceleration lanes.

Some findings referred in the AASHTO Green book are not consistent with today's practice, for example, it is very hard to find taper–type acceleration lanes in heavily congested freeways. In general, the results from these studies fall short as performance measures of merge junctions or as design guidelines because they limited their attention only to ramp vehicles travelling on acceleration lanes.

3. FIELD DATA COLLECTION

A study scope is limited to the cases where single–lane on–ramps merge with six–lane freeway facilities. Only parallel–type acceleration lanes were studied because numerous site visitations had revealed that most agencies responsible for operating and maintaining freeway facilities prefer the parallel-type to the taper-type which was rarely found.

This study examines the effect of the length of the acceleration lanes in particular and focuses on the operation of single lane ramps in 6-lane freeway facilities. A total of eight ramp-freeway junctions with the length of acceleration lanes ranging from 325 ft to 1,650 ft were observed for analysis of lane distribution and prediction of speed measures. Table 3-1 summarizes the characteristics of each junction studied.

To achieve the objectives stated in above, many operational aspects of the junctions pertaining to different lengths of acceleration lanes were carefully investigated. Macroscopic observations of many merge areas formed a data base for analyses.

A 5-minute analysis period was chosen because of its steadiness and stability in terms of variation in count and speed. The longer analysis period, such as a 15-minute period, has often turned out to be inadequate to use because, within that time period, several dissimilar operations have been frequently observed. The 15-minute periods often contain dramatic changes in speeds and relatively big fluctuations in counts. They need to be stratified into the shorter periods so as to differentiate the sporadic dissimilarities, because averages over the 15-minute periods usually distort or dilute the transition undergone.

Data were collected mostly when freeway volume peaks up.

Table 3-1. Description of Sites Collected

Site No.	# of Period	Upstream Ramp		Downstream Ramp		Volumes		Length of Acceleration
		Type	Distance	Type	Distance	Freeway	Ramp	
1	30	off	600ft	off	5280ft	3420-5390	520-860	950ft
3	44	off	4000ft	off	3200ft	1760-4120	1670-2400	325ft
8	24	off	400ft	off	4500ft	4730-5790	110-350	450ft
12	23	off	500ft	off	7900ft	3820-5190	820-1180	1100ft
18	18	off	1500ft	off	7920ft	2080-4300	430-620	1000ft
20	18	off	1100ft	off	2900ft	4370-6690	450-900	1200ft
21	18	off	1350ft	off	1800ft	5700-7360	260-660	1250ft
23	18	off	2500ft	off	10000ft	3570-4990	870-2040	1650ft

Most on-ramp junctions had close upstream off-ramps and very far downstream off-ramps mainly because they were part of diamond interchanges, and spacings be tween these interchanges were usually more than 3,000 ft.

3.1 Data Collection Methodology

Data collection methodology for this research relies on multiple ground-mounted videotaping equipment. A typical site consists of setting up the 8mm video recorder/camera system at five locations, uniformly spaced at 375 feet. The cameras were mounted on 30-ft copper poles placed approximately 15 feet from the edge of the road. This effectively yields a study range of 1,500 feet. At sites where conditions were such that this spacing did not cover the entire length of the acceleration lane, the spacings were altered to accommodate the study section. Since it will be important to maintain uniform camera locations, the study area can be effectively increased by eliminating one intermediate location in favor of another 375 ft further downstream. Thus, common reference points were still maintained. Figure 3-1 illustrates a typical site setup where 375-ft spacings adequately cover the study section.

Each site was videotaped for a period of between 2-3 hours. At each trap location where cameras are set up, traffic-related information, such as volume counts, vehicle classification, and average speeds, for both freeway and acceleration lane are obtained.

Counts and vehicle classification at upstream and downstream ramps from the study ramp of interest were done with the use of magnetic sensing traffic counters. The units are programmed with the aid of a portable PC to record selected information. The time at which the unit is to start and end the gathering of data and the interval for summaries is programmed into the unit. Vehicle classification and traffic counts are downloaded to a portable PC, and the data is summarized by 5-minute periods as programmed.

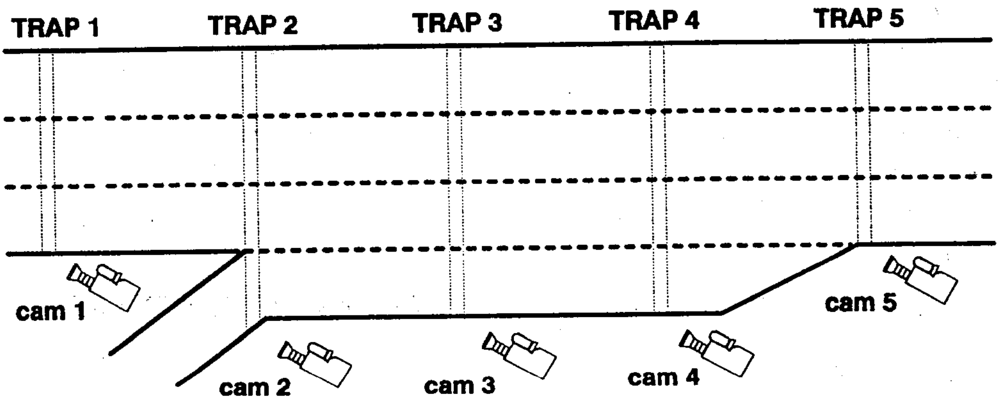


Figure 3-1. Typical Deployment Layout of Video Cameras

3.2 Data Reduction Process

The majority of videotapes acquired from the study were reduced by manual technique. With the help of an on—line recording program connected to a personal computer, vehicle counts, vehicle classification and speeds at each trap location were reduced. Each reduced output file contains traffic counts, speeds, and time—lapse records.

