DEFINING, MEASURING AND ESTIMATING FREEWAY CAPACITY

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INTRODUCTION

The term "capacity" has been used to quantify the traffic-carrying ability of transportation facilities, and its definition and numerical value have evolved over time. In current practice the terms “capacity”, “breakdown” and “congestion occurrence” are very often used, but they are not well defined and quantified. The Highway Capacity Manual (HCM 2000) is the publication most often used to estimate capacity. The current published version of the HCM 2000 defines the capacity of a facility as "...the maximum hourly rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period, under prevailing roadway, traffic and control conditions (HCM, p. 2-2)." Specifically for freeway facilities, capacity values are given as 2,250 passenger cars per hour per lane (pcphpl) for freeways with free-flow speeds of 55 mph, up to 2,400 pcphpl when the free-flow speed is 75 mph (ideal geometric and traffic conditions).

For a long time, researchers have recognized the inadequacy and impracticality of this definition for freeway facilities. Firstly, implied in the current definition and understanding of freeway capacity is the notion that the facility will become congested and "break down" (i.e., transition from a non-congested state to a congested state) when demand exceeds the specified capacity value. Elefteriadou et al (1995) showed however that breakdown does not necessarily occur always at the same demand levels, but can occur when flows are lower or higher than the numerical value traditionally accepted as capacity. In a recent paper, Lorenz and Elefteriadou (2001) conducted an extensive analysis of speed and flow data collected at two freeway-bottleneck-locations in Toronto, Canada, to investigate how the probabilistic model previously developed by Elefteriadou et al (1995) compared to field data. At each of the two sites, the freeway breakdown process was examined in detail for over 40 breakdown events occurring during the course of nearly 20 days. Based on field observations, the authors defined breakdown to occur: “...when the average speed of all lanes on the freeway dropped below 90 km/h (56 mph) for a period of at least five minutes (15 consecutive 20-second intervals).” The authors defined also the breakdown flow rate as: “the flow rate (expressed as a per-lane, equivalent hourly rate) that occurs during the one-minute time period immediately prior to the breakdown event.” Next, the authors recorded the number of breakdown events that occurred at various demand levels. As expected, the data indicated that the probability of breakdown increases with increasing flow rate. The authors confirmed that the existing freeway capacity definition does not capture the stochastic nature of congestion occurrence. The analysis however did not examine data prior to, or after the breakdown events, but focused on the occurrence of breakdowns, and the numerical value of breakdown flows.

Jia et al (2001) also found that capacity predictions based on physical capacity as suggested in the HCM can be misleading. The authors defined the “revealed” capacity of a link to be the maximum sustained flow in vehicles per hour that is reached at that link, and they found that these revealed capacities observed at different links are different and varying from day to day, even though the geometric configurations of each link are somewhat similar.
Regarding pre-breakdown (or pre-queue) and discharge flows, Banks (1990) collected data at a California bottleneck site and provided evidence to support the hypothesis that maximum flows drop after queues form (i.e., the two-capacity phenomenon). The paper did not examine quantitatively the transition to congestion (speed drop was the criterion, but no value for it was given), nor did it provide quantitative definitions for congested conditions.

The implications of the findings reported above are:

- The maximum sustained flow at a given freeway segment varies, and does not necessarily occur in conjunction with breakdown.
- The flow at the time of breakdown varies at a given freeway site.
- Congestion occurrence (i.e. the transition from non-congested to congested state) is not well defined and quantified in the HCM, thus analysts either use qualitative criteria, or develop their own definitions, which does not allow for consistency and continuity in research and practice.
- Given the variability reported in the values of maximum pre-breakdown flow, and in the values of breakdown flow, maximum discharge flows should be similarly examined to determine what their variability is, and whether the two-capacity phenomenon exists. Maximum discharge flow is defined in this paper as the maximum flow that occurs after the breakdown, i.e., during congested conditions.

The primary objective of this paper is to examine freeway traffic data at two sites over a period of several days, including transitions from non-congested to congested state, and recommend appropriate definitions for the terms “capacity”, “breakdown”, and “congestion occurrence”. These definitions should be clear and quantitative in nature, provide adequate information for their field data collection, and should be easy to identify and obtain using simulation modeling.

