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ABSTRACT

This report documents work accomplished during Fiscal Year 2005-2005 as a part of a research project entitled "New Approach to Bottleneck Capacity." This project is developing an alternative to the traditional Highway Capacity Manual approach to capacity analysis in which capacity flow [either pre-queue flow (PQF) or queue discharge flow (QDF)] is related to a set of intervening variables, including the average time gaps in the critical lane (i. e., that with the highest flow rate) and the distribution of flow across the lanes, represented by the critical lane flow ratio (i. e., the flow in the critical lane divided by the average flow per lane). These intervening variables, in turn, are to be related to the geometric characteristics of bottleneck sites, their vehicle populations, and their driver populations. Work to date has included the collection and analysis of data, analysis of traffic data to document flow characteristics at individual study sites, and an analysis of the relationships among the various traffic flow characteristics, including relationships among the intervening variables and between the intervening variables and capacity flows. Major findings to date are that (*a*) there are significant differences in the mean values of the flow characteristics during different episodes of PQF and QDF at individual sites; (*b*) means of flow characteristics are significantly different among the sites (with the exception of critical lane average time gaps in PQF); (*c*) flow variances also differ significantly among the sites; (*d*) QDF appears to vary by time of day at some sites; (*e*) critical lane average time gaps and critical lane flow ratios are not correlated with one another in either PQF or QDF; (*f*) there is a significant negative correlation between the time gaps and the flow per lane; and (*g*) there is a very strong negative correlation between flow in the critical lane and critical lane average time gaps; when plotted, this relationship is virtually linear. On the basis of these findings, models relating flow per lane (for PQF and QDF) to critical lane flow ratios and critical lane average time gaps are proposed for use in the next stage of the research, which will focus on relating the flow ratios and time gaps to the geometric, vehicle-population, and driver population characteristics of the study sites.

Keywords: Bottleneck Capacity

EXECUTIVE SUMMARY

This report documents work accomplished during Fiscal Year 2004-2005 as part of a research project entitled "New Approach to Bottleneck Capacity Analysis." This project is developing an alternative to the traditional Highway Capacity Manual approach to capacity analysis in which capacity flow is related to a set of intervening variables, including the average time gaps in the critical lane (i. e., that with the highest flow rate) and the distribution of flow across the lanes, represented by the critical lane flow ratio (i. e., the flow in the critical lane divided by the average flow per lane). These intervening variables, in turn, are to be related to the geometric characteristics of bottleneck sites, their vehicle populations, and their driver populations. Work to date has included the collection and analysis of data, analysis of traffic data to document flow characteristics at individual study sites, and an analysis of the relationships among the various traffic flow characteristics, including relationships among the intervening variables and between the intervening variables and capacity flows.

The proposed to capacity analysis begins with the observation that flow is the reciprocal of the average headway and that and that the headway, which is the time separation between common points on successive vehicles (for instance, front bumper to front bumper), may be decomposed into the passage time (that is the time it takes the vehicle to pass a point) and the time gap between the rear of the lead vehicle and the front of the following one. Consequently, the maximum flow in the critical lane at a bottleneck is a function of the average values of the time gaps and the passage times in capacity flow, and the average flow per lane is a function of the critical lane flow and the critical lane flow ratio. The overall relationship is

$$\bar{q} = \frac{1}{r_c\left(\bar{g}_c + \bar{p}_c\right)}$$

where $\bar{q}$ = average flow per lane

$\bar{g}_c$ = average time gap in the critical lane

$\bar{p}_c$ = average passage time in the critical lane

r_c = critical lane flow ratio

Time gaps and lane flow distributions appear to be fundamental features of driver behavior. Since past research indicates that critical lane flow ratios and average time gaps in the critical lane vary widely among bottleneck sites, it is hoped that they will prove to be stable and predictable features of the sites, and that this will allow more accurate predictions of bottleneck capacity.

As originally proposed, the study was to have involved analysis of data from at least 20 freeway bottlenecks. During the previous fiscal year, 25 potential sites in the San Diego, Seattle, and Minneapolis-St. Paul metropolitan areas were identified. As a result of various problems with the availability and quality of the automatically-collected data

used by the project, several of these had to be eliminated outright. In addition, most of the sites in San Diego did not have detectors located directly in the bottleneck sections. It had originally been hoped that the average time gaps in the critical lanes would be similar at the bottleneck and at locations immediately upstream from the bottleneck, so that these sites could still be used in the development of models of bottleneck capacity. When this proved not to be the case, these sites had to be eliminated from the set used to develop the models, although they can still be used to verify the models once they are developed. Eventually, the set of sites to be used to develop the models was reduced to 15.

For each of the sites, traffic data were collected for a total of about 60 weekdays during the summer of 2004. In addition, a smaller set of data collected during an experiment in the Minneapolis-St. Paul area in which ramp meters were turned for the period October 16 to December 1, 2000 was made available by researchers at the University of Minnesota. These data were screened to eliminate obviously corrupt data, days with rainfall, days with known incidents, and time periods with obviously anomalous flow.

It is well established that the highest-volume uncongested flows occurring immediately before flow breakdown are typically higher than the queue discharge rate following breakdown, and that such pre-queue flows can sometimes last for considerable periods of time. There is continuing debate among researchers about whether bottleneck capacity should be defined as queue discharge flow (QDF), pre-queue flow (PQF), or some combination of the two. The approach followed in this research is to consider PQF and QDF as both representing “capacity” in some sense, and to attempt to model both conditions. An initial step in data reduction was to identify periods of PQF and QDF. Queue discharge flow was taken to begin at flow breakdown, as indicated by an abrupt decrease in speed upstream from the bottleneck and to end when speeds recovered. Pre-queue flow was defined as any period of near-constant flow ending in local flow breakdown. Abrupt decreases and increases in speed were identified from re-scaled plots of cumulative speed, and periods of near-constant flow were identified from re-scaled cumulative flow plots. Once periods of PQF and QDF were identified, means and standard deviations of flow, time gaps, and critical lane flow ratios were calculated for each episode. These were subsequently aggregated over all episodes of PQF and QDF at each site to produce means and standard deviations of the flow characteristic for each site.

Analysis of variance was used to test whether there were significant differences in the mean values of the flow characteristics during different episodes of PQF and QDF at each site. In most cases, there were significant differences. This may indicate that PQF and QDF are not entirely homogeneous flow conditions – that is, the characteristics of what has been identified as PQF or QDF are not necessarily the same every time they occur. Analysis of variance was also used to test whether there were significant differences among the different sites in the means of the flow characteristics, and a similar technique, Levene’s Method, was used to test for differences in variances in flows. Means of flow characteristics were found to be significantly different among the sites with the exception of critical lane average time gaps in PQF. Flow variances were also found to differ significantly among the sites. In the case of the means of the time gaps in PQF, the result

of the statistical test is believed to be a result of relatively large variance in the time gaps at the different sites; otherwise, there is considerable evidence that the time gaps do vary among the sites in PQF. In terms of practical significance, the difference in the average flow per lane between the site with the highest flow and that with the lowest flow was about 35 percent of the average flow per lane for PQF and 27 percent for QDF. For critical lane flow ratios, the difference between the highest and lowest site was about 19 percent in PQF and 17 percent in QDF; for critical lane average time gaps, it was 33 percent in PQF and 32 percent in QDF.

Another interesting result of the analysis of the flow characteristics for the individual sites was evidence that, at some of them, QDF varies by time of day. The sites in question experienced long period of congestion, so that queuing extended outside the traditional commute trip periods. Queue discharge flows at these sites were averaged for 30-minute intervals, and analysis of variance was used to confirm that the differences among them were statistically significant. In general, QDF at morning-peak sites peaked early in the period of congestion (around 6:00 or 6:30) whereas that at evening-peak sites peaked relatively late, around 17:00 or 17:30. These findings may indicate that queue discharge rates are affected by the relative presence of commute traffic; however, the number of sites involved was too small to warrant any definite conclusions about this.

Relationships among flow characteristics at different sites were investigated by preparing scatter plots and calculating correlation coefficients. Critical lane average time gaps and critical lane flow ratios were not correlated with one another in either PQF or QDF, suggesting that these two intervening variables are independent of one another. There is a significant negative correlation between critical lane average time gaps and critical lane average passage times, however, in both PQF and QDF. This, coupled with the fact that the time gaps are considerably larger than the passage times, suggests that the passage times can be omitted from the predictive models to be developed. There is also a significant negative correlation between the time gaps and the flow per lane; that between critical lane flow ratios and flow per lane is weaker, and is significant at the 0.05 level for QDF but not PQF. Finally, there is a very strong negative correlation between flow in the critical lane and critical lane average time gaps (approximately -0.91 for both PQF and QDF) and when plotted, the relationship between these two variables is virtually linear. This linearity, which is unexpected in light of the basic equation linking flow to passage time and time gap, is presumably a result of the correlation between passage times and time gaps. Since this relationship appeared to be linear, least-squares regression analysis was used to determine lines of best fit.

On the basis of these findings, the most promising model for relating the intervening variables to flow per lane appears to be

$$q = \frac{1}{r_c}\left(a - bg\right)$$

With a = 3733 in PQF and 3250 in QDF and b = 1071.3 in PQF and 831.9 in QDF. The next stage of the research will attempt to relate g and r_c to the various site, vehicle population, and driver population characteristics at the study sites.

