The Effect of Acceleration Lanes on Entrance Ramp Operation

(This paper is excerpted from the author's Ph.D. thesis.)

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1. INTRODUCTION AND OVERVIEW

A ramp may be described as a length of roadway providing an exclusive connection between two roadway facilities. As the 1985 HCM 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. Among the three elements, the ramp-freeway junction is perhaps the most important area in terms of traffic operation because this is an area of competing traffic demand for space, which creates conflict or turbulence.

A ramp-freeway junction is generally designed to permit high speed merging or diverging movements to take place with a minimum of disruption to the adjacent freeway traffic stream and a maximum safety to the drivers. An entrance ramp junction inherently bears more disruption than an exiting ramp junction and often leads to breakdown in operation reducing mobility drastically. Accordingly, entrance ramp junctions have been a prime subject of interest to many traffic engineers. A typical entrance ramp junction consists of three kinds of paved roadways; the freeway section, the ramp proper, and the acceleration lane connecting the two.

1.1 Role of Acceleration lanes in Entrance Ramp Junctions

Vehicles from a ramp terminal enter an acceleration lane, and then try to move into a freeway stream. To get into the traffic stream while traveling on the acceleration lane, drivers need to decide whether gaps 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 highway traveled way, 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 where heavy interaction 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. In general, an acceleration lane should, as a minimum requirement, have sufficient length to enable a driver to make the necessary change in speed in a safe and comfortable manner.

1.2 Problem Statement

Traffic engineers frequently face a need to evaluate operational quality and design features of ramp-freeway junctions. A precise analysis/design of the junction is a very important task to them because undesirable operation in one junction can aggravate the operation in an entire freeway corridor.

To date, no analysis method exists which takes into account the impact of acceleration lanes on operational quality of ramp-freeway junctions although differences of acceleration lanes in length and type are expected to bring about 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.

Several empirical studies were conducted in the late 1960's. 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. Some mathematical models for calculation of merging delay experienced by ramp vehicles existed as well, but many of the assumptions made in these heavy probability models were basically unrealistic.

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. It is therefore suggested that a well-established empirical study 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.

1.3 Study Scope and Research Objectives

This research mainly deals with the effect of acceleration lanes on entrance ramp operation and investigates its effect on the capacity and 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.

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/authorities responsible for operating/maintaining freeway facilities prefer the parallel-type acceleration lanes to the taper-type which was rarely found.

Observation of all 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. More specific objectives of the research are stated as follows:

1) Investigate the impact of length of acceleration lanes on lane distribution of freeway traffic volumes, and develop predictive models for lane 1 volume.
2) Identify an optimal length of acceleration lanes within which most drivers merge into lane 1.
3) Develop models describing the behaviors of merging time required by ramp vehicles under the ranges of freeway volumes.
4) Identify responsive indicators of operational changes, reflective of the length of acceleration lanes, which lead to the LOS model development.
5) Compare freeway/ramp volume conditions, under different length of acceleration lanes, which lead to breakdown of operation.

2. BACKGROUND AND LITERATURE REVIEW

Most studies related to acceleration lanes were conducted in the late 1960's. These studies can be categorized into two groups; The first includes several mathematical formulations of merging delay spent by ramp vehicles; the second group includes a few field studies on spatial usage of acceleration lanes by ramp vehicles.

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[7]~[12]. The gap-acceptance model is a probabilistic treatment of such random variables as headways or time-gaps created by successive vehicles in the right lane of the freeway. It utilizes a probability density function of those random variables and simplifies the behavior of ramp vehicles, but 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[23] 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[24] also pioneered in this field and formulated many mathematical models for various types of the gap-acceptance process.

Mine and Mimura[25], extended Weiss and Maradudin's study[24], were able to derive the probability density function (PDF) of delay to merging vehicles. Some important assumptions were made in their formulation such as vehicles on the highway have a constant velocity V and the velocity of merging vehicles $v(\tau) < V$ for any τ, where τ is time, and the length of the acceleration area is infinite.

Weiss and Blumenfeld[26] later revised the previous model[24] and formulated the merging process from an acceleration lane of finite length. It is assumed that cars 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. This approach has at least overcome a barrier of infinite acceleration-lane length imposed by Mine and Mimura's study[25].

Michaels and Fazio[29] 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.[19] 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. Drew later recommended a regression formula for the estimation of critical gap size associated with the type and the length of the acceleration lane.

2.2 Some Empirical Analyses

A study by Fukutome and Moskowitz[27] is of interest due to their analytical approach that traces paths on acceleration lanes taken by merging vehicles. Their findings can be summarized 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[28] 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.

2.3 Treatment of Acceleration Lanes in Current Technical Manuals

The 1985 HCM[2] maintains 13 regression formulas in its chapter 5 - originally developed for its predecessor, the 1965 USHCM - for analyses of various features of ramp-freeway junctions. Each regression equation predicts volumes in freeway outerlane (lane 1 volume) using freeway volume, ramp volume, and geometric/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.

"A Policy on Geometric Design of Highways and Streets", widely known as the AASHTO Green book[32], discusses findings accumulated over time. Such are the findings: The taper-type entrance of proper dimensions usually operates smoothly at all volumes up to the design capacity of merging area.....Some agencies use the taper type for exit and the parallel type for entrances: The operational and safety benefits of long acceleration lanes are well recognized, particularly where both the freeway and ramp carry high traffic volumes.....An acceleration lane length of at least 1,200 ft, plus taper, is desirable whenever the capacity level of traffic is anticipated.

The Green book also 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.