The next section contains a brief (not exhaustive) literature overview on some capacity definitions, while the third section presents the selected study sites, and data collection information. The fourth section presents summaries of the field data and analyses conducted, and the last section presents conclusions and recommendations.

LITERATURE OVERVIEW

This section of the paper reviews selected previous research that has considered the issues of capacity and breakdown. Persaud and Hurdle (1991) discussed various definitions and measurement issues for capacity, including maximum flow definitions, mean flow definitions, and expected maximum flow definitions. They collected data at a three-lane freeway site, over three days. In concluding, they recommended that the mean queue discharge flow is the most appropriate, partly due to the consistency the researchers observed in its day-to-day measurement.
Agyemang-Duah and Hall (1991) collected data over 52 days on peak periods to investigate the possibility of a drop in capacity as a queue forms, and to recommend a numerical value for capacity. They plotted pre-queue peak flows and queue discharge flows in 15-minute intervals, which showed that the two distributions are similar, with the first one slightly more skewed toward higher flows. They recommended 2,300 pcphpl as the capacity under stable flow, and 2,200 pcphpl for post-breakdown conditions, which corresponded to the mean value of the 15-minute maximum flows observed under the two conditions. The researchers recognized the difficulty in defining and measuring capacity, given the variability observed. Wemple et al (1991) also collected near-capacity data at a freeway site and discussed various aspects of traffic flow characteristics. High flows (above 2,000 vphpl) were identified, plotted, and fitted to a normal distribution, with a mean of 2315 vph, and a standard deviation of 66 vph.

Elefteriadou et al (1995) developed a model for describing the process of breakdown at ramp-freeway junctions. Observation of field data showed that, at ramp merge junctions, breakdown may occur at flows lower than the maximum observed, or capacity flows. Furthermore, it was observed that, at the same site and for the same ramp and freeway flows, breakdown may or may not occur. The authors developed a probabilistic model for describing the process of breakdown at ramp-freeway junctions, which gives the probability that breakdown will occur at given ramp and freeway flows, and is based on ramp-vehicle cluster occurrence. Similarly to this research, Evans et al (2001) also developed a model for predicting the probability of breakdown at ramp freeway junctions, which was based on Markov chains, and considered operations on the entire freeway cross-section, rather than the merge influence area.

Minderhoud et al (1997) discuss and compare empirical capacity estimation methods for uninterrupted flow facilities, and recommend the product limit method because of its sound theoretical framework. In this method, non-congested flow data are used to estimate the capacity distribution. The product-limit estimation method is based on the idea that each non-congested flow observation having a higher flow rate than the lowest observed capacity flow rate contributes to the capacity estimate, since this observation gives additional information about the location of the capacity value. The paper does not discuss transitions to congested flow, nor discharge flow measurements.