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1. INTRODUCTION

This report documents work accomplished during Fiscal Year 2004-2005 as part of a research project entitled "New Approach to Bottleneck Capacity Analysis." Major activities during this period included the collection, reduction, and analysis of traffic data to document the traffic characteristics of the project's study sites and to identify relationships among the traffic characteristics across the sites. In addition, collection and analysis of data related to study site geometric characteristics, driver population characteristics, and vehicle population characteristics has begun. This report documents tentative conclusions related to the characteristics of individual sites, relationships among traffic characteristics, and relationships among traffic, site, vehicle population, and driver population characteristics.

2. BACKGROUND

2.1 Project Concept

This goal of this research is to develop and evaluate a proposed new approach to the analysis of freeway bottleneck capacity. Currently, the standard reference on highway capacity is the 2000 edition of the Highway Capacity Manual [HCM-2000 (Transportation Research Board 2000)]. Analysis methods for freeway bottlenecks are addressed in Chapters 23, 24, and 25 of HCM-2000, and are applicable to basic freeway segments, weaving sections, and ramp junctions respectively. Methods for determining capacity for basic freeway segments and ramp junctions are identical, since HCM-2000 states that the "turbulence due to merging and diverging maneuvers does not affect the capacity of the roadways involved." Those for weaving sections are somewhat different.

In all cases, the HCM methods involve the following:

1. Capacity is defined as the maximum flow that can be sustained over a 15-minute period

2. Capacity is assumed to occur at a critical density, expressed in equivalent passenger cars/lane/unit distance

3. Density is calculated by dividing the flow rate in equivalent passenger cars/hr by speed

4. Peak flow rates in equivalent passenger cars/hr are related to measured hourly volumes by formulas incorporating peak hour factors, heavy vehicle factors, and driver population factors

5. Speed is assumed to be a function of both free-flow speed and flow over the entire range of the speed-flow relationship; hence, capacity depends on free-flow speed

For basic freeway segments, the HCM method attributes differences in capacity at different sites to differences in free-flow speeds, impact of heavy vehicles, and differences in driver population characteristics. In the case of weaving sections, the type of weaving section, its length, and the ratio of weaving flow to total flow are also important. Heavy vehicle factors depend on the presence of different classes of heavy vehicles and on the length and steepness of upgrades and downgrades. Driver population factors range from 0.75 to 1.00 but are left up to the judgment of the analyst except that the factor 1.00 is used for urban commute traffic.

An alternative to the HCM approach is to note that flow is the reciprocal of the average headway, and that the headway, which is the time separation between common points on successive vehicles (for instance, front bumper to front bumper), may be decomposed into the passage time (that is the time it takes the vehicle to pass a point) and the time gap between the rear of the lead vehicle and the front of the following one. Thus the capacity of an individual lane is a function of speed and of some critical average time gap. The mathematical relationships are

$$q_i = \frac{1}{\overline{h}_i} \tag{1}$$

where q_i = flow in lane i
$\overline{h}_i$ = average headway in lane i

and

$$\overline{h}_i = \overline{g}_i + \overline{p}_i \tag{2}$$

where $\overline{g}_i$ = average time gap in lane i
$\overline{p}_i$ = average passage time in lane i

Consequently,

$$q_i = \frac{1}{\overline{g}_i + \overline{p}_i} \tag{3}$$

From this perspective, the capacity of an individual lane at a bottleneck will depend on the speed in the bottleneck (which may not vary a great deal, especially in queue discharge flow) and the average time gap, which depends on the collective behavior of the drivers. Past research by Banks (Banks 2003) indicates that although individual time gaps vary widely, and for reasons that are by no means fully understood, average time gaps in congested flow at any given site display very little variation with speed over the range of 20 – 80 km/h. On the other hand, average gaps do vary by as much as 50 percent at different sites and/or times of day. Consequently, under congested conditions the capacity of individual bottleneck lanes may be primarily the result of a relatively staple feature of the collective behavior of the drivers in a particular traffic stream.

Meanwhile, capacity is usually thought of as being flow per lane averaged across all the lanes of the bottleneck section. Consequently, the overall capacity of a bottleneck may be a function of both the capacity of some individually critical lane and the distribution of flow across all lanes. That is

$$\bar{q} = \frac{1}{r_c\left(\bar{g}_c + \bar{p}_c\right)} \tag{4}$$

where

$$r_c = \frac{q_c}{\bar{q}} \tag{5}$$

and r_c = critical lane flow ratio
q_c = critical lane flow
$\bar{q}$ = average flow per lane

In uncongested flow, lane flow distributions appear to be largely a result of a combination of driver behavior and the configuration of the facility. For instance, it has been hypothesized that individual drivers choose particular lanes because of a desire (or lack thereof) to go fast (Daganzo 2002a, 2002b). At the same time, however, the presence of entrances and exits will affect lane use in their immediate vicinity. In heavily congested flow, flow rates in individual lanes (and hence the lane flow distribution) may be controlled by conditions downstream. For instance, flow in the outside lane may be greater that of other lanes upstream of an exit because it is the sum of the flow in the lane downstream of the exit and the exiting flow, but be less than that in other lanes upstream of an entrance because of the need to absorb the traffic merging onto the freeway. Immediately upstream of a bottleneck, lane flow distributions may reflect some combination of driver behavior and downstream flow constraints, but in the bottleneck itself they should be primarily the result of local driver behavior. Past research indicates that this behavior is affected by the transition from uncongested to congested flow (Banks 1991, Ringert 1993). In addition, recent research by Amin (Amin 2003) shows that the critical lane flow ratio (with the critical lane understood as the highest volume lane) varies by as much as 40 percent at different sites in congested flow in the vicinity of bottlenecks. This suggests that critical lane flow ratios (like average time gaps) are site-specific.

The evidence of wide variation among sites in critical lane flow ratios and average critical lane time gaps and the evident dependence of these characteristics on fundamental features of driver behavior suggest an alternative approach to bottleneck capacity analysis, in which time gaps and lane volume distributions serve as intervening variables to link variations in capacity to observable characteristics of bottleneck sites, driver populations, and vehicle populations. If it proves easier to explain the lane volume distributions and average time gaps in terms of site, vehicle, and driver characteristics than to explain variations in capacity flow directly, this approach could lead to a better understanding of bottleneck capacity and thus more accurate capacity predictions. To evaluate this approach, it is necessary to quantify capacity flows, critical lane flow ratios, average critical lane time gaps, and passage times at different bottleneck sites; identify and quantify relationships among these flow characteristics across sites; identify and quantify relationships between these flow characteristics and site, driver population, and vehicle population characteristics; and finally demonstrate that the resulting relationships are better predictors of capacity than existing methods such as those in the HCM.

2.2 Work Accomplished in 2003-2004

Work accomplished in 2003-3004 included identification of study sites, identification of sources for traffic data, development of data reduction software, and the initial stages of data collection and reduction. These activities are documented in a previous interim report (Banks 2004).

2.3 Work Accomplished in 2004-2005

Work accomplished in 2004-2005 included collection, reduction, and analysis of traffic data for individual study sites, and analysis of relationships among traffic flow characteristics across study sites. In addition, collection and analysis of data related to site, driver population, and vehicle population characteristics is underway. Finally, data collected and reduced as part of the project were used to investigate the somewhat related issue of the relationship between pre-queue flow rates and the duration of pre-queue flow (VidhyaShankar 2005).

3. STUDY SITES AND DATA

3.1 Study Sites

As initially conceived, the study was to have involved at least twenty sites, which were to consist of both local sites in the San Diego area and sites in other metropolitan areas. A total of twenty-five potential study sites were eventually identified in the San Diego, Seattle, and Minneapolis-St. Paul areas. The site selection process is described in detail in the previous interim report (Banks 2004).

In the San Diego area, most loop detectors were originally installed as a part of the ramp metering system and were consequently located immediately upstream from on-ramps. In most cases, such detectors are not actually in the sections believed to be bottlenecks. Where this was the case, it was proposed to supplement the San Diego loop detector data with traffic counts from videotapes. The original assumption was that the critical time gaps would occur in the most heavily-traveled lane in the queue immediately upstream of the bottleneck and that data from the loop detectors could be used to establish these. The critical lane flow ratios, on the other hand, would need to be established for the bottleneck itself. It was proposed to use hand counts from the videotapes to determine flow ratios for a relatively small sample of time periods, relate these to the flow ratios at the detectors upstream of the bottleneck, and then use these relationships to estimate the flow ratios in the bottleneck for other time periods.

Subsequent data analysis showed that critical lane average time gaps in the queue upstream of the bottleneck did not correlate well with those in the bottleneck, and that time gaps in the bottleneck section were much more strongly related to the bottleneck flow rates than were those upstream. As a result, all but three of the San Diego study sites had to be excluded from the portions of the study that relate average critical lane time gaps and critical lane flow ratios to capacities. These sites were used, however, for portions of the study related to the variability of bottleneck capacities and will be used to verify the accuracy of any methods developed to predict capacity.