2.4 Deficiencies in the Previous Studies

Mathematical models are usually far from real-world phenomena which have been observed through empirical studies. Their concern is focused only on prediction of merging delays (times) to minor street (ramp) 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. They are:

1) The models begin with lane 1 volume, ramp volume, and their speeds known. Application of Poisson property (exponential PDF for headway distribution) is limited to only freeway lane 1 vehicles and ramp vehicles.
2) Poisson arrival process assumed is usually valid under light ramp traffic condition, and the platoon of traffic and heavy traffic volume situation generally disable the assumption.
3) Distribution function of intercar-headways for freeway tends to underestimate the number of big size gaps within which many merges are effected together by ramp vehicles.
4) Assumption of ramp vehicle's merging behavior that they minimize the merging time, is not realistic since a group of drivers who keep using the large portion of the acceleration lane regardless of mainline traffic condition were observed.
5) The models require empirical assessment of critical gap size or distribution of accepted gaps, which varies from location to location and volume to volume. Finding a critical gap size or distribution is another hindrance to the practitioners.

All previous 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. The outcome from the empirical 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 all the mathematical models. 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.

The HCM models are insufficient to use as performance indicators since the checkpoint volumes over 2,500 pcphpl have been frequently observed without any operational deterioration. In addition, the studies and the observations were done almost 3 decades ago and need to be updated.

3. DATA COLLECTION, REDUCTION AND ANALYSIS

In order to achieve the objectives stated in the introduction, many operational aspects of the junctions pertaining to different types/lengths of acceleration lanes were carefully investigated. Macroscopic observations of many merge areas formed a data base for analyses. Microscopic observations of merging behavior in the acceleration lanes were also needed to find the length utilized and the time required by most ramp vehicles.

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 dilute the transition undergone.

3.1 Selection of Study Sites and Physical Study Area

This study examines the effect of the length of the acceleration lanes and focuses on the operation of single lane ramps in 6-lane freeway facilities. A total of eleven ramp-freeway junctions with the length of acceleration lanes ranging from 325 ft to 1,650 ft were observed and analyzed; the first eight sites were used for analysis of lane distribution and prediction of speed measures; the last three sites were added only for analysis of merging time and merging distance. Table 3-1 summarizes their characteristics.

Table 3-1. Description of Sites Collected

Site	Upstream Ramp		Downstream Ramp		Volumes		Length of
No.	Type	Distance	Type	Distance	Freeway	Ramp	Acceleration
1	off	600 ft	off	5280 ft	heavy	moderate	950 ft
3	off	4000 ft	off	3200 ft	moderate	heavy	325 ft
8	off	400 ft	off	4500 ft	heavy	low	450 ft
12	off	500 ft	off	7900 ft	heavy	moderate	1100 ft
18	off	1500 ft	off	7920 ft	moderate	low	1000 ft
20	off	1100 ft	off	2900 ft	heavy	moderate	1200 ft
21	off	1350 ft	off	1800 ft	heavy	moderate	1250 ft
23	off	2500 ft	off	10000 ft	moderate	heavy	1650 ft
5	off*	2700 ft	on	2400 ft	heavy	heavy	650 ft
7	n.a.	n.a.	n.a.	n.a.	moderate	light	750 ft
24	on	750 ft	off	4750 ft	heavy	low	650 ft

*: case where lane-drop existed at upstream adjacent ramp or there are more than two cameras failures.

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 between these interchanges were usually more than 3,000 ft.

3.2 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. Typically, this would be 375 ft upstream and 1,125 ft downstream of a merge area. 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.

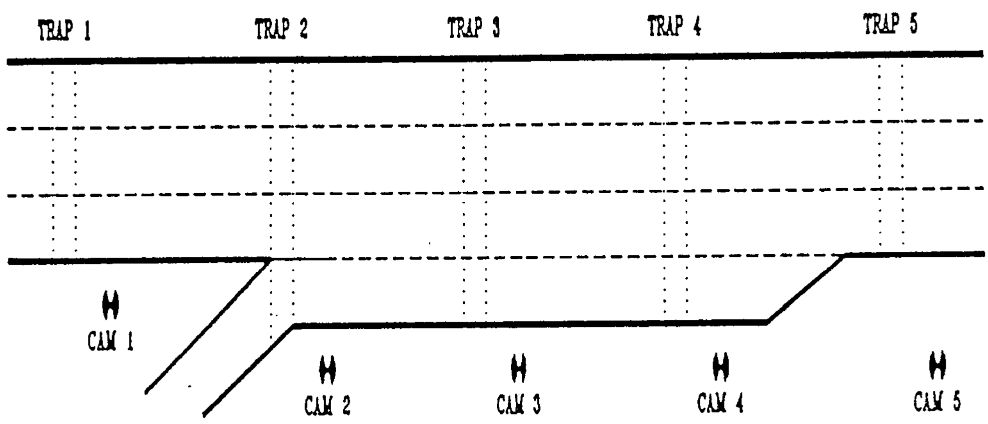


Figure 3-1. Typical Deployment Layout of Video Cameras

A typical setup consists of an 8mm video recorder connected to a black & white video camera. Each camera recorded approximately a 200 foot discrete section of a merge area within which a "trap" line perpendicular to the roadway was placed.

Each site was videotaped for a period of between 2-3 hours. At each trap location where cameras are set up, volume counts, vehicle classification, and average speeds, for both freeway and acceleration lane are obtained. For the sites where a full length of acceleration was visible, traces for entering ramp vehicles were manually reduced to find merging times and lengths. Counts and vehicle classification at upstream and downstream ramps from the study ramp being video-taped were done with the use of magnetic traffic counters.

3.3 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. 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.