Lorenz and Elefteriadou (2001) conducted an extensive analysis of speed and flow data collected at two freeway-bottleneck-locations in Toronto, Canada, to investigate whether the probabilistic models previously developed replicated reality. At each of the two sites, the freeway breakdown process was examined in detail for over 40 breakdown events occurring during the course of nearly 20 days. Examining the time-series speed plots for these two sites, the authors concluded that a speed “boundary” or “threshold” at approximately 90 km/h existed between the non-congested and congested regions. When the freeway operated in a non-congested state, average speeds across all lanes generally remained above the 90 km/h threshold at all times. Conversely, during congested conditions, average speeds rarely exceeded 90 km/h, and even then they were not maintained for any substantial length of time. This 90 km/h threshold was observed to exist at both study sites and in all of the daily data samples evaluated as part of that
research. Therefore, the 90 km/h threshold was applied in the definition of breakdown for these sites. Since the traffic stream was observed to recover from small disturbances in most cases, only those disturbances that caused the average speed over all lanes to drop below 90 km/h for a period of five minutes or more (15 consecutive 20-second intervals) were considered a true breakdown. The same criterion was used for “recovery periods”. The authors recorded the frequency of breakdown events at various demand levels. As expected, the probability of breakdown increases with increasing flow rate. Breakdown however, may occur at a wide range of demands (i.e., 1,500 – 2,300 vphpl). The authors confirmed that the existing freeway capacity definition does not accurately address the transition from stable to unstable flow, nor the traffic carrying ability of freeways under various conditions. Freeway capacity may be more adequately described by incorporating a probability of breakdown component in the definition. A suggested definition reads: “...the rate of flow (expressed in pcp/hpl and specified for a particular time interval) along a uniform freeway segment corresponding to the expected probability of breakdown deemed acceptable under prevailing traffic and roadway conditions in a specified direction.” The value of the probability component should correspond to the maximum breakdown risk deemed acceptable for a particular time period. A target value for the acceptable probability of breakdown (or "acceptable breakdown risk") for a freeway might initially be selected by the facility’s design team, and later revised by the operating agency or jurisdiction based on actual operating characteristics. With respect to the two-capacity phenomenon, the researchers observed that the magnitude of any flow drop following breakdown may be contingent upon the particular flow rate at which the facility breaks down. Flow rates may remain constant or even increase, following breakdown. This may explain the fact that some researchers have observed the two-capacity phenomenon and others haven’t: it seems to depend on the specific combination of the breakdown flow and the queue discharge flow for the particular observation period. The paper does not discuss maximum pre-breakdown flow however, nor does it directly compare breakdown flows to maximum discharge flows for each observation day.

Zhang (2001) proposed that there can be three kinds of capacity for any location: one for acceleration flow, one for deceleration flow, and one for stationary flow. He proposes that the stationary flow capacity should be adopted as the ideal capacity of a roadway. Theoretical results are supported by field data at a single freeway location, taken over a period of 100 minutes. The paper does not define quantitatively breakdown nor discharge flow issues.

In summary, several studies have shown that there is variability in the maximum sustained flows observed, and that the pre-breakdown maximum flow may be higher than the maximum discharge flow. The breakdown flows have also been shown to vary at a given site. In previous studies however, either the number of observations is limited, or the scope of the analysis does not include maximum pre-breakdown, breakdown and maximum discharge flows, to address capacity definition issues in a comprehensive manner. This study examines traffic operations at two freeway bottleneck sites including 40 breakdown events for each site, to develop capacity definitions considering maximum pre-breakdown flow, breakdown flow and maximum discharge flow.
STUDY SITES AND FIELD DATA COLLECTION

The two sites along the Highway 401 freeway system in Toronto, Canada studied in Lorenz and Elefteriadou (2001), which regularly experience breakdown, are used in this study as well. The two sites are denoted as Site “A” and Site “B.” Highway 401 is instrumented with several detector stations which collect and archive detailed speed and volume data. The data were obtained through the Ministry of Transportation of Ontario and McMaster University. The two sites selected are both bottleneck locations instrumented with detector stations within the bottleneck.

The posted speed limit on the 401 freeway is 100 km/h (approximately 62 mph). Free flow speeds during off-peak time periods were found to range between 100 km/h and 120 km/h (approximately 62 mph and 75 mph). Traffic volumes on the 401 freeway system are generally heavy during weekday morning and afternoon peak periods, and breakdowns in traffic flow are typical during these peak times. It is not uncommon for congested conditions to persist for several hours.

Figure 1 presents a sketch of Site “A,” located in the vicinity of the Highway 427/Highway 401 interchange. At this site, traffic traveling northbound-to-eastbound in three ramp lanes on 427 merges with express traffic traveling in three eastbound mainline lanes on 401. A detector station (denoted as “Station 1”) is located approximately 550-meters downstream of the ramp gore point. Figure 2 illustrates the lane geometry at Site “B,” located in the vicinity of the Highway 400-Black Creek Drive/Highway 401 interchange. One detector station (denoted as “Station 2”) is located approximately 1,250-meters downstream of the ramp gore point. At both stations, paired detectors in each of the travel lanes provide vehicle counts