In addition, several other sites had to be excluded because of chronic data failures or evidence that they were rarely (if ever) active bottlenecks. Table 1 identifies the study sites as originally identified. Table 2 summarized their characteristics and current status. In the tables, the prefix MN indicates that the site is in the Minneapolis-St. Paul area, SD that it is in the San Diego area, and WA that it is in the Seattle area. In Table 2, "lanes' refers to the number of directional lanes in the bottleneck section. In the status column, "used" indicates that the site was used in the development of the relationships, "used for verification only" indicates that there were no detectors in the bottleneck section itself but that the site will be used in the verification of any relationships developed. In the case of the Minneapolis-St. Paul sites, data were available for two different periods, October 16 – December 1, 2000 and June 1 – August 27, 2004. The dates in parentheses indicate the period or periods used at each site; where a site was used for only one of these periods, valid data were not available for the other period.

3.2 Data and Data Sources

Data required for the project includes loop detector data, rainfall and incident log data used to screen for bad weather and incidents, geometric data for the study sites (including lane configurations and grades), vehicle classification data, and census data used to estimate driver population characteristics.

3.2.1 Loop Detector Data

Loop detector data were sought for three locations at each site: the bottleneck section itself, and detector stations immediately upstream and downstream. Data from the upstream station was used to establish traffic characteristics in the queue, particularly the critical lane average time gaps, and data from the downstream station was used to screen for deactivation of the bottlenecks as a result of the growth of queues from downstream. Where there were no detectors in the bottleneck section itself, data were collected from the detector stations immediately upstream and downstream, and the bottleneck flow was calculated by summing mainline flows and on-ramp flows. Figure 1 is a schematic diagram showing the typical ramp and detector layout.

Table 1 Study Sites

Site	Location
MN-02	NB TH-169 between Plymouth Ave and Medicine Lake Road.
MN-08	NB I-35W between Diamond Lake Rd. and 46^{th} Street
MN-14	EB I-394 between TH 100 and Penn Ave.
MN-18	WB I-94 between TH-169 and CR-61
MN-21	EB I-94 between Boone Ave and CR-81
MN-22	EB I-94 between CR-81 and CR-152
MN-23	EB I-94 between Huron Ave and TH-280
MN-25	WB I-94 between CR-152 and CR 81
MN-26	WB I-94 between Huron Ave and Riverside Ave
SD-01	SB I-15 between Via Rancho Parkway and West Bernardo Road
SD-02	NB I-805 just downstream from 47^{th} Street entrance
SD-03	SB I-5 between Manchester Avenue and Lomas Santa Fe Drive
SD-04	SB I-5 between Lomas Santa Fe Drive and Via de la Valle
SD-05	NB I-805 between University Ave and El Cajon Blvd
SD-06	NB I-5 between Via de la Valle and Lomas Santa Fe Drive
SD-07	NB I-15 between Rancho Bernardo Road and West Bernardo Road
SD-08	SB I-805 between Nobel Drive and Governor Drive
SD-09	WB I-8 between Fletcher Parkway and 70^{th} Street-Lake Murray Blvd
SD-10	SB I-15 between Murphy Canyon Road and Friars Road
SD-11	SB I-805 between Imperial Avenue and 47^{th} Street
SD-12	EB I-8 between Fairmount Avenue and College Avenue
WA-01	NB I 405, downstream from NE 85^{th} Street
WA-02	NB I-405 between NE 195^{th} St-Beardslee Blvd and SR 527
WA-03	NB I-5 between 236^{th} St SW and 220 St SW
WA-04	NB I-405 between Coal Creek Parkway and I-90

The primary types of loop detector data that were collected were volumes and occupancies. At all sites, these were available for individual lanes. The time base for these counts was 30 s for the San Diego and Minneapolis-St. Paul sites and 20 s for the Seattle sites. Average time gaps, lane flow ratios, and estimated speeds were derived from the volumes and occupancies. Average time gaps were calculated by

$$\overline{g}_i = \frac{1-\Omega_i}{q_i} \tag{6}$$

where $\overline{g}_i$ = average time gap, lane i

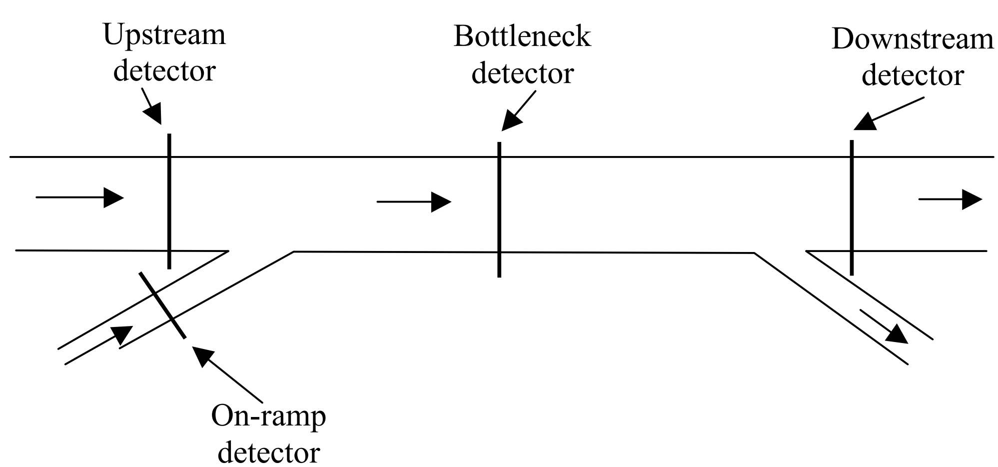
Ω_i = occupancy, lane i, dimensionless ratio

q_i = flow rate, lane i

Table 2 Study Site Characteristics and Status

Site	Peak	Lanes	Type	Status
MN-02	PM	2	merge	used (2000, 2004)
MN-08	PM	3	merge	used (2004)
MN-14	PM	3	merge	used (2000, 2004)
MN-18	PM	2	merge	used (2000)
MN-21	PM	2	merge/horizontal curve	used (2000)
MN-22	PM	2	merge	used (2000)
MN-23	PM	3	merge/3-d curve	used (2000, 2004)
MN-25	PM	2	lane drop	used (2000)
MN-26	PM	4	merge/3-d curve	not used – not active bottleneck
SD-01	AM	4	merge/grade	used for verification only
SD-02	AM	4	merge	used for verification only
SD-03	AM	4	merge/grade	used for verification only
SD-04	AM	4	merge	not used – not independent of site 3
SD-05	AM	4	grade	used
SD-06	PM	4	merge/grade	used for verification only
SD-07	PM	4	lane trapped off	used
SD-08	PM	4	grade/weave	used
SD-09	AM	4	merge	used for verification only
SD-10	PM	4	merge/weave	not used – under construction
SD-11	PM	4	merge	not used – bad data
SD-12	PM	5	merge or grade	used for verification only
WA-01	PM	3	lane trapped off	used
WA-02	PM	2	diverge	used
WA-03	PM	3	merge	used
WA-04	AM	2	weave	used

Figure 1 Typical Ramp and Detector Locations



Lane flow ratios were calculated by

$$r_i = \frac{q_i}{\overline{q}} \tag{7}$$

where r_i = flow ratio, lane i
q_i = flow rate, lane i
$\overline{q}$ = flow rate averaged across all lanes

Estimated speeds were calculated by

$$\hat{u} = \frac{\ell \sum_i q_i}{\sum_i \Omega_i} \tag{8}$$

where $\hat{u}$ = estimated speed
ℓ = average effective vehicle length, assumed to be 7.5 m

Loop detector data for the San Diego sites were obtained directly from Caltrans District 11. Initially, data for the Minneapolis-St. Paul sites were obtained directly from Lei Zhang and David Levinson of the University of Minnesota; subsequently, these data became available through internet sites maintained by the University of Minnesota at Duluth (TDRL 2005a, 2005b). Data for the Seattle sites were provided by a web site maintained by the University of Washington (University of Washington 2005).

3.2.2 Rainfall Data

Rainfall data consisted of hourly precipitation data from Minneapolis and Seattle were used to screen detector data from these areas to exclude periods with precipitation. These data were obtained from the National Climatic Data Center (National Oceanographic and Atmospheric Administration 2000, 2004a, 2004b). In the case of San Diego, periods with precipitation were identified by direct observation.

3.2.3 Incident Data

In the case of San Diego, incident log data were used to screen loop detector data for periods during which flow may have been affected by incidents. These data were generated by the California Highway Patrol Computer-Aided Dispatch System and provided by the Freeway Performance Management System (PeMS 2005). In addition, an incident detection flag is included in the Seattle data. This flag was never found to be set, however, so it does not appear that it is actually used.

3.2.4 Geometric Data

Geometric data, including lane configurations and grades, were used to classify the bottleneck sites and will be as a potential explanatory variable for average time gaps and critical lane flow ratios. These data were obtained from the applicable State Departments of Transportation, except that in the case of San Diego, lane configurations were determined by direct observation.