A random sampling of merging time spent by ramp vehicles was manually accomplished. This requires individual observation of most entering ramp vehicles. Using the data recording program and summary program written for this study, one could identify time of entry and time of completion of merges by hitting corresponding keys.

In order to acquire information on merging lengths and spatial usages by ramp vehicles in the acceleration lanes, for sites where a full section of an acceleration lane is seen on monitors, ramp vehicles passing the lines spaced at every 150 ft on the acceleration lanes were counted manually and summarized by 5-minute periods.

3.4 Analyses of Data

The data obtained from the program/manual reduction and portable traffic counters is transferred to a spreadsheet program for simple analysis (e.g. tabular summaries, multiple regression, identification of maximum and minimum values, calculations of mean, standard deviation, etc.).

Subsequently the data is imported into a statistical analysis software package (STATGRAPHICS) for creating scatterplots and performing correlation analyses, linear or non-linear regression analyses, and some advanced statistical analyses.

From the data 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

4. MODEL DEVELOPMENT

To examine the highly likely impact of acceleration lanes on overall operations of ramp-freeway junctions, three major approaches of evaluations of the impact were taken which led to the establishment of several models.

The first was to examine the impact that the acceleration lanes have on lane distribution of freeway volumes. Conclusions drawn from this study approach was ratified through microscopic observations of lane-changes made by lane 1 drivers approaching ramp-nose areas. The second approach was to develop level of service models which incorporate the length of acceleration lanes. Some newly defined speed measures were successfully predicted. Thirdly, possible changes in capacities at ramp-freeway junctions due to the length of the acceleration lanes were discussed. Some unfavorable aspects of unstable operation at the junctions with long acceleration lanes were described.

Each approach examines several candidate variables. However, many of them including microcopic variables will be briefly summarized here on account of limited space.

4.1 Impact on Lane Distribution of Freeway Volumes

Freeway vehicles tend to move away from the outer lane due to the conflict anticipated by freeway drivers. Some of most apparent factors comprising this conflict freeway vehicles perceive are the volumes of entering ramp vehicles and the spatial configuration of a junction.

Among the volumes on three different lanes, the lane 1 volume is most affected by two factors cited above and best relates to overall junction operation. Lane 1 volume is defined as the volume in lane 1 immediately upstream of the merge point. It is the single most important determination in the models of the 1985 HCM, and is critical if any sort of microscopic analysis is to be undertaken.

For unstable flow condition, lane 1 flow rates quickly equalizes with other freeway lanes depending upon ramp flow rates so that the impact of acceleration lanes on the lane distribution of freeway volumes is very hard to discern. For stable flow condition, however, a certain level of ramp volume on a very short acceleration lane is more likely to increase a visual intensity of the ramp volume perceived by freeway drivers than the same ramp volume with a long acceleration lane is. Consequently, the subject junction discourages more freeway drivers from using Lane 1. It was concluded that the length of acceleration lanes has a positive relationship with the lane 1 volume on the basis of the analyses done for this study.

Prediction of Lane 1 Volume

Lane distribution immediately upstream of a ramp-freeway junction is a complex issue. It is influenced by the total freeway volume, the ramp volume, the proximity and volumes on adjacent ramps, and other issues. And Lane 1 flow rates have been traditionally related to the freeway volume, the ramp volume, and the distance/volume ratio on adjacent ramps.

As an analysis effort to identify any specific impacts of acceleration lane design, the length of acceleration lanes were added to the traditional format of the lane 1 volume prediction models in the 1985 HCM. The lane 1 flow rate is a function of V_F,V_R, V_U/D_U, and L_A. Since all the sites had close upstream off-ramps and relatively far downstream off-ramps, the lane 1 flow rates at junctions paired with upstream off-ramps were analyzed.

Table 4-1. Correlation Matrix for Independent Variables

	V_F	V_R	V_U/D_U	L_A
V_F	1.00	-.601	.402	.408
V_R	-.602	1.00	-.456	-.314
V_U/D_U	.402	-.456	1.00	-.179
L_A	.408	-.314	-.179	1.00

As shown in the Table 4-1, the relatively moderate correlation between V_F and V_R still existed. This was due to heavy traffic flow conditions which made up the majority of data points. If and when the relationship is observed for many sites with full ranges of traffic volumes, the problem associated with these two variables will be extenuated. Historically, the independence between these two variables has been assumed because and V_R is usually location specific and not affected by V_F unless the V_F is very heavy. Typical relationships between V_1 and the four variables shown in the following figures.