The video tapes were also reduced with the aid of a computer program which digitizes images of unobstructed roadway sections. By discerning the differences between digitized picture frames from a videotape, the program is capable of processing the detected differences into counts, speeds and vehicle classification for each lane based on pre—defined trap locations.

The data went through a cleaning process which eliminated some obvious human errors made in the data reduction process and was summarized by 5—minute intervals showing for each trap the number of vehicles, speeds, and corresponding standard deviations of speeds.

The data obtained from the reduction process and portable traffic counters is transferred to a spreadsheet program for simple analysis first and subsequently imported into a statistical analysis software package for creating scatterplots and performing correlation analyses, linear or non—linear regression analyses, and some advanced statistical analyses.

From the data set available, the first task was to isolate variables which could feasibly be used as either independent or dependent variables in the modeling process. These variables fall into three general categories:

- volumes/flow rates
- geometric variables
- performance variables

Volume data included all of the familiar variables used in current models, and three new ones. Unlike the 1985 HCM and its predecessors, all volume data is in terms of equivalent hourly flow rates for a 5—minute study period, and all flow rates are converted to passenger—cars unit reflecting heavy vehicle presence and lane width.

Many geometric variables were measured and are available for the study. Such variables as distances to upstream/downstream ramps in the 1985 HCM, were maintained. The length of acceleration lanes is precisely defined, and components of them such as length of tapered portion and length of parallel portion were measured respectively.

Some new performance variables, by definition, were defined and none of them are explicitly considered in current methodologies. Performance variables mainly focused on speed measures.

All the variables are described in Appendix.

4. MODEL DEVELOPMENT

193 data points from the eight sites, which include 160 stable and 33 unstable data points were chosen and used in model-

ing process.

The attempt to develop level of service models which incorporate the length of acceleration lanes was consitently made. And some newly defined speed measures appeared to be promising.

4.1 Prediction of Performance Variables

The practice in a 1985 USHCM and a KHCM in evaluating the performance of a ramp–freeway junction is based upon the calculation of merged volume. One major procedure of this is the estimation of lane 1 volume, which is then added to ramp volume to get the merged volume. The 1965 USHCM well states the reason: ramps are the important input–output elements of the freeway, the emphasis is on estimating volumes which will result in ramp gore area.

Using the merged volume as a performance measure, however, often create shortcomings. For instance, the study site 3 carries extremely high ramp volume, and the lane 1 volume was very low due to the combined impact of the ramp volume and a short acceleration lane. Resulting merge volumes ranged from 1,000 pcphpl to 2,800 pcphpl with no significant operational deterioration. According to the manuals stated above, the level of service(LOS) at this junction is F. Therefore, threshold values of the merged volumes for level of service criteria should be updated, or alternative measures for the level of service should be sought.

The principal task in creating LOS methodology is the identification of measures of effectiveness which can be used to adequately describe the quality of service provided to drivers in merge areas. Since speed–related measures are undoubtedly the historical choice as an MOE, many attempts to predict these measures, incorporating the length of acceleration lane, were made hereafter.

It was intended to expose the role of acceleration lanes rather than to predict such performance measures. From the seven sites which have all the required information, 175 data points were analyzed and calibrated.

4.2 Predictive Model of S_R Utilizing the Weaving Algorithm

Chapter 4 of the 1985 HCM contains an algorithm for the estimation of speed of weaving and non–weaving vehicles in a freeway weaving section as:

S_W or S_{NW}

$$= 15 + 50/[1+a(1+VR)^b(V/N)^c/L^d]\}$$

where:

SW (S_{NW}) = average speed of (non) weaving vehicles, mph;

VR = volume ratio of weaving volume to total volume;

V = total flow rate in weaving section, pcph;

N = no. of lanes in weaving section;

L = length of weaving section, ft;

a,b,c,d = constants of calibration.

In this equation, estimated speeds are

bounded by 15 mph at the low end and (15 +50) = 65 mph at the high end. Taking this into account, the algorithm can be shown in a simplified form as:

S_{NW} or S_W

$= 15 + [(S_{MAX} - 15) / (1 + W)]$

where:

W = weaving intensity factor

$= a(1+VR)^b(V/N)^c/L^d$

S_{MAX} = max. average expected speed of vehicles, mph.