3.2.5 Vehicle Classification Data

Vehicle classification data were obtained from the applicable State Department of Transportation for each metropolitan area. In the case of Seattle, data were obtained directly from the Washington State Department of Transportation. Data for San Diego and Minneapolis-St. Paul were obtained from web sites (California Department of Transportation 2004, Minnesota Department of Transportation 2002). One difficulty with the vehicle classification data is that the exact vehicle classification schemes are different for each of the States concerned, and none is really compatible with that used by the HCM. In the case of Washington State, large vehicles are classified by length, and data are available by time of day on an hourly basis. In the case of California, vehicles are classified by the number of axles, and data are available only as annual average daily traffic volumes. In the case of Minnesota, data are available as average daily traffic volumes for all traffic versus heavy commercial vehicles, which are defined as those with six or more tires. In no case are recreational vehicles treated as a separate category. In the case of San Diego, data supplied by Caltrans were supplemented by hand counts taken from videotapes. The hand counts for the San Diego area were taken during the applicable peak periods and classified vehicles as (*a*) passenger cars; (*b*) vans, light trucks, and SUVs; (*c*) recreational vehicles; and (*d*) trucks.

3.2.6 Census Data

Census data are used to characterize driver populations. These data were downloaded from a web site maintained by the U. S. Census Bureau (U. S. Census Bureau 2005). The initial approach to estimating the socio-economic characteristics of the driver populations is to define a region consisting of several census tracts that are judged to be a plausible commuter-shed for each of the study sites. In the case of morning peak sites, these regions are upstream from the study site, and in the case of evening peak sites, they are downstream. For each set of census tracts, the following classification tables were downloaded: (*a*) sex by age, (*b*) sex by educational attainment for the population 25 years or older, (*c*) family type by number of workers in family, (*d*) sex by occupation for employed civilian population 16 years and older, and (*e*) household income. From these classification tables, a summary was prepared for each set of census tracts; this summary includes (*a*) percent males, (*b*) percentage distribution of age, (*c*) percentage of high school and college graduates, (*d*) percentage of households with 1, 2 and 3 or more workers, (*e*) percentage distributions of types of occupations (managerial/professional, service, office and administrative, agricultural, construction, production, and material moving), (*f*) median household income, and (*g*) median age.

3.3 Data Quality Issues

Loop detector data are subject to a variety of types of errors (Chen 1987, Jacobson 1990). These include missing data, obviously corrupt data, and less obvious count biases. Data quality had a considerable impact on site selection for this project, with a number of otherwise attractive sites rejected because of missing or obviously corrupt data. For instance, one goal in the case of the Minneapolis-St. Paul sites was to compare the performance of the bottlenecks during the period when meters were off (September-December 2000) with that during the summer of 2004. Unfortunately, this could be done for only three of the eight available sites because of data problems at the other sites during one or the other of these periods.

As a part of the data-reduction process, data sets were screened for missing or obviously corrupt data. Data screening tests were carried out for data from individual lanes and included: (*a*) missing data; (*b*) estimated speed greater than 150 km/h (excessive volume/occupancy ratio) where the vehicle count was 8 or more; (*c*) estimated average time gap too small (estimate time gap less than 0.5 s); (*d*) speed in lane inconsistent with speed averaged for all lanes; (*e*) volume and occupancy identical for two or more successive count intervals; and (*f*) in the case of San Diego, bad-data flag set by the Caltrans district detected. In all cases except volumes and occupancies identical in successive time periods, data were eliminated and replaced by a flag if they failed the data screen test. In the case of identical data in successive time periods, the data were retained but flagged. This condition was particularly prevalent for the Seattle data, where it apparently represents either some kind of detector error that is not identified by the system's data screening algorithms or an error in polling the detector cabinets (e-mail, Joel Bradbury, TDAD, 4/12/05).

In addition to such relatively obvious corrupt data, there was evidence of data biases in a number of cases. Where possible, cumulative counts taken at successive stations were compared to determine the relative biases between them. For each pair of stations compared, data were selected for five days during which the sites were uncongested at both the beginning and end of the data collection period, and the total counts for the entire data collection period were compared. Results ranged from virtually identical counts to discrepancies of up to about 3 percent. Where such counts disagreed, it is clear that data for at least one of the detector stations were biased, but it is not clear whether one or both stations were biased, which station was biased (if only one), nor what the true count should have been. Thus a major limitation of this study (and any other that relies on loop detector data) is that the apparent count biases are fairly large relative to the range of flows measured; consequently, there is uncertainty about the extent to which the apparent difference in capacity from site to site is real as opposed to being the result of biased counts.

4. DATA REDUCTION AND ANALYSIS

Reduction and analysis of data for the project as a whole involves three general stages: (*a*) reduction and analysis of traffic data for individual sites; (*b*) analysis of relationships among flow characteristics at different sites; and (*c*) analysis of relationships between flow characteristics and site, vehicle population, and driver population characteristics. The first two of these stages are essentially complete and the third is underway.

4.1 Reduction of Traffic Data

Reduction of traffic data included data extraction and collation, data screening, identification flow periods representing different flow conditions, and calculation of derived flow characteristics such as critical lane flow ratios and average time gaps.

As previously described, loop detector data for the project were obtained from sites in three different metropolitan areas. The structure and format of the data files as originally downloaded differed depending on the source. Consequently, the first step in the reduction of the data was to use custom data extraction and reduction software produced by the project to extract the data from larger files; screen the data (see Section 2.3); calculate derived measures such as estimated speeds, average time gaps, and lane flow ratios for each count interval (see Section 2.2 for formulas); and arrange the data from different sources in a common format. This software produced a single text file for each daily peak period at each study site. Spread sheets were then used for detailed analysis of the daily bottleneck data files.

4.2 Identification of Flow Periods

The first step in this analysis was to determine periods representing different flow conditions within each daily peak period at each bottleneck. Past research regarding bottleneck performance has established that the highest flows usually occur immediately prior to flow breakdown (Banks 1990, 1991; Hall 1991; Urbanik 1991; Ringert 1993, Cassidy 1999; Persaud 1998, 2001; Zhang 2004a, 2004b; Elefteriadou 2003). Past authors have proposed different definitions of capacity based on the existence of high-volume pre-queue and queue-discharge flow periods. These definitions include equating capacity with queue discharge flow (Hall 1991), defining two or more “capacities” depending on the flow period (Elefteriadou 2003), or using a weighted average of pre-queue and queue discharge flow (Zhang 2004a). The approach used in this study is to analyze pre-queue and queue discharge periods separately. In addition, cases were observed in which high-volume uncongested flows persisted for extended periods of time without flow breakdown or in which queues dissipated and then reformed. In the latter case, it was sometimes clear that demand flow had dropped and later increased to produce a second period of pre-queue flow; in other cases the flow did not appear to increase prior to the formation of the second queue. The existence of high-volume uncongested flow periods that were not obviously pre-queue flow eventually led to a fourfold classification of high-volume flow periods:

1. *Queue discharge flow (QDF)* – any period during which the bottleneck was active – that is, there was evidence of a queue upstream and no evidence of interference from queue spillbacks from downstream.

2. *Pre-queue flow (PQF)* – any period of near-constant flow preceding local flow breakdown. Note, this definition is not the same as that of Zhang (Zhang 2004a), in that it does not require the flow rate during a period of pre-queue flow to exceed the average queue discharge flow rate.

3. *Non-queue flow (NQF)* – any period of high-volume near-constant flow that did not result in flow breakdown.

4. *Inter-queue flow (IQF)* – Any period of high-volume flow between the dissipation of one queue and the formation of another for which there was not a distinct increase in flow prior to the formation of the second queue.

The beginnings and ends of periods of queue discharge flow were determined from plots of time series of estimated speeds and re-scaled cumulative speeds. Re-scaled cumulative speed was calculated as

$$S(T) = \sum_{t=1}^{t=T-1} \left[\hat{u}(t) - r \right] \tag{14}$$

where $S(T)$ = re-scaled cumulative speed prior to time T
$\hat{u}(t)$ = estimated speed for time interval t
r = a re-scale factor used to rotate the curve

The advantage of the re-scaled cumulative function is that it smoothes the data while allowing changes in average speed (indicated by changes in the slope of the plot) to be detected very precisely. Plots of time series of estimate speeds and re-scaled cumulative speeds were prepared for locations upstream of, downstream of, and in the bottleneck sections (where data for all these locations were available). Rapid decreases in speed at the upstream station indicated the beginning of the queue and subsequent rapid increases in speed accompanied by decreases in flow (or other indications of bottleneck deactivation) indicated its end. In a few cases, the upstream detectors were a considerable distance away from the point of flow breakdown; in these cases, minor decreases in speed at the bottleneck station were used to indicate the beginning of the queue, and minor increases in speed accompanied by decreases in flow to indicate its end.