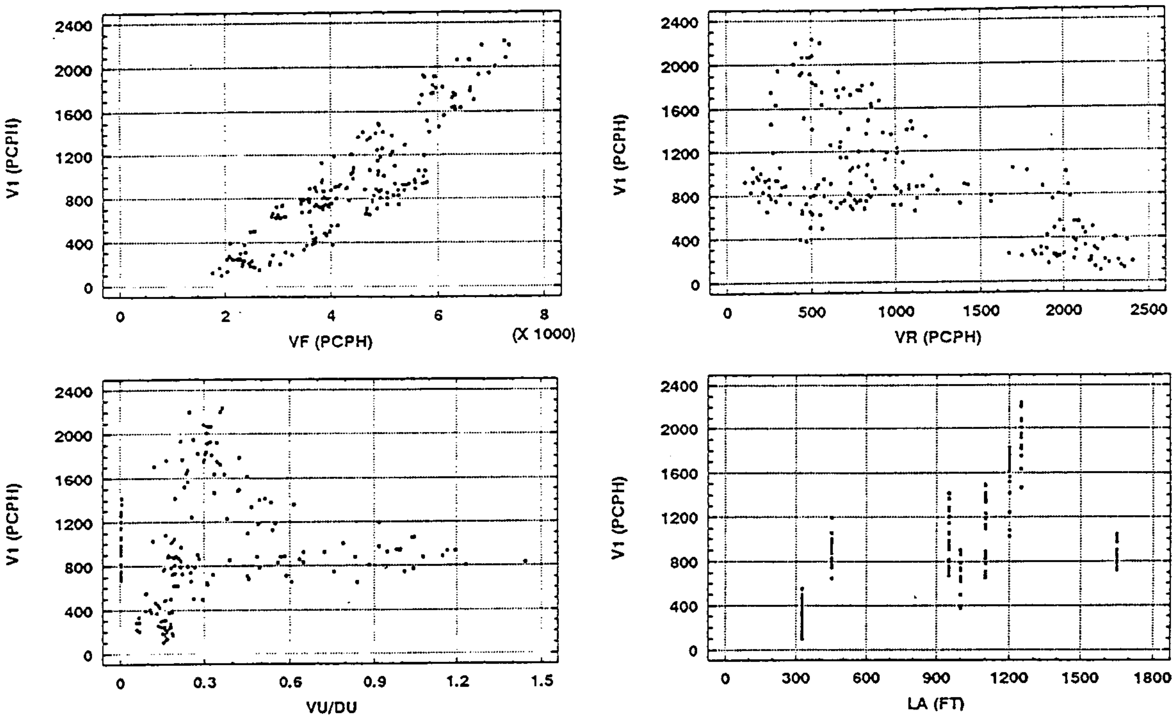


Figure 4-1. Relationships between V_1 vs Four Variables

193 data points from the eight sites, which include 160 stable and 33 unstable data points were analyzed. Resulting regression equations were compared for both stable condition and unstable condition. They are:

$$V_1 = -312.4 + .2907V_F - .1252V_R - 358.7(V_U/D_U) + .1826L_A \ : \textit{stable} \qquad \text{Eq. (1)}$$

Table 4-2. Model Fitting Results for Eq. (1)

variables	constant	V_F	V_R	V_U/D_U	L_A
t-value	5.2	27.1	6.3	7.8	6.1
	R-SQ.=	0.9205		SE=	134.7

$$V_1 = 422.5 + .1881V_F - .6009V_R + .5324L_A \ : \textit{unstable} \qquad \text{Eq. (2)}$$

Table 4-3. Model Fitting Results for Eq. (2)

variables	constant	V_F	V_R		L_A
t-value	0.8	2.6	3.6		1.1
	R-SQ.=	0.8169		SE=	136.3

$$V_1 = -421.9 + .3122V_F - .1135V_R - 349.3(V_U/D_U) + .2345L_A \ : \textit{all} \qquad \text{Eq. (3)}$$

Table 4-4. Model Fitting Results for Eq. (3)

variables	constant	V_F	V_R	V_U/D_U	L_A
t-value	5.5	23.4	4.4	6.3	6.2
	R-SQ.=	0.8821		SE=	178.2

For stable flow condition, all the coefficients were significant. The signs of the coefficients on the variables were all consistent with logic, and L_A specially held a big significance suggesting that short acceleration lanes force the freeway drivers to move away from the lane 1.

For unstable conditions, however, the significance level for all the coefficients dropped drastically. Particularly, no significance on V_U/D_U was expected because the vehicle distribution becomes almost equal among the lanes after a breakdown. The regression analysis without this term improved the quality of fit. A coefficient on L_A was not significant, either. But it still appears to play a role to a limited extent. Moreover, a relatively high significance level on V_R strongly suggested that V_R is governed by V_1 to a greater degree under the unstable condition. In general, inherent complexities of the unstable flow conditions pushed down the overall quality of fit.

A slightly different rationale for prediction of lane 1 flow was applied; freeway drivers' anticipation of the conflict in merge areas is more likely to be affected by their impression of mixed image of prevailing conditions. Two major conditions which constitute such conflict would be the ramp flow and the geometric layout (length of acceleration lane).

Therefore, it would be more logical to assume that the lane 1 flow rates are subject to composite impact of ramp flow and length of acceleration lane. Using V_R/L_A, a dividend of two independent variables, as an interacting variable, another set of multiple linear-regression analyses was performed.

$$V_1 = -237.3 + .3058V_F - 45.88(V_R/L_A) - 422.76(V_U/D_U) \quad : \textit{stable} \qquad \text{Eq. (4)}$$

Table 4-5. Model Fitting Results for Eq. (4)

variables	constant	V_F	V_R/L_A	V_U/D_U
t - value	4.3	27.1	8.1	10.3
	R-SQ.=	0.9057	SE=	146.7

$$V_1 = 1052.4 + .19185V_F - 728.87(V_R/L_A) \quad : \textit{unstable} \qquad \text{Eq. (5)}$$

Table 4-6. Model Fitting Results for Eq. (5)

variables	constant	V_F	V_R/L_A	V_U/D_U
t - value	2.3	2.9	4.1	
	R-SQ.=	0.8249	SE=	133.3

$$V_1 = -293.55 + .33237V_F - 48.213(V_R/L_A) - 451.97(V_U/D_U) \quad : \textit{all} \qquad \text{Eq. (6)}$$

Table 4-7. Model Fitting Results for Eq. (6)

variables	constant	V_F	V_R/L_A	V_U/D_U
t - value	4.3	24.4	6.7	8.9
	R-SQ.=	0.8662	SE=	189.8

All the coefficients of calibration turned out to be significant and logical. The quality of fit remained almost the same although the number of independent variables was reduced by 1. There were little changes in the estimated coefficients for the variables regardless of changes in sample size in the data base. It would be noteworthy that this equation format with the interacting variable seemed to be a viable one, considering the volatility of the estimated coefficients in the case where ramp flow and length of acceleration lanes were treated as two separate variables.