A similar analogy was tried in the predictive model for speed related measures in merge areas. Instead of predicting two separate speeds, one characteristic speed was used to depict merge operations.

The application involves the following assumptions; (1) in the merge area, it is the acceleration lane plus lanes 1 and 2 of the freeway which are influenced in terms of operating speed; (2) additional freeway lanes, it is assumed, will not be seriously impacted by right-hand merge operations; later this was indirectly witnessed in a weaving section under the Hannam bridge of Korea; (3) while the length of a weaving section is identifiable, the influence length of a merge area is not; as shown in Figure 4-1, 1,500 ft was chosen as this was a length which appeared to approximate the area of maximum impact when site videos were reviewed by reading the license plate numbers of ramp vehicles to see how equally they distributed themselves among three lanes at that distance; (4) the design speed or free flow speed of the freeway should be used as an estimate of S_{MAX} in the algorithm.

The weaving intensity factor, W, is replaced by M, a merge intensity factor. This factor may or may not have the same algorithmic form as the weaving intensity factor. Several forms for this factor were applied and compared. Since the design speed of a study site, noted as S_{DSGN}, is known, and the free flow speed, noted as S_{FF}, can be reasonably determined or observed, either of these two speeds can replace S_{MAX}. In addition, the values of S_R, defined in the Appendix, can be reduced from the data. Therefore, the actual value of the merging intensity factor, M, could be computed from field data in two ways as:

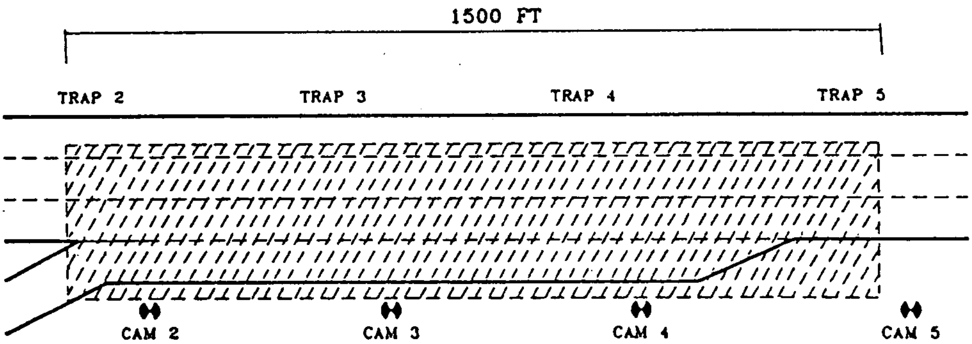


Figure 4-1. Influence Zone of Merge Area

$$M = [(S_{FF} - 15)/(S_R - 15)] - 1$$
$$M = [(S_{DSGN} - 15)/(S_R - 15)] - 1$$

and can then be correlated to other traffic/geometric variables which might adequately describe the merging intensity.

The next step in the application of the weaving algorithm to merge cases would be to calibrate a predictive relationship for S_R. If the basic format of the weaving algorithm is retained, the following equation might be tried:

$$S_R = 15 + [(S_{DSGN} \text{ or } S_{FF} - 15)/(1 + M)]$$
$$M = a\ (1+MR)^b\ (V_1+V_2+V_R)^c\ /\ L_A{}^d$$

Since the total flow rate in the ramp influence zone of the merge area is equal to $V_1+V_2+V_r$ (V_{R12}), then it should be possible to estimate V_1+V_2, which will be hereafter referred to as V_{12}, using similar equations as in the prediction of lane 1 volume. Two conclusions have been reached from an NCHRP study; (1) V_{12} is predicted with greater accuracy than V_1, and (2) inclusion of upstream and downstream impacts continues to improve the quality of fit and accuracy of predictions.

The merge ratio term, 1+MR, replaces the volume ratio term, 1+VR, of the weaving algorithm. For this merge ratio term, three different ratio terms were developed for the calibration. MR1 is a microscopic merge ratio, MR3 is a ramp volume to VR12 ratio, and MR4 is a ratio designed to increase sensitivity to ramp volumes. The first two, when added to value of 1, is bounded with a minimum of 1 and a maximum of 2, and the last by itself has a minimum of 1 and a maximum of very high number as long as total freeway flow is higher than ramp flow. Three obvious forms tried for this ratio term, 1+MR, were:

$$1+MR1 = 1+ V_R/V_M$$
$$1+MR3 = 1+V_R/V_{R12}$$
$$MR4 = (V_F+V_R)/(V_F-V_R)$$

N, the number of freeway lanes which divided the total weaving flow in the weaving algorithm, was omitted in the format of the merge intensity factor as this study deals with only six-lane freeways. The length of the weaving section was replaced by the length of the acceleration lane or microscopic components of the acceleration lanes defined in Section 3.4.2.