For purposes of this study, pre-queue flow was defined as near-constant flow over some time period (of varying length) immediately prior to flow breakdown. This might occur prior to the initial breakdown in a given peak or prior to subsequent breakdowns in cases where queuing was intermittent. In most cases, the end of the period of pre-queue flow was taken to be the beginning of the period of queue discharge flow, as indicated by the speed time series and re-scaled cumulative speed plot. The beginning of the period of pre-queue flow was determined from plots of re-scaled cumulative flows. Re-scaled cumulative flow was calculated as

$$N(T) = \sum_{t=1}^{t=T-1} \left[q(t) - r \right] \tag{14}$$

where $N(T)$ = re-scaled cumulative flow prior to time T
$q(t)$ = flow for time interval t
r = a re-scale factor used to rotate the curve

Changes in the slope of the cumulative plot represent changes in the mean flow; the degree of rotation was chosen so as to make these more obvious while retaining the smoothing effect (Cassidy 1995). In this particular case, the object was to identify some portion of the re-scaled cumulative plot immediately prior to flow breakdown that had a near-constant slope (if any). Figures 2 and 3 show examples of the re-scaled cumulative speed and flow plots and illustrate the identification of periods of pre-queue and queue discharge flow. Note that on this occasion, the period of queue discharge flow was terminated by an apparent queue spillback from downstream that temporarily deactivated the bottleneck.

Figure 2 Re-scaled Cumulative Speed, Site WA-01, June 25, 2004

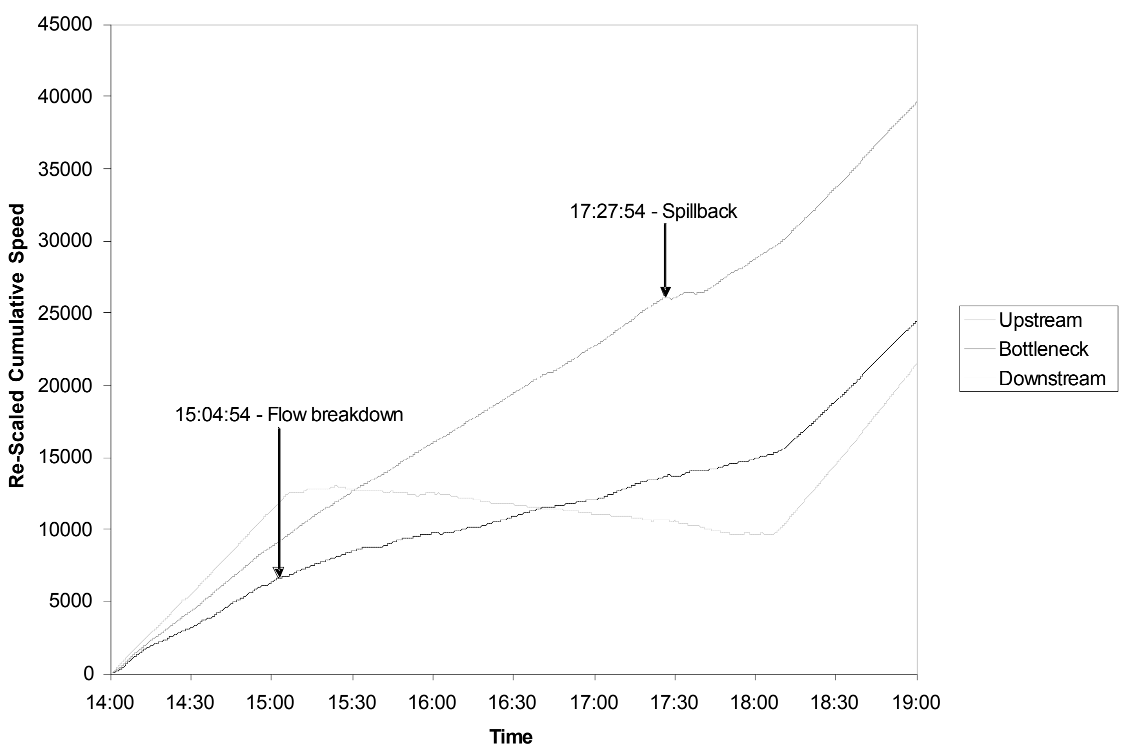
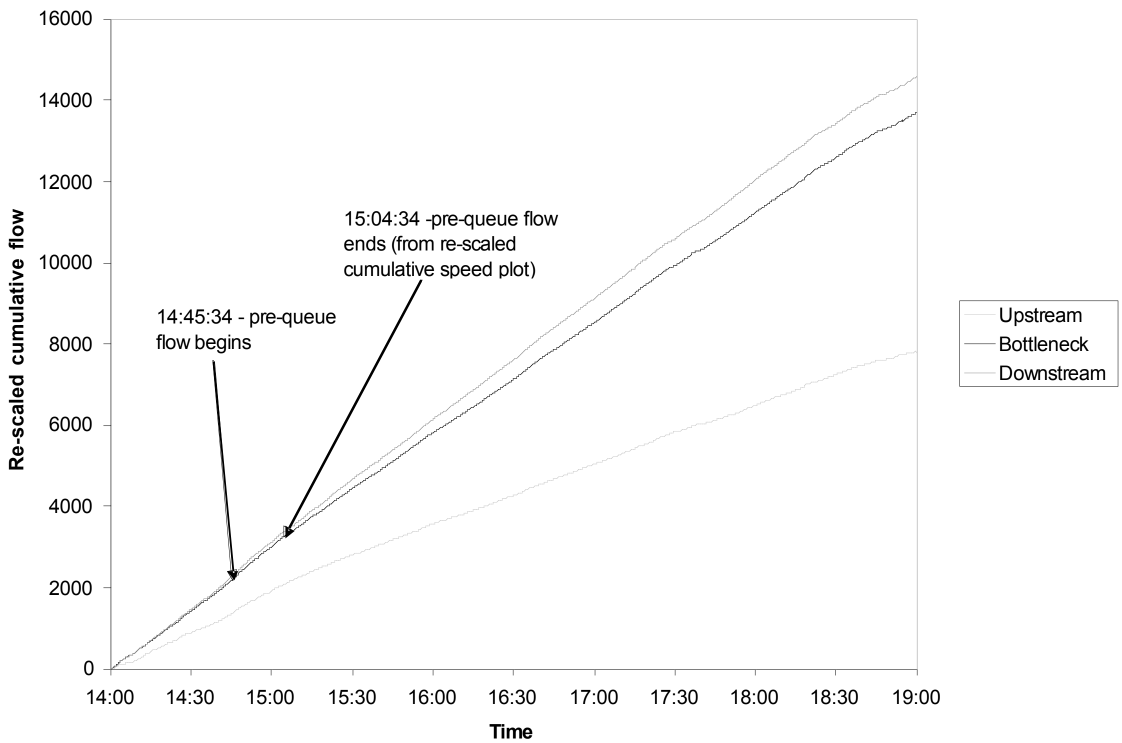


Figure 3 Re-scaled Cumulative Flow, Site WA-01, June 25, 2004



Where available, incident logs were consulted to identify periods during which bottlenecks were deactivated or bottleneck flows were apparently affected by incidents; such time periods were excluded. Also, periods of affected by rainfall were excluded. In the case of San Diego, periods of precipitation were identified by direct observation (there were none during summer 2004); for sites in the Minneapolis-St. Paul and Seattle areas, periods of precipitation were identified from hourly precipitation summaries obtained from the National Climatic Data Center of the National Oceanographic and Atmospheric Administration (NOAA). Time periods were excluded if any precipitation (including a trace) was recorded.

4.3 Data Analysis

4.3.1 Flow Characteristics at Individual Sites

Once periods of pre-queue and queue discharge flow were identified, averages and standard deviations were calculated for flow per lane through the bottleneck. Average values were also calculated for time gaps for each lane upstream of the bottleneck, and lane flow ratios and average time gaps were calculated for the bottleneck section where data were available. The lane flow ratios were used to identify the critical lane for each time period, and the critical lane flow ratio and the average time gap for the critical lane were noted. The standard deviation of the flow rate and average values for the flow rate, the critical lane flow ratio, and the critical lane average time gap were then recorded in summary files that were kept for each site.

Following the data collection period, time-weighted means, standard deviations, and coefficients of variation were calculated over all periods of pre-queue and queue discharge flow for the each of flow characteristics (flow rate, critical lane flow ratio, and critical lane average time gap).

Once average values of the flow characteristics had been determined for each site, analysis of variance was used to verify that their differences from site to site represented more than random variation; a related technique, Levene's Method (Levene 1960), was used to investigate whether there were statistically significant differences in variance of the flows at different sites. In addition, average pre-queue and queue discharge flow rates for each site were compared with one another by subtracting the average pre-queue flow from the average queue discharge flow and calculating the difference as a percentage of the average pre-queue flow. Other data analyses focusing on individual sites included analysis of the relationship between average time gaps in the critical lane at detector stations in and upstream from the bottleneck, analysis of possible variations in queue discharge flow by time of day at sites where queuing lasted for several hours, and analysis of the relationship between the duration of pre-queue flow and the pre-queue flow rate.

Analysis of the relationship between critical lane average time gaps at detector stations in and upstream from the bottleneck section was undertaken because it had been assumed initially that time gaps in the queue upstream of a bottleneck would be more strongly related to maximum queue discharge flows than those in the bottleneck itself; hence, data from sites in San Diego where there were no detectors in the bottleneck section could still be used to help establish the relationship between average time gaps and bottleneck capacity. Preliminary analysis of the relationship between critical lane average time gaps and queue discharge flow did not support this assumption, however. This led to the question of whether the average time gaps in the critical lane in the bottleneck section could be estimated from the average time gaps and critical lane flow ratio upstream. Analysis of the relationships between flow characteristics in the bottleneck section and upstream included (*a*) calculation of the differences in the average time gaps at the two locations, (*b*) calculation of the ratios of critical lane average time gaps upstream of the bottleneck to those in the bottleneck and calculation of similar ratios for critical lane flow ratios (that is, g_u/g_b and r_u/r_b, where the u and b subscripts represent data taken upstream and in the bottleneck respectively), (*c*) preparation of a scatter plot, and (*d*) calculation of the correlation coefficient between g_u/g_b and r_u/r_b to determine whether there was any relationship.