Overall, whether the length of acceleration lanes and the ramp flow rate were pooled as one or not, the coefficients on the variables associated with the length of acceleration lane were highly significant. Therefore, it can be concluded that the long acceleration lane helps the lane 1 drivers stay in that lane, and the short acceleration lane deters the drivers more from lane 1 than the long acceleration does under the same ramp flow level. The equation (1) and (4) would be adequate models for use since the lane distribution under unstable flow condition is not a matter of concern.

4.2 Prediction of Performance Variables

A current USHCM practice in evaluating the performance of a ramp-freeway junction is based upon the calculation of merged volume. The merged volume is used for LOS evaluation because ramps are the important input-output elements of the freeway and the emphasis is on estimating volumes which will result in gore area.

Using the merged volume as a performance measure, however, seem to often create obscure shortcomings. In the study site 3, resulting merged volumes, however, ranged from 1,000 pcphpl to 2,800 pcphpl without having any significant operational deterioration, and total junctionwide merged volumes varied from 2,500 pcph to 7,800 pcph. 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 a level of service methodology is the identification of measures of effectiveness which can be used to adequately describe the quality of service 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. From the seven sites, 175 data points were analyzed and calibrated.

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:

$$S_W \text{ or } S_{NW} = 15 + \{50/[1+a(1+VR)^b(V/N)^c/L^d]\}$$

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:

$$S_{NW} \text{ or } S_W = 15 + [(S_{MAX} - 15) /(1 + W)]$$

where:

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

S_{MAX}= maximum 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 are assumed not to be seriously impacted by right-hand merge operations; (3) while the length of a weaving section is identifiable, the length of a merge area is not; as shown in Figure 4-2, 1,500 ft was chosen as this was a length which encompassed most acceleration lanes, and appeared to approximate the area of maximum impact when site videos were reviewed; (4) the design speed or free flow speed of the freeway should be used as substitues of S_{MAX} in the algorithm.

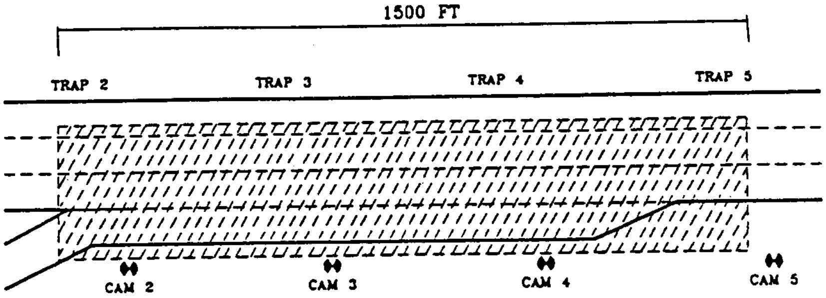


Figure 4-2. Influence Zone of Merge Area

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 of the weaving algorithm. Several forms for this factor was 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, 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, 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:

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

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

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 ongoing 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, different ratio terms were developed for the calibration. MR1 is a microscopic merge ratio and MR3 is a ramp volume to V_{R12} ratio. These two, when added to value of 1, is bounded with a minimum of 1 and a maximum of 2. The obvious forms tried for this ratio term, 1+MR, were:

$$1+MR1 = 1+ V_R/V_M$$
$$1+MR3 = 1+V_R/V_{R12}$$

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 Appedix.

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 two merge ratios for MR required four different regression analyses. Table 4-8 and 4-9 show the results produced for various non-linear regressions against the merge intensity factor, M;

$$M = a\,(1+MR)^b\,(V_{R12})^c / L_{AP}^{\,d} \qquad : \textit{stable}$$

Table 4-8. 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
S_{DSGN}	1+MR1	.0001753	1.00366	1.09074	0.19785	0.81
	1+MR3	.0001969	1.36636	1.04405	0.14483	0.83
	t -value	12 - 13	9.0 - 10.1	14 - 15	5.4 - 7.1	

$$M = a\,(1+MR)^b\,(V_{R12})^c / L_{AP}^{\,d} \qquad : \textit{unstable}$$

Table 4-9. Model Fitting Results for M

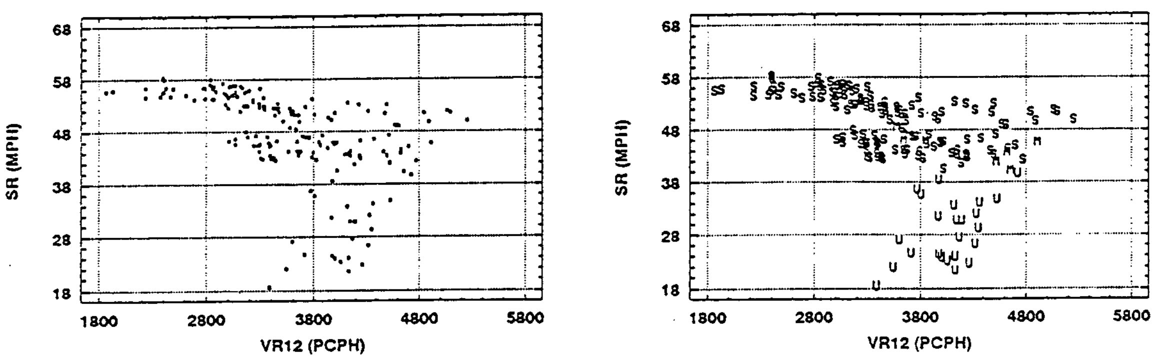
S_{MAX}	1+MR	a	b	c	d	R^2
S_{FF}	1+MR1	478565	-8.6864	-6.1923	-6.2721	0.46
	1+MR3	79001-E3	-3.9787	-4.2412	-2.8073	0.29
S_{DSGN}	1+MR1	353218	-8.3779	-5.9897	-6.0721	0.46
	1+MR3	49200-E3	-3.8112	-4.1017	-2.7203	0.29
	t -value	0.8 - 1.1	0.3 - 2.6	1.3 - 3.4	0.2 - 2.9	

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 V_{R12} 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} (length of parallel portion) 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 four 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.