The "field" values of "M," derived from the measured values of S_R and S_{DSGN} or S_R and S_{FF} can be regressed against the other variables in the equation format for M. Two speeds for S_{MAX} and three merge ratios for 1+MR term required six different non-linear regression analyses, and each analysis underwent an average of 50 adjusting iterations to find the appropriate coefficients. Table 4-1 and 4-2 show the results produced for various non-linear regressions against the merge intensity factor, M;

For stable points only,

$$M = a\ (1+MR)^{b}\ (V_{R12})^{c}\ /\ L_{AP}^{d} \qquad \text{Eq. (1)}$$

Table 4-1. Model Fitting Results for M

S_{MAX}	1+MR	a	b	c	d	R^2
S_{FF}	1+MR1	.0000416	1.18561	1.25142	0.23722	0.81
	1+MR3	.0000428	1.64574	1.20265	0.16670	0.83
	MR4	.0000255	0.19118	1.38740	0.27668	0.80
S_{DSGN}	1+MR1	.0001753	1.00366	1.09074	0.19785	0.81
	1+MR3	.0001969	1.36636	1.04405	0.14483	0.83
	MR4	.0001949	0.16321	1.12941	0.22412	0.80
	t-value	12 - 13	9.0-10.1	14-15	5.4-7.1	

For unstable points only,

$$M = a\ (1+MR)^{b}\ (V_{R12})^{c}\ /\ L_{AP}^{d} \qquad \text{Eq.(2)}$$

Table 4-2. Model Fitting Results for M

S_{MAX}	1+MR	a	b	c	d	R^2
S_{FF}	1+MR1	478565	-8.6864	-6.1923	-6.2721	0.46
	1+MR3	79001-E3	-3.9787	-4.2412	-2.8073	0.29
	MR4	29315-E5	1.9658	-2.2166	0.4522	0.29
S_{DSGN}	1+MR1	353218	-8.3779	-5.9897	-6.0721	0.46
	1+MR3	49200-E3	-3.8112	-4.1017	-2.720.	0.29
	MR4	16241-E5	1.9202	-2.1419	0.4345	0.30
	t-value	0.8-1.1	0.3-2.6	1.3-3.4	0.2-2.9	

All coefficients calibrated for stable flow condition appeared to agree with logic and were significant; the primary constant, a, was significant having a t-value ranging from 12 to 13, and the volume-related predictor variables such as (1+MR) and VR12 were significant as well having t-values ranging from 9 to 15. The geometric variable, the length of acceleration lane, was significant, too. As written in the equation above, L_{AP} was the best predictor among several component measures of the acceleration lane. The t-value for the parameter of L_{AP} ranged from 5.4 to 7.1 suggesting that a long acceleration lane diminishes the merge intensity under stable condition. Among all the six cases calibrated, the case where the design speed of freeway and

MR3 were used for the maximum operating speed and the merge ratio term resulted in the best fit.

It is obvious that the formats of the merge intensity do not work well for the unstable flow condition. The volume–related variables showed typical relationships between the volume measures and the speed measures found in speed–flow curves. But, the sign on the coefficients for the length of parallel portion of the acceleration lane was very interesting. The role of the acceleration lane seemed to turn into the opposite direction implying that a long acceleration lane increases the merging intensity under unstable operation.

For a comparative purpose, the same speed was calibrated using the logic parallel to the algorithm defined for the merge intensity. Instead of predicting the merge intensity, three predictor variables, the ramp merge volume, the merge ratio, and the length of the acceleration lane were directly regressed against that speed. Multiple linear –regression analyses with a slightly different merge ratio from the ones used in the weaving algorithm formats produced a relatively high quality of fit. All the coefficients met the logic. The role of acceleration lanes was of interest, particularly for unstable operating condition, such that the longer the acceleration lane is, the lower the operating speed becomes under unstable condition.

For stable condition,

$$S_R = 62.5 - 0.003782V_{R12} - 0.55617(V_R/V_1) + 0.005125L_{AP} \quad \text{Eq. (3)}$$

variables	constant	V_{R12}	V_R/V_1	L_{AP}
std. error	1.18	0.0002	0.0569	0.001
t–value	53.0	13.3	9.8	5.0

$R^2 = 0.783$ SE = 2.2

For unstable condition,

$$S_R = 13.3 + 0.01269\,V_{R12} + 20.762(V_R/V_1) - 0.05925L_{AP} \quad \text{Eq. (4)}$$

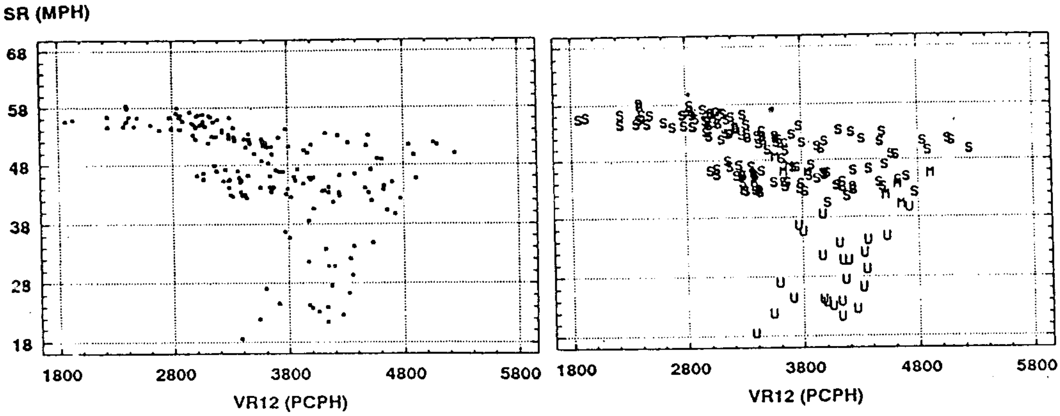
variables	constant	V_{R12}	V_R/V_1	L_{AP}
std. error	19.4	0.0039	7.515	0.021
t–value	0.7	3.2	2.8	2.9