Analysis of possible variations in queue discharge flow by time of day was undertaken because the appearance of the re-scaled cumulative flow plots at several sites suggested that there might be such a relationship. This possibility was analyzed by calculating average queue discharge flow for each 30-minute interval during which queue discharge flow was normally present, plotting these against time of day, and using analysis of variance to determine whether the differences in the flow rates for different 30-minute periods were statistically significant. Where they were available, critical lane flow ratios and critical lane average time gaps for the bottleneck section were also averaged for each 30-minute interval and plotted against time of day to show the extent to which flow variations by time of day were influenced by variations in these characteristics.

Finally, the relationship between the pre-queue flow rate and the duration of queuing episodes was analyzed as part of a masters' thesis investigating the relationship between the flow rate and the probability of flow breakdown (VidhyaShankar 2005). This analysis included preparation of scatter plots and calculation of correlation coefficients for pre-queue flow rates versus duration of pre-queue flow at selected study sites.

4.3.2 Relationships among Flow Characteristics at Different Sites

The average values by site of the various flow characteristics were also analyzed to identify any relationships among them. Scatter plots were prepared and correlation coefficients were calculated to identify significant relationships. Where appropriate, regression analysis was used to quantify the relationship. Relationships were investigated for the following pairs of flow characteristics for the bottleneck sections of the study sites:

- Critical lane average time gaps and critical lane flow ratios
- Critical lane average time gaps and critical lane average passage times
- Critical lane average time gaps and critical lane average flows
- Critical lane flow ratios and average flow per lane
- Critical lane average time gaps and average flows per lane

In all cases, separate analyses were carried out for periods of pre-queue flow and periods of queue discharge flow.

4.3.3 Analysis of Site, Vehicle Population, and Driver Population Characteristics

Some preliminary analysis of site, vehicle population, and driver population characteristics and their relationship to the intervening flow characteristics has been conducted. Collection of census data and vehicle classification data is complete for all sites that are to be used in the development of predictive relationships. Collection of geometric data is also complete, with the exception of three sites in Minneapolis-St. Paul where vertical alignment data is still being sought from the Minnesota Department of Transportation. To date, analysis of these data has included:

- Determination of the amount of variation from site to site
- Calculation of heavy vehicle factors, as defined by the HCM, from vertical alignment and vehicle population data
- Calculation of correlations between
  - Median household income and critical lane average time gap
  - Males between the ages of 18 and 24 as a fraction of the population and critical lane average time gap
  - Percentage of heavy vehicles and critical lane average time gap

- Percentage of heavy vehicles and critical lane flow ratio
- Heavy vehicle factor and critical lane flow ratio

5. RESULTS

This section summarizes results of the project to date. It should be emphasized that these results are still preliminary and are subject to revision as the project continues.

5.1 Flow Characteristics at Individual Sites

5.1.1 General Characteristics

Tables 3 and 4 summarize flow characteristics, including mean flow, critical lane average time gap and critical lane flow ratio, for PQF and QDF respectively. Table 5 summarizes the standard deviation of flow and the coefficients of variation for both PQF and QDF.

Table 3 Flow Characteristics for Pre-Queue Flow

Site	Period	Mean flow	Critical lane time gap	Critical lane flow ratio	n
MN-02	2000	2153	1.28	1.10	24
	2004	1999	1.43	1.09	30
MN-08	2004	2041	1.45	1.05	38
MN-14	2000	1824	1.46	1.18	15
	2004	1686	1.58	1.18	6
MN-18	2000	2043	1.44	1.03	18
MN-21	2000	2016	1.28	1.15	4
MN-22	2000	2047	1.38	1.15	13
MN-23	2000	2173	1.18	1.13	14
	2000	2059	1.15	1.15	33
MN-25	2000	2130	1.36	1.09	21
SD-01	2004	2419			60
SD-02	2004	2416			30
SD-03	2004	2129			60
SD-05	2004	2095	1.13	1.24	46
SD-06	2004	1916			35
SD-07	2004	2108	1.34	1.13	42
SD-08	2004	2179	1.21	1.12	51
SD-09	2004	2287			10
SD-12	2004-05	2137			48
WA-01	2004	2097	1.45	1.04	40
WA-02	2004	2055	1.50	1.03	53
WA-03	2004	2120	1.44	1.07	52
WA-04	2004	2064	1.37	1.10	42

Table 4 Flow Characteristics for Queue Discharge Flow

Site	Period	Mean flow	Critical lane time gap	Critical lane flow ratio	n
MN-02	2000	2037	1.35	1.05	29
	2004	1920	1.41	1.05	52
MN-08	2004	1936	1.47	1.07	69
MN-14	2000	1745	1.42	1.20	32
	2004	1647	1.59	1.18	15
MN-18	2000	1916	1.50	1.01	24
MN-21	2000	1842	1.41	1.10	11
MN-22	2000	1884	1.43	1.08	19
MN-23	2000	2046	1.24	1.09	25
	2004	2022	1.15	1.10	48
MN-25	2000	1940	1.46	1.08	26
SD-01	2004	2175			65
SD-02	2004	2184			80
SD-03	2004	1926			65
SD-05	2004	1989	1.26	1.12	67
SD-06	2004	1818			71
SD-07	2004	2043	1.35	1.07	64
SD-08	2004	2085	1.15	1.12	72
SD-09	2004	2094			12
SD-12	2004-05	1960			61
WA-01	2004	1986	1.36	1.08	78
WA-02	2004	1983	1.48	1.01	80
WA-03	2004	1966	1.46	1.07	92
WA-04	2004	1747	1.46	1.12	63

Table 6 compares PQF and QDF for the different sites. As might be expected from past literature, average PQF exceeds average QDF at all sites. Percentage differences range from 1.8 percent at MN-23 in 2004 to 15.4 percent at WA-4.

5.1.2 Analysis of Variance

Analysis of variance was used to address two issues related to the flow characteristics at individual sites. The first of these was whether the mean flow rates in different episodes of PQF and QDF at each site were significantly different from one another – that is, are PQF and QDF homogeneous flow conditions in terms of the flow rates, or are they possibly combinations of different flow conditions? The second was whether differences in the average flow characteristics at different sites were significant – that is, do the differences in average flow characteristics among the sites represent more than the random variation in the traffic data that forms their basis? In addition, Levene's Method was used to investigate whether differences in flow variances at different sites were statistically significant.

Table 5 Standard Deviations and Coefficients of Variation of Flow

		PQF			QDF		
Site	Period	Mean	Std. dev.	C.o.v.	Mean	Std. dev.	C.o.v.
MN-02	2000	2153	83	0.04	2037	57	0.03
	2004	1999	113	0.06	1920	57	0.03
MN-08	2004	2041	82	0.04	1936	63	0.03
MN-14	2000	1824	85	0.05	1745	47	0.03
	2004	1686	62	0.04	1647	70	0.04
MN-18	2000	2043	113	0.06	1916	67	0.04
MN-21	2000	2016	106	0.05	1842	83	0.04
MN-22	2000	2047	144	0.07	1884	98	0.05
MN-23	2000	2173	91	0.04	2046	77	0.04
	2004	2059	71	0.03	2022	51	0.03
MN-25	2000	2130	114	0.05	1940	82	0.04
SD-01	2004	2419	91	0.04	2175	48	0.02
SD-02	2004	2416	94	0.04	2184	71	0.03
SD-03	2004	2129	70	0.03	1926	38	0.02
SD-05	2004	2095	62	0.03	1989	59	0.03
SD-06	2004	1916	83	0.04	1818	63	0.04
SD-07	2004	2108	83	0.04	2043	32	0.02
SD-08	2004	2179	171	0.08	2085	35	0.02
SD-09	2004	2287	36	0.02	2094	75	0.04
SD-12	2004-05	2137	73	0.03	1960	58	0.03
WA-01	2004	2097	80	0.04	1986	50	0.03
WA-02	2004	2055	88	0.04	1983	52	0.03
WA-03	2004	2120	77	0.04	1966	53	0.03
WA-04	2004	2064	87	0.04	1747	58	0.03

Differences in queue discharge flow rates during different episodes at the same site were found to be significant in all cases. Differences in pre-queue flow rates during different episodes were also significant in most cases; however, in three cases (sites MN-14 and MN-23 for 2004 data and site SD-09) the differences among the different episodes were not significant at the 0.05 level.

Table 7 summarizes results of analysis of variance tests for average flow characteristics at different sites. The table shows that in all but one case, the differences among the sites are statistically significant at the 0.05 level. The exception is critical lane time gaps in PQF. In this case the reason that the differences among sites are not statistically significant appears to be the relatively high within-group variance. That is, the relative variation in critical lane time gaps during different episodes of PQF at individual sites is considerably greater than that in QDF and is also greater than that of the other flow characteristics in either flow condition.