Figure 4-3 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 V_{R12} 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.



a. Typical Trend Shown

b. Stable, Unstable Shown

Figure 4-3. Relationship between S_R and V_{R12}

4.3 Capacity Issues

The Freeway Subcommittee of the Highway Capacity and Quality of Service ·Committee has agreed that the most reasonable definition of capacity is the outflow from a bottleneck location. Merge areas are the most common of these, where the number of lanes entering the area are generally larger than the number leaving. Thus, when queues appear on the entering roadways, analyst can be assured that sufficient supply of vehicles exists to fill the capacity of the downstream section.

Of the seven on-ramps studied, many situations existed. Site 1 broke down during three 5-min periods but had discharging problem: Site 3 did not break down, but 3 periods experienced slow-down at the downstream presumably due to very high flows engaged: Site 8 also underwent slow-down at 1,500 ft downstream of the gore for 5 minutes, but did not affect the flow at the gore; Site 12 broke down after 9 stable periods and recovered its stable condition for 4 periods, and it again broke down: Site 18 experienced only stable conditions throughout the study periods: Site 20 broke down for 2 periods and recovered: Site 21 also broke down after 9 stable periods, and the formation of queue was visible in the freeway.

In order to observe the capacity of a merge junction, there can, of course, be no additional breakdowns downstream which influence discharge from the merge when such observations are made. This was the prime concern associated with estimation of capacities at the merge areas.

To make sure that no influence from breakdowns downstream existed, a relationship between average speeds in the lane 1 and time periods was investigated for each site, and comments written at the fields were reviewed. If the breakdown observed occurred only from traffic conditions at the study junction without having any downstream breakdown, the speeds measured should show the following history; the speed declines steadily; then it drops drastically due to the breakdown/major slow-down; and it should hopefully bounce back due to the speed recovery of the vehicles leaving the physical bottle neck after some unstable periods. Site 3, site 8, site 12, site 20, and site 21 had shown such speed trends. Figure 4-4 is a typical speed-flow relationship for the five sites.

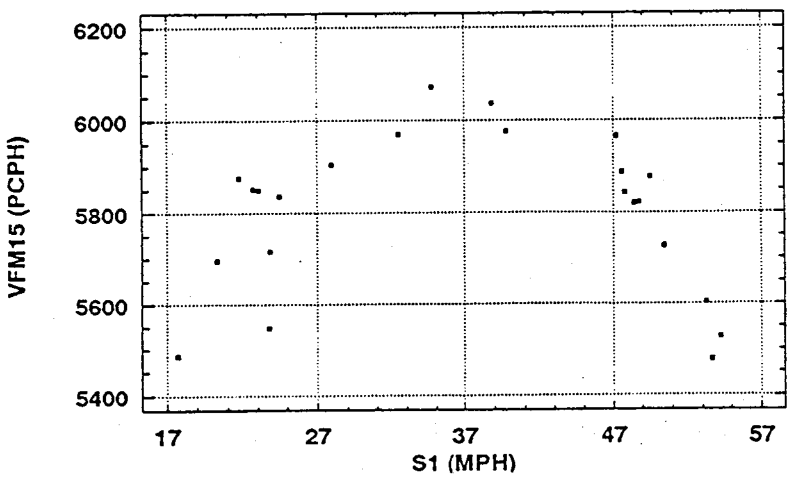


Figure 4-4. Typical Relationship between S_1 and V_{FM15} at Sites

From 5 minute counts available for each site, 15 minute volume counts were obtained by integrating three consecutive 5 minute periods, and then hourly flow rates were calculated. This was intended to follow the definition described in the 1985 USHCM and needed to avoid any overestimation of capacity which might be introduced from using the highest 5 minute count observed. As each site had its distinctive length of the acceleration lane, the hourly flow rates were plotted against the length of acceleration lane. The maximum flow rates appeared to be correlated to the length of acceleration lane. Figure 4-5 shows a possible relationship, under worst scenario case, between the capacity of merge junction and the length of acceleration lane.

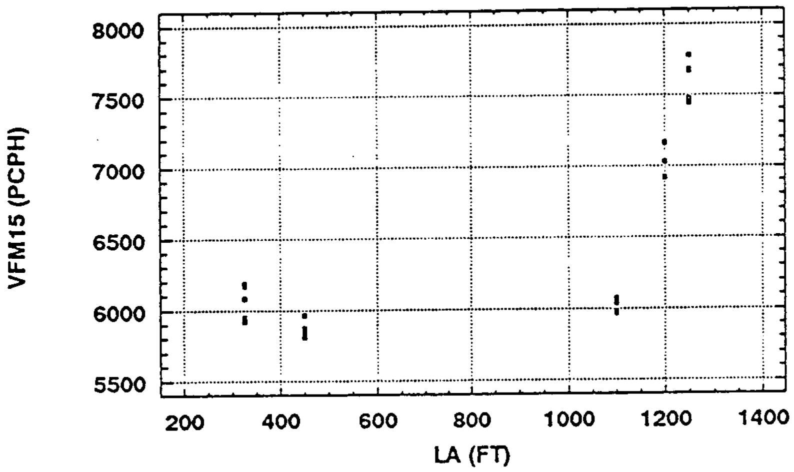


Figure 4-5. Hourly Flow Rates vs Length of Acceleration Lane

The highest discharge rate observed occurred at Site 21. As the length of the acceleration lane changes from 325 ft to 1,250 ft, the observed highest flow rate went up as much as 1,500 pcph or 500 pcphpl. Site 8 was not appropriate to observe possible volume level for capacity because the slow-down in speed was actually caused by a small van that made a sudden stop at about 1,000 ft downstream of the gore for 2-3 minutes. Site 12 had a discharge problem at 1,500 ft downstream of the gore; from this point, a slightly upgraded bridge was placed that had concrete Jersey barriers on both sides and no clearance areas between the moving lanes and the barriers.

The maximum flow rates at these two sites, if above non-ideal conditions were eliminated, is very much likely to go up. Therefore, such big differences in the maximum flow rates would be eventually narrowed. The relationship between the capacity and the length of the acceleration lane was not conclusive because of the problems in the sites from the NY region such as site 8, and site12. Even with a likelihood that those sites would have much higher capacity values, there still appears to be a strong positive relationship between the capacity of merge junction and the length of acceleration lane.

As shown in Figure 4-5, possible capacity volumes which can be deduced from the observations of five highest flow rates - hourly flow rates calculated from 15-min counts - for each site are significantly higher than 6,000 pcph (2,000 pcphpl). It is obvious that, when and if ideal geometric conditions are present, capacity flow rate often reaches up to 2,500 pcphpl.

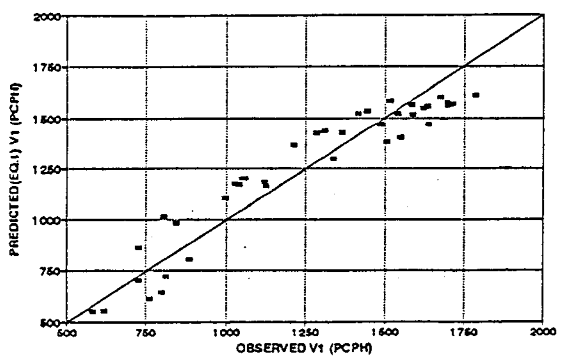
The microscopic merged flow, V_M, was also well over 2,500 pcph for most sites. This apparently reflects the effect of acceleration lanes; the merge operation is no longer a point process because the acceleration lanes render many a diffusion opportunities for ramp drivers. The calibration of threshold values for Level of Service criteria soley based upon this microscopic merged volume should be reexamined or requires in-depth discussions.

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 which came from three sites not used in the development was tested. It would be noteworthy that the sites used were rather unusual because (1) they carried relatively high heavy vehicles and (2) the horizontal or the vertical alignment conditions of the roadways of the three facilities were non-ideal unlike the sites used in the calibration. Therefore, a high accuracy of the models in their prediction capabilities was not expected.

To begin with, the lane 1 volume prediction models developed were tested. The following two plots represent the relationships between the observed lane 1 volumes and the predicted lane 1 volumes. The prediction capability of the developed models, equations (1) and (4) hold validity showing highly precise estimation in the heavy flow region. It was also obvious that drastic changes in characteristics of traffic flow, during the past three decades, could not been accommodated by the HCM models.



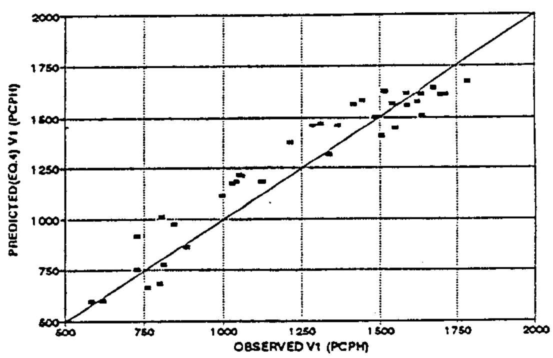


Figure 4-6. Plot of Actual V_1 vs Predicted V_1 Using Eq. (1) & Eq.(4)

To test the validity of the developed models in a more precise way, the Chi-square goodness of fit test was conducted. For 37 sets of data points, frequencies of occurrences for seven volume ranges were counted for the actual lane 1 volume and the two predicted lane 1 volumes respectively. And test statistics were calculated and compared to the Chi-square values in order to see if a hypothesis that the distribution of the predicted values is the same with that of the actual values, is retained. Table 4-10 shows the frequency table and the calculated test statistics. It was found that the two models all make fairly good fits for a significance level of 5%.

Table 4-10. Chi-square Goodness of Fit Test for Two Models

range	actual obs.	predicted Eq. (1)	test statistic	predicted Eq. (4)	test statistic
<600	2	3	0.33	2	0.00
<800	5	4	0.25	5	0.00
<1000	5	3	1.33	3	1.33
<1200	6	6	0.00	6	0.00
<1400	5	5	0.00	4	0.25
<1600	8	14	2.57	11	0.82
<1800	6	2	8.00	6	2.27
sum	37	37	X^2=12.49	37	X^2=2.40

(df= 7-1, α=.05): χ^2=12.59

All the predicted values from the developed models for performance variables were compared to the actual measures using the same three sites which were not used in the development of the models. In general, most of the predicted values turned out to be accurate demonstrating less than 5% of relative difference between the predicted and the actual.