$R^2 = 0.352$ SE = 4.7

Figure 4–2 shows the relationship between S_R and V_{R12}. In the stable flow range, the speed seems to linearly decline from about 60 mph to about 40 mph as VR12 goes from 2,000 pcph to 4,800 pcph. This trend is rarely found in the observation of commonly defined speed which usually operate with almost no speed decline for a wide range of flow level. This does, therefore, provide a range of speeds which might be useful in establishing level of service.

4.3 Predictive Models for Other Speed Measures(S_I and S_D)

Initially, this research focused on two measures of average speed which appeared to have some promise. These two measures were S_M and S_{FM}; S_M is the minimum average speed in a given lane for a given 375-ft trap, and S_{FM} is the minimum average speed across all freeway lanes in a given trap.



a. Typical Trend Shown b. Stable, Unstable Shown

Figure 4-2. Relationship between S_R and V_{R12}

This was a natural relationship to investigate, as it relates the microscopically-defined merge flow rate with the speed in the lane-trap most affected by it. These two measures, however, were proven to be insensitive to traffic variables and the length of the acceleration lane.

Hence, two additional speed measures were examined in trying to incorporate the length of acceleration lane as one of the determinant variables. S_I is the average speed in lane 1 within a ramp influence area, and S_D is the average speed-difference between lane 1 and the rest of the lanes within the ramp influence area. These two speed measures, when used together, may better describe the quality of operation in merge areas, and matrix tabulation of these two would be tried as inputs for LOS analysis.

Prediction of these two variables is based on the premise that lane 1 volumes is predicted with high accuracy. In fact, this premise was satisfied since outcomes from another part of this study on the lane distri bution models had demonstrated excellent relationships between the observed values and the predicted values of V_1. Use of V_1 as an known variable to the predictive models actually clarifies the role of the acceleration lane in a more precise way because it eliminates some possible statistical conflicts. To sort out the geometric impact on speed measures, the impact on the lane 1 volume due to the adjacent ramp conditions has to be known in advance since the lane 1 volume is one of the major determinants for the speed measures. The impact of lane 1 volume on the speed should not override that of the acceleration lane; the positive relationship between the lane 1 volume and the length of acceleration lane often falsify respective impact on the speed measures unless V_1 is used. Interaction between V_1 and V_R should be carefully managed so as not to misinterpret the role of acceleration lane in the speed measures. Multiple linear-regression analysis was done for two cases.

For stable condition,

$$S_1 = 57.5 - .00593V_1 - .00588V_R + .00841L_{AP} \qquad \text{Eq. (5)}$$

variables	constant	V_1	V_R	L_{AP}
std. error	0.89	0.0005	0.0004	0.001
t-value	64.1	11.0	16.2	7.7

$R^2 = 0.78$ SE = 2.2

$$S_D = 6.6 - .000826V_F + .00238V_1 + .00301V_R - .00768L_{AP} \qquad \text{Eq. (6)}$$

variables	constant	V_F	V_1	V_R	L_{AP}
std. error	0.96	0.0002	0.0008	0.0003	0.008
t-value	6.8	3.4	3.1	12.0	9.2

$R^2 = 0.848$ SE = 1.5

For unstable condition,

$$S_1 = 60.0 - .008534V_1 - .004952V_R - .013203L_{AP} \qquad \text{Eq. (7)}$$

variables	constant	V_1	V_R	L_{AP}
std. error	33.3	0.0068	0.0208	0.0393
t-value	1.8	1.2	0.2	0.3

$R^2 = 0.11$ SE = 9.5

$$S_D = 10.0 - .00172V_F + .00332V_I - .00574V_R + .00082L_{AP} \quad \text{Eq. (8)}$$

variables	constant	V_F	V_I	V_R	L_{AP}
std. error	3.5	0.0009	0.0017	0.0046	0.009
t−value	2.8	1.9	1.9	1.2	0.1

$R^2 = 0.417$ SE = 2.0

For the stable flow condition, all coefficients followed the expected logic. V_I and V_R demonstrated typical relationships with the speed measures; the volume variables reduced the operating speed in the lane 1 and enlarged the speed difference as they increased. L_{AP} contributed to enhance the operating speed in lane 1 and helped reduce the speed difference between lane 1 and the others, which is supposed the role of acceleration lane.

For the unstable condition, however, overall reliability of coefficients decreases drastically. But the sign on length of acceleration lane is still of interest suggesting reversal of the supposed function of acceleration lane. The most probable reason for the poor quality of fit could be attributed to the congestion that concurrently followed at downstream ramps.

In general, the models for the stable condition appear to be excellent in predicting the defined speed measures. Statistically, however, the models for the unstable condition have almost no meanings and were intended only to see what kind of role L_{AP} plays.

4.4 Validation of Models

All of the developed models must be evaluated to determine their ability to represent actual conditions and to explain the variability present in a sample other than the one used for their calibration. This section represents that final step in the development procedure and examines the applicability of the developed models.

To ensure a justifiable validation task, the input data for this task should lie in the region for which the developed models were calibrated. Thus, a sample of 37 stable data points from site 5, site 25 and site 28 which is not used in the model development, was tested.

It is noteworthy that the sites used were rather unusual because (1) they carried relatively high percentages of heavy trucks and (2) the geometric conditions of the roadways of the three facilities were non−ideal when compared to the conditions of the sites used in the calibration. Therefore, a high accuracy of the models in their prediction capabilities was not expected.

All the predicted values from the developed models for performance variables were compared to the actual measures from the three sites above. In general, most of the predicted values turned out to be accurate demonstrating less than 5% of relative dif ference between the predicted and the actual. However, the models for SD showed relatively poor prediction capabilities. This is probably because all three sites carried high (5 to 10) percentages of heavy trucks, and vertical/horizontal alignment conditions – for instance 3% vertical grade in site 5 and a roadway curved to the left in site 25 – were not generally ideal unlike those of the sites used for the model calibration. Table 4–3 shows comparison of the predicted to the actual measures.

5. CONCLUSIONS

This study has presented some predictive models pertaining to the quality of operation in freeway–ramp junctions. Major effort has been made to incorporate one of the geometric elements of a merge junction, the length of acceleration lanes, into analytical procedures. A set of regression models for both stable condition and unstable condition was developed to reflect or differentiate the effect of acceleration lanes.

Overall, the impact of acceleration lanes on lane distribution of freeway traffic, merging time and speed–related measures was highly significant. Therefore, it is strongly suggested for any prediction model for freeway–ramp junctions to employ the length of acceleration lanes as a major geometric variable.

Some predictive models of speed–related measures in the merge areas were successfully formulated especially for stable operation. Without a doubt, long acceleration lanes helped smooth the operation under stable operation. Predictive models for speed measures such as S_R, S_I and S_D were excellent in a statistical sense showing R–square value greater than 0.80 most of the time. The adaptation of the weaving algorithm to ramp areas was also considered a successful approach for further analysis and calibration.

Each parameter calibrated for the unstable operation is not recommended for use since it shows relatively low level of confidence. It was intriguing, however, to glimpse the adverse effect of long acceleration lanes under the unstable operation.

One of limitations of the models is that the models were formulated using 6–lane freeways only. In order to apply the models to 4–lane and 8–lane freeways, necessary revision and fine–tuning considering the number of lanes will be crucial. Another limitation is due to a limited data base. The data base used for the calibration of models has freeway flow ranging from 2,000 pcph to 6,800 pcph and ramp demands varying from 200 pcph to 2,400 pcph. The length of acceleration lanes varied from 325 ft to 1,650 ft. This was a wide spectrum of volume and geometric conditions. However, a desire to see a wide range of the ramp demand at one junction which has one specific length of the acceleration lane was not satisfied.

As a final note, it is becoming clear that it will be difficult to develop a level of service system based upon a single measure of effectiveness. While speed does vary to a certain degree through the range of stable flows, the variation is not large. The widely scattered data points of any speed measures at merge areas would also make a proper stratification of level of service on the basis of one speed measure alone difficult. In order to overcome this dilemma, an appropriate combination of the speeds in conjunction with the traditional microscopic merged volume could be utilized as level of service criteria.

		Actual		Developed Models			Comparison		
SITE	PERIOD	SR	S1	SR(1)	SR(2)	S1	100*(Model-Actual)/Actual (%)		
5	1	46.4	46.0	47.1	48.9	48.8	1.4	5.3	6.1
5	2	46.3	47.9	46.0	47.7	48.3	-0.7	2.9	0.8
5	3	47.4	48.3	48.4	50.4	50.3	2.0	6.4	4.2
5	4	47.0	47.1	47.6	49.7	50.2	1.3	5.7	6.5
5	5	47.2	47.9	47.3	49.4	49.4	0.2	4.7	2.9
5	6	48.1	48.5	49.1	50.6	50.3	2.2	5.4	3.6
5	7	45.7	46.2	47.3	49.2	49.2	3.4	7.7	6.4
5	8	45.9	46.6	47.3	49.0	50.5	3.0	6.8	8.4
5	9	45.0	45.1	47.2	49.1	48.8	5.0	9.1	8.0
25	1	52.8	50.8	54.6	53.7	55.1	3.3	1.7	8.4
25	2	53.4	52.5	55.0	54.3	55.2	3.0	1.6	5.2
25	3	54.2	52.4	55.0	53.4	54.5	1.4	-1.6	4.1
25	4	53.8	52.3	52.5	51.9	53.2	-2.3	-3.4	1.8
25	5	53.1	51.2	52.0	51.8	52.7	-2.0	-2.5	2.9
25	6	54.3	52.7	53.6	52.3	53.5	-1.4	-3.7	1.6
25	7	53.5	52.8	51.9	51.6	52.2	-3.0	-3.5	-1.2
25	8	53.6	52.3	51.9	51.9	52.7	-3.1	-3.1	0.7
25	9	51.9	50.1	51.6	51.5	52.5	-0.6	-0.7	4.9
25	10	51.2	49.3	49.0	49.0	50.0	-4.4	-4.4	1.5
25	11	51.0	48.9	48.3	47.9	49.7	-5.4	-6.1	1.6
25	12	52.8	50.2	50.6	50.0	51.8	-4.2	-5.4	3.3
25	13	50.4	48.6	48.5	48.2	50.1	-3.7	-4.2	3.1
28	1	51.5	52.2	52.1	51.5	51.9	1.0	-0.1	-0.6
28	2	51.2	51.5	51.1	50.4	50.5	-0.2	-1.5	-2.0
28	3	50.2	51.3	49.4	49.1	49.1	-1.6	-2.2	-4.4
28	4	51.5	52.0	50.6	50.0	50.5	-1.6	-2.9	-3.0
28	5	51.7	52.4	49.2	48.8	50.4	-4.8	-5.6	-3.8
28	6	51.3	52.4	49.8	48.9	49.3	-2.9	-4.7	-5.9
28	7	51.2	51.6	49.7	48.8	49.1	-2.9	-4.7	-4.8
28	8	50.3	50.7	50.3	49.6	49.7	-0.1	-1.5	-1.8
28	9	50.6	51.3	49.4	49.0	49.3	-2.3	-3.1	-3.9
28	10	51.1	52.1	50.5	49.4	50.1	-1.3	-3.4	-3.8
28	11	51.4	52.4	49.3	48.7	49.9	-4.0	-5.2	-4.8
28	12	50.5	51.1	50.3	49.6	49.3	-0.5	-1.9	-3.6
28	13	50.6	51.0	48.3	47.8	48.7	-4.5	-5.5	-4.5
28	14	50.4	51.1	49.0	48.3	48.3	-2.8	-4.2	-5.5
28	15	49.0	50.5	48.0	47.3	47.2	-2.2	-3.5	-6.5

SR(1): WEAVING FORMAT SR(2): REGRESSION FORMAT

Table 4-3. Validation of Predictive Models for Performance Variables

REFERENCES

1. *Highway Capacity Manual 1965*, HRB SR 87, HRB, Washington, D.C. 1965
2. *Highway Capacity Manual 1985*, TRB SR 209, TRB, Washington, D.C.1985
3. "Capacity of Ramp－Freeway Junction : NCHRP Project 3－37 Interim Report", TTRC, Polytechnic Univ., 1991
4. Warrall R. and et. al. "Merging Behavior at Freeway Entrance Ramps: Some Elementary Empirical Consideration", HRR－157, HRB, Washington, D.C.
5. Drew, D. "Gap Acceptance Characteristics for Ramp－Freeway Surveillance and Control", HRR－157, HRB, Washington, D.C.
6. Dawson, R. and Michael, H. "Analysis of On－Ramp Capacity by Monte Carlo Simulation", HRR－118, HRB, Washington, D.C.
7. Drew, D., Wattleworth, J., and et al. "Gap Acceptance in The Freeway Merging Process", HRR－208, HRB, Washington, D.C.
8. Drew, R. and et. al. " Determination of Merging Capacity and Its Applications to Freeway Design and Control", HRR－244, HRB, Washington, D.C
9. Munjal, P. and Hsu, Y. "Prediction and Stability of Freeway Gaps and On－Ramp Merging", HRR－469, HRB, Washington, D.C.
10. Drew, D. and Wattleworth, J. "Gap Acceptance and Traffic Interaction in the Freeway Merging Process", Texas A&M Univ., Report No. HS000 452, 1966
11. Tolle, J. "Vehicular Headway Distribution: Testing and Results", TRR－567, TRB, Washington, D.C.
12. Haight, F. and Bisbee, F. "Some Mathematical Aspects of the Problem of Merging", HRB Bulletin 356, 1962
13. Weiss, G. and Maradudin, A. "Some Problems in Traffic Delay", Operation Research Vol. 10, No.1, 1962
14. Mine, H. and Mimura, T. "Highway Merging Problem with Acceleration Area", Transportation Science Vol. 3, No. 3, 1969
15. Blumenfeld, D. and Weiss, G. "Merging from an acceleration Lane", Transportation Science 5(2), 1971
16. Fukutome, I. and Moskowitz, K. "Traffic Behavior and On－ramp Design", HRB Bulletin 235, HRB, Washington, D.C., 1960
17. Polus, A. and Livneh, M. "Comment on Flow Characteristics on Acceleration Lane", Transportation Research Vol. 21A, No. 1, 1987
18. Michaels, J. and Fazio, J. "Driver Behavior Model of Merging", TRR－1213, TRB, Washington, D.C.
19. Roess, R. "Development of Modified Procedure for Analysis of Ramp Capacity", TRR－772, TRB, Washington, D.C.
20. Theophilopoulos, Nikos. "A Turbulence Approach at Ramp Junctions," Doctoral Dissertation, Polytechnic University, Brooklyn NY, 1986
21. *A Policy on Geometric Design of Highways and Streets*, AASHTO, Washington,

D.C., 1984

22. Gross, D. and Harris, C., *Fundamentals of Queueing Theory: 2nd Ed.*, John Wiley & Sons, NY, 1985

APPENDIX

DATA COLLECTION AND REDUCTION

This study examines the effect of the types/length of the acceleration lanes in particular and focuses on the operation of single lane ramps in 6–lane freeway facilities. Junctions of which acceleration–lane length ranges from 325ft to 1650ft were investigated.

1. Data Collection

Major source of data acquisition is NCHRP 3–37 Research Project. The project team conducted a field investigation of approximately 60 sites in 6 major cities, 13 of which were suitable for this study. Team also had obtained some specific information for these sites regarding location, geometry, and volume and have conducted macroscopic analyses for majority of them.

2. Data Collection Methodology

A typical setup consists of an 8mm video recorder connected to a black & white video camera, each powered by 6 and 12–volt rechargeable batteries (with a battery life of 6–8 hours), respectively. Black & white cameras were chosen over color due to the black & white camera's higher resolution capabilities, which are needed in the data reduction program.

To provide synchronization between cameras, pocket radios are connected from the headphone jack to the audio input of the video recorder. All radios are tuned to the same station, preferably to an "all–news" station where the time is broadcast on a regular basis.

Each site was videotaped for a period of between 2–3 hours. At each trap location where cameras are set up, traffic related information, such as volume count and average speed, for both freeway and acceleration lane are obtained. Traces for entering vehicles was manually attempted to find merging times. Volumes and speeds only for the traffic trespassing trap lines were obtained.

Counts and vehicle classification at upstream and downstream ramps from the study ramp (the one being video taped) will be done with the use of the new magnetic sensing traffic counters.

3. Data Reduction Process

Majority of videotapes acquired for the study has been reduced by manual reduction technique. With help of on–line recording program connected to PC computer, vehicle counts, vehicle classification and speed at each trap location have been summarized.

Each reduced output file contains individual count, speed and time-lapse records. Measurements like merging times spent by ramp vehicles were also obtained using manual count/observation technique. In general, vehicle count and vehicle classification are considered to be reliable, and speed related measures less reliable from trap to trap.

The video tapes were reduced also with the aid of a computer program written specifically for the project where video shows unobstructed images of sections. The program is capable of digitizing the video image and process it to obtain counts, speeds and vehicle classifications for each lane based on pre-defined trap locations for a specified analysis interval. As vehicles pass through the trap area, changes occurring at each trap triggers the "video-detector" as being activated, thereby counting that change as a vehicle passes by. The duration of continuous activity at the trap area indicates the length of the vehicle, and the time it takes for a vehicle to go from one trap to another is used to calculate the vehicle speed.

4. Description of Variables Isolated

Volume/Flow Rate Variables

V_F = freeway flow rate just upstream of merge;

V_R = ramp flow rate;

V_1 = lane 1 flow rate just upstream of merge;

V_{12} = total flow rate in lanes 1 and 2, just upstream of merge;

V_{23} = total flow rate in lanes 2 and 3, just upstream of merge;

V_U ≐ adjacent upstream ramp flow rate;

V_D = adjacent downstream ramp flow rate;

V_M = traditional merge flow rate: $V_1 + V_R$;

V_{R12} = ramp merge flow rate, defined as $V_1 + V_2 + V_R$;

V_{FM} = total freeway merge flow rate: $V_F + V_R$.

Geometric Variables

The length of acceleration lane is measured from the merge gore area where the right edge of freeway lane 1 and the left edge of ramp lane are 2 ft apart to a point where the right edge of ramp lane meets freeway lane 1.

L_A = length of acceleration lane, ft;

L_{AP} = parallel portion of L_A, ft;

L_{AT} = taper portion of L_A, ft;

D_U = distance to the adjacent upstream ramp, ft;

D_D = distance to the adjacent downstream ramp, ft.

Performance Variables

Some new performance variables, by definition, were defined here, and none of them are explicitly considered in current methodologies. Performance variables mainly focused on speed measures.

S_R = average speed of all vehicles in "merging area" defined as the acceleration lane plus lane 1 and 2, from point of merge to 1,500 ft downstream, mph;
S_l = average speed of all vehicles in lane 1 of the merging area, mph; and
S_D = difference in average speeds bet. lane 1 and 2 within the merging area, mph.