Table 6 Comparison of Pre-Queue Flows with Queue Discharge Flows

Site	Period	PQF	QDF	Difference QDF – PQF	Difference, pct. of PQF
MN-02	2000	2153	2037	-120	-5.6
	2004	1999	1920	-78	-3.9
MN-08	2004	2041	1936	-104	-5.1
MN-14	2000	1824	1745	-80	-4.4
	2004	1686	1647	-39	-2.3
MN-18	2000	2043	1916	-128	-6.3
MN-21	2000	2016	1842	-174	-8.7
MN-22	2000	2047	1884	-163	-8.0
MN-23	2000	2173	2046	-127	-5.8
	2004	2059	2022	-37	-1.8
MN-25	2000	2130	1940	-191	-9.0
SD-01	2004	2419	2175	-244	-10.1
SD-02	2004	2416	2184	-233	-9.6
SD-03	2004	2129	1926	-253	-11.6
SD-05	2004	2095	1989	-107	-5.1
SD-06	2004	1916	1818	-98	-5.1
SD-07	2004	2108	2043	-65	-3.1
SD-08	2004	2179	2085	-94	-4.3
SD-09	2004	2287	2094	-193	-8.4
SD-12	2004-05	2137	1960	-177	-8.3
WA-01	2004	2097	1986	-111	-5.3
WA-02	2004	2055	1983	-72	-3.5
WA-03	2004	2120	1966	-154	-7.2
WA-04	2004	2064	1747	-318	-15.4

Table 7 Summary of Results of Analysis of Variance

		Degrees of freedom		
Measure	F	p – 1	N – p	Level of significance
Pre-queue				
Flow	2.97	22	713	7.1×10^{-6}
Flow ratio	4.23	17	523	4.2×10^{-8}
Gap	1.16	17	523	0.30
Queue discharge				
Flow	5.63	22	1136	5.4×10^{-16}
Flow ratio	11.68	17	848	4.4×10^{-29}
Gap	3.90	17	848	2.0×10^{-7}

In addition, differences in the variances of flow at the different sites were found to be highly significant in both PQF and QDF.

5.1.3 Comparison of 2000 and 2004 Results at Minnesota Sites

Data were collected for two different periods at the sites in the Minneapolis-St. Paul area – October 16 – December 1, 2000 (when the ramp meters were off) and June 1 – August 27, 2004. Because of data quality problems, comparisons between these periods were possible at only three sites: MN-02, MN-14, and MN-23. Contrary to expectations, both PQF and QDF rates were less in 2004 when the ramp meters were on than in 2000 when they were off. Mean flow rates in PQF and QDF for the two periods were compared using one-tailed t-tests to determine whether the decrease was significant. The decreases in flow were statistically significant at the 0.01 level for all sites except MN-23, where the difference in average QDF for the two periods was almost but not quite significant at the 0.05 level.

5.1.4 Variations in Queue Discharge Flow by Time of Day

Inspection of re-scaled cumulative flow plots during the data reduction phase of the project suggested that there might be fairly consistent variations by time of day in the queue discharge flows at several of the sites. This was particularly the case at morning peak sites where congestion extended well beyond the normal commute trip peak. To explore the possibility that there were significant variations in queue discharge flows by time of day, five sites with especially long congested periods were selected, and queue discharge flows from different days were averaged by time of day, using 30-minute averaging intervals. Where they were available, critical lane flow ratios and critical average time gaps were also averaged over the 30-minute interval. Analysis of variance was then conducted to determine whether the 30-minute flow averages were different from one another.

Preliminary analysis indicates that there are variations in QDF with time of day where queuing episodes last long enough. At all five sites, the differences in flow for the various 30-minute intervals were statistically significant at the 0.05 level. At morning peak sites there was a tendency for QDF to peak shortly after the normal time of flow breakdown and to decline somewhat thereafter. At afternoon peak sites queue discharge rates tended to increase slightly from the time of initial flow breakdown (normally between 14:00 and 14:30) and to peak at around 17:00 or 17:30. Variation in queue discharge flow was less for the evening peak sites than the morning peak sites, with the difference between the highest and lowest 30-minute intervals being 5 percent or less. Although the general patterns were as described, there were variations in the details of the patterns from site to site.

The interrelationships among changes by time of day in flows, critical lane flow ratios, and critical average lane time gaps varied from site to site. This seems to indicate that, although queue discharge flows tend to be highest early in the morning peak and late in the afternoon peak, and thus (presumably) to correlate with the presence of commute traffic, the behavioral basis for the correlation between commute traffic and high queue discharge flows may not be simple.

5.1.5 Relationship between Pre-Queue Flow Rate and Duration of Pre-Queue Flow

The relationship between the pre-queue flow rate and the duration of individual episodes of pre-queue flow was investigated for selected sites as part of a masters' thesis examining the relationship between flow rates and the probability of flow breakdown (*9*). In general, such relationships were found to be very weak: in most cases the correlation coefficients between the average flow rate and the duration of pre-queue flow were negative (as would be expected) but not significant. It was concluded that these relationships are not an adequate basis for models of the probability of flow breakdown.

5.2 Relationships among Flow Characteristics across Sites

Average values of the different flow characteristics were compared across sites to determine whether there were relationships among them. This analysis was intended to provide the basis for relationships linking PQF and QDF to the intervening variables (time gaps, flow ratios, and passage times). Equation 4 gives an exact relationship among these variables; however, if any of them are correlated with one another, it may be possible to simplify the relationship. Also, Equation 4 by itself gives no indication of the relative extent to which flow is influenced by the different intervening variables.

Relationships were investigated for the following pairs of flow characteristics:

- Critical lane average time gaps and critical lane flow ratios
- Critical lane average time gaps and critical lane average passage times
- Critical lane average time gaps and critical lane average flows
- Critical lane flow ratios and average flow per lane
- Critical lane average time gaps and average flow per lane

Table 8 summarizes the resulting correlation coefficients. Critical lane average time gaps and critical lane flow ratios are not significantly correlated with one another in either PQF or QDF. On the other hand, there is a significant negative correlation between critical lane average passage times and critical lane average time gaps at the 0.01 level in both PQF and QDF. There is also a significant negative correlation between critical lane average time gaps and average flow per lane at the 0.01 level for both PQF and QDF. On the other hand, average flow per lane is not significantly correlated with the critical lane flow ratio in PQF, although is there is a significant negative correlation between these variables in QDF at the 0.05 level but not the 0.01 level. Finally, there is a very strong negative correlation between critical lane average time gaps and critical lane flow in both PQF and QDF and, when plotted, this relationship appears to be virtually linear. This linearity, which is contrary to what might be expected from the form of Equation 4, is presumably the result of the correlation between the time gaps and the passage times.

Table 8 Summary of Correlation Analysis for Average Flow Characteristics

Relationship	Correlation coef.	Deg. of freedom	Correlation significant? 0.01 level	Correlation significant? 0.05 level
Pre-queue flow				
CLFR vs. gap	-0.438	16	No	No
Passage time vs. gap	-0.591	16	Yes	Yes
Flow/lane vs. gap	-0.599	16	Yes	Yes
Flow/lane vs. CLFR	-0.338	16	No	No
CL flow vs. gap	-0.909	16	Yes	Yes
Queue discharge flow				
CLFR vs. gap	-0.056	16	No	No
Passage time vs. gap	-0.711	16	Yes	Yes
Flow/lane vs. gap	-0.696	16	Yes	Yes
Flow/lane vs. CLFR	-0.584	16	No	Yes
CL flow vs. gap	-0.905	16	Yes	No

Figures 4 and 5 are scatter plots showing the relationship between critical lane average time gaps and critical lane flows for PQF and QDF respectively. Lines of best fit determined by least-squares regression are superimposed on the plots. The regression equation for PQF is

$$q = 3733 - 1071.3g$$

where q = critical lane flow in veh/h
g = critical lane average time gap, s

For QDF, the regression equation is

$$q = 3250 - 831.9g$$

6. CONCLUSIONS

This section presents tentative conclusions based on the results of the project to date. Like the preliminary results, they are subject to revision as the research continues. These conclusions are related to the variations in flow characteristics among the study sites, interrelationships among the average values of the proposed intervening variables at different sites, and the most promising forms for models linking the intervening variables to bottleneck capacity flows. They address the following questions:

Figure 4 Relationship between Critical Lane Average Time Gaps and Critical Lane Flow for Pre-Queue Flow