5. CONCLUSIONS AND RECOMMENDATIONS

This paper has presented some predictive models pertaining to the quality of operation in freeway-ramp junctions. Major effort has been made to incorporate the length of acceleration lanes, into analytical procedures. A set of regression models for both stable condition and unstable condition was developed to differentiate the effect of acceleration lanes. The rationale was clear such that, although a designer cares about stable operation most of the time, he or she should not overlook the adverse role of long acceleration lanes under unstable operation that most systems eventually will experience.

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.

5.1 Conclusions

The impact created by one of geometric elements on lane distribution of freeway traffic is not easy to assess because impact of geometric/traffic conditions in close adjacent ramps dominate lane 1 volume. Screening out an independent impact of factor variables seemed nearly impossible because the lane 1 flow is also subject to many other redundant information. With a well-structured database, however, the impact of acceleration lanes was successfully demonstrated using linear regression models. The lane-shift behavior in the lane 1 right upstream of the subject gore also enabled the author to fortify the conclusion on the reaction of the lane 1 drivers to the ramp traffic travelling on a certain acceleration lane. The regression models clearly explained the impact due to the length of acceleration lanes. The findings were congruous with a common expectation that lane 1 drivers tend to stay on that lane if they are presented with a long acceleration lane which is supposed to reduce the turbulence of merges.

The optimal length of acceleration lanes was suggested based upon an empirical study conducted. With advantages of video camera deployment, several distance points in the acceleration lanes were recorded, and relative usage of those points by ramp vehicles was summarized. It was concluded that the usage of section between 900 ft to 1,050 ft in the acceleration lanes remained almost constant no matter how much freeway/ramp volumes was engaged. The fact that no more than 4% to 5% of the ramp vehicles used up to this section can justify a 1,000 ft long acceleration lane as a practically optimal one for use.

A merging time study revealed that long acceleration lanes tend to ease the tension of the ramp drivers so that it increases the merging time required. However, this does not mean that provision of a extremely short one can make merging times shorter; the junctions with YIELD or STOP signs actually suffer from excessive times required for merges because they are inherently different from the ones that have a certain length of an acceleration lane where drivers have a space to enter and chances to adjust their speeds.

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. Each parameter calibrated for the unstable operation had shown the reversed role of acceleration lanes with a relatively low confidence level, but the fact that facilities with long acceleration lanes underwent severer congestions is highly interesting. Predictive models for three speed measures such as S_R, S_1 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.

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.

Regression analyses utilizing kinetic energy concept served not as applicable models but as a way to ratify the reversal of the role of acceleration lanes. Predicting the external energy using given traffic volumes is nothing different from predicting actual speeds of streams. However, it produces much smoother relationships and much clearer presentation of the state of operation. It seemed to be a sound way to assess the impact of geometric conditions at any freeway system.

The issue of capacity is still difficult to assess, as the presence and impact of downstream congestion on merge area frequently caused discharge problems. Besides, a regional difference was too immense. The sites from the northeastern USA showed only 6,200 pcph for a total thruput while the sites from Florida have more than 7,500 pcph. Unless traffic demand has a non-decreasing pattern until it reaches a breakdown condition, identification of capacity value is not valid. Any breakdown caused by unsound reasons should be disregarded for capacity estimation as well. Nonetheless, the flow rates over 2,500 pcphpl were frequently observed. With a limited number of observations made, a possibility that a junction with the longer acceleration lane would have the higher capacity was demonstrated.

The final note is about the database used. In the beginning, it was hoped that one specific site would carry a wide range of freeway/ramp traffic. It was, however, too idealistic to happen because the ramp demands in most junctions remained more or less the same while freeway tends to show variety of flow levels. 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 very wide spectrum of volumes 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.

5.2 Recommendations for Future Research

This research work has identified a number of interesting avenues for potential future work to pursue. As described in the previous section, the data base is limited in 6-lane freeway-ramp junctions. Extensive studies on 4-lane and 8-lane freeway-ramp junctions should be considered, and necessary revision and fine-tuning will be crucial.

A comparative study regarding the types of acceleration lanes is suggested. More extensive study on the lane-shifting behavior right upstream of the gore area might be meaningful as well. As far as merging time study is concerned, the shorter analysis period would be a more fruitful approach to take because of microscopic aspects of the process. In the midst of analysis, the author frequently speculated that measuring of acceleration and deceleration noise as a quality measure for the operation of freeway-ramp junction would be worth trying.

Lastly, it is strongly believed that, whenever speed measures are engaged, the precise measurement is the utmost important task of all, and a great care should be taken through the devices or the reduction techniques.

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APPENDIX
(The list below defines the 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

L_A = length of acceleration lane, ft;
L_{AP} = parallel portion of L_A, ft;
L_{AT} = taper portion of L_A, ft;
L_{AC} = composite length $L_{AP} + L_{AT}/2$, ft;
D_U = distance to the adjacent upstream ramp, ft;
D_D = distance to the adjacent downstream ramp, ft.

Performance Variables

S_{1M} = minimum average speed of vehicles in the primary merge cell (lane 1) immediately after the initial merge point, mph; the lowest value among the 5 camera locations only in lane 1 is chosen for this variable;
S_{FM} = minimum average speed of all vehicles across all freeway lanes, mph; average speed of all vehicles is available for each camera location, or trap -- the lowest value is chosen for this variable from among the 5 camera locations or traps;
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_1 = average speed of all vehicles in lane 1 of the merging area, mph; and
S_D = difference in average speeds between lane 1 and lane 2 within the merging area, mph.