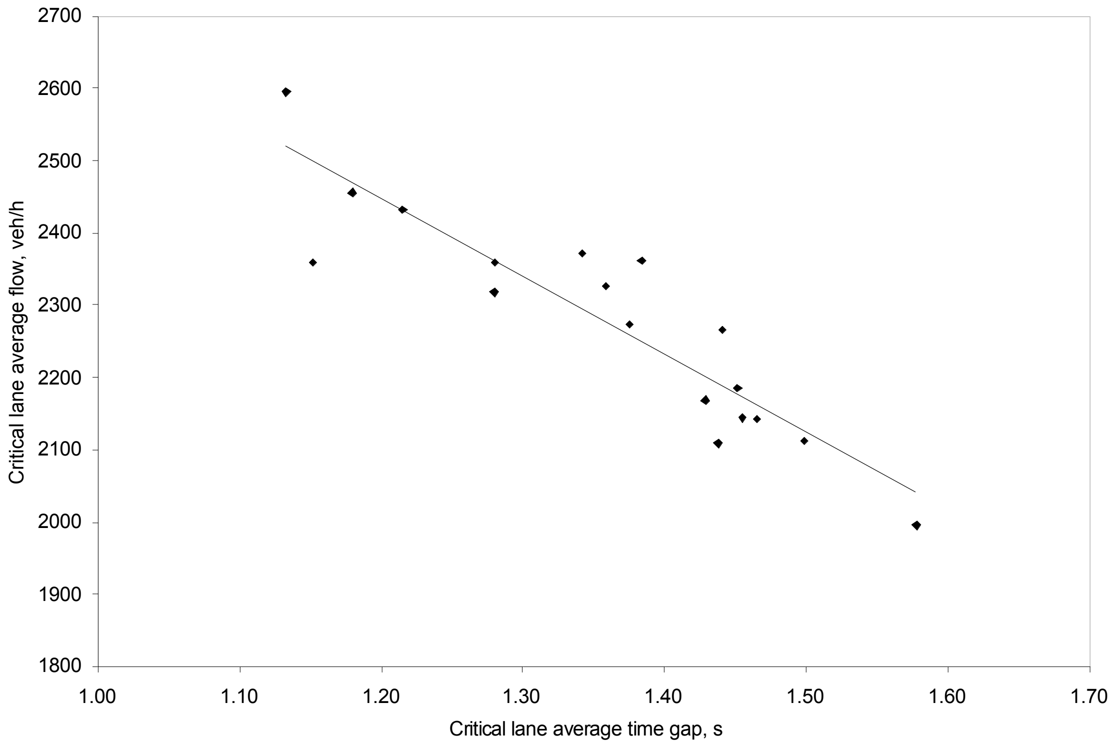
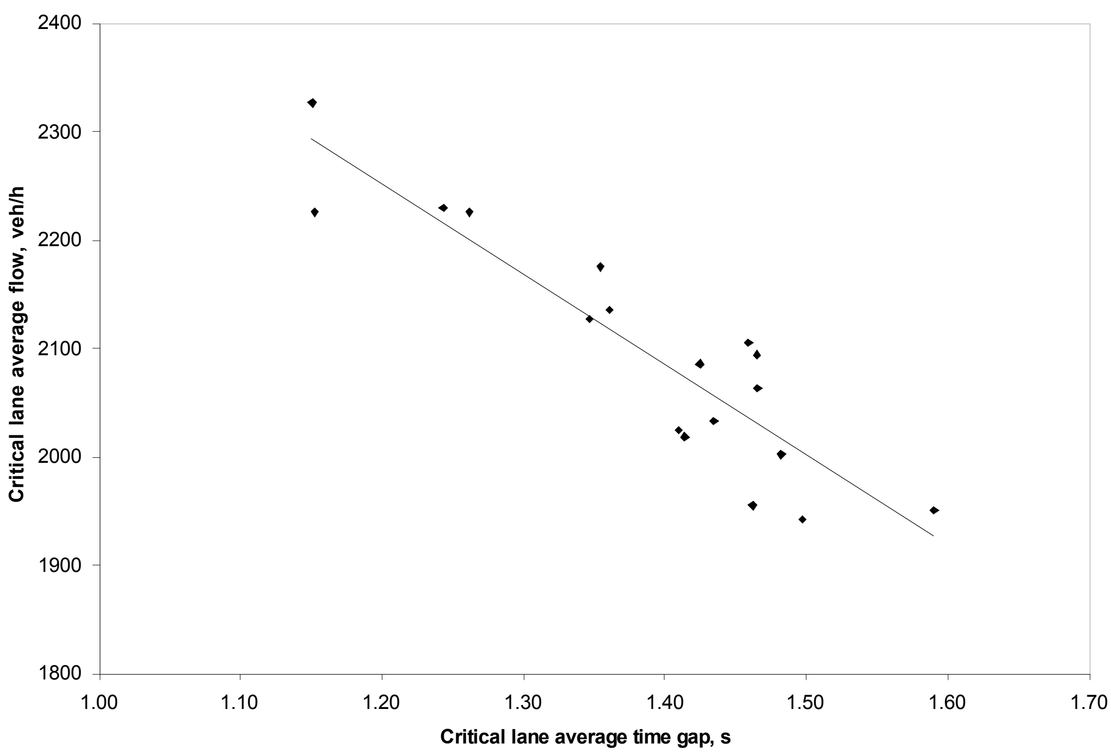


Figure 5 Relationship between Critical Lane Average Time Gaps and Critical Lane Flow for Queue Discharge Flow



a. Are PQF and QDF distinct flow conditions? Should capacity be modeled in terms of one or the other of these flow conditions, or both?

b. Are there significant variations in the flow characteristics from site to site?

c. Are there interrelationships among the intervening variables that could lead to simplification of predictive models for bottleneck capacity flow?

d. Which of the intervening variables are most strongly related to PQF and QDF?

Are PQF and QDF distinct flow conditions? Should capacity be modeled in terms of one or the other of these flow conditions, or both? It appears that PQF and QDF are distinct flow conditions, but they are not necessarily homogeneous. That is, what has been identified as PQF or QDF may actually represent several different flow conditions that are present at different times. The main evidence that they are distinct flow conditions is that average pre-queue flows are greater than average queue discharge flows at all sites. This consistency in the relationship between flows before and during queuing is in agreement with past research and would seem to indicate that PQF and QDF are indeed distinct conditions. Consequently, this research will continue to model PQF and QDF as different flow conditions, each of which can be taken to represent the "capacity" of a bottleneck in some sense.

Evidence that PQF and QDF may not be homogeneous conditions includes the following:

- In almost all cases, analysis of variance indicated that there were significant differences in average flows in different episodes of PQF and QDF at individual sites.

- There appeared to be significant time-of-day trends in QDF where congestion episodes lasted long enough. The nature of these trends suggests that queue discharge rates may vary with the composition of the driver population, with the highest queue discharge rates occurring at times when the proportion of commuters in the traffic stream was presumably at its highest.

- There were significant differences in average flow rates in both PQF and QDF at the same sites for widely separated time periods (2000 and 2004). Note that Zhang and Levinson (Zhang 2004b) had previously found differences in average flows at these sites when they compared data from 1999 with that from 2000. They found a decrease in flow between 1999 and 2000 and attributed it to the fact that ramp meters were in operation in 1999 but had been turned off on an experimental basis in 2000. The findings here are that there was a further decrease between 2000 and 2004, when the meters were back on, but with a different metering control algorithm. It is not clear whether the differences in average flow for these different time periods were really related to the presence of metering or not, but it is clear that the values of PQF and QDF at these sites were not the same during the different periods.

Are there significant variations in the flow characteristics from site to site? It appears that there are indeed significant variations in average flow characteristics from site to site, and that these variations are of practical significance as well as statistical significance. Analysis of variance showed that there were statistically significant differences in average flow characteristics at the different sites in all cases except critical lane average time gaps in PQF. In this case, there is considerable reason to believe that the average time gaps actually are different at the different sites, and that the result of the statistical test was due to relatively large variations in the time gaps during different episodes of PQF and relatively small sample sizes (when compared, for instance, with time gaps in QDF). Evidence that the time gaps in PQF really do differ by site includes the fact that they are strongly correlated with both flows in PQF and time gaps at the same site in QDF, both of which vary significantly by site.

The practical significance of the differences in the average flow characteristics at the different sites is indicated by the fact that the difference in the average flow per lane between the site with the highest flow and that with the lowest flow was about 35 percent of the average flow per lane for PQF and 27 percent for QDF. For critical lane flow ratios, the difference between the highest and lowest site was about 19 percent in PQF and 17 percent in QDF; for critical lane average time gaps, it was 33 percent in PQF and 32 percent in QDF.

Are there interrelationships among the intervening variables that could lead to simplification of predictive models for bottleneck capacity flow? Critical lane average passage times and critical lane average time gaps are significantly correlated (at the 0.01 level) in both PQF and QDF. This finding, coupled with the fact that the time gaps are considerably larger than the passage times for the conditions prevailing in the bottleneck sections, suggests that passage times may be omitted from the proposed predictive models.

Which of the intervening variables are most strongly related to PQF and QDF? Results of the correlation analysis indicate that critical lane average time gaps are more strongly correlated with flow per lane than are critical lane flow ratios, especially in PQF. Critical lane flows and critical lane average time gaps are highly correlated for both PQF and QDF, and in both cases the relationship appears to be virtually linear.

On the basis of the relationships among the intervening variables and between the intervening variables and flow, the most promising model for relating the intervening variables to flow per lane appears to be

$$q = \frac{1}{r_c}(a - bg) \tag{9}$$

With a = 3733 in PQF and 3250 in QDF and b = 1071.3 in PQF and 831.9 in QDF. Research during 2005-2006 will concentrate on modeling r and g as functions of the various site, vehicle population, and driver population characteristics and on verifying the resulting models for PQF and QDF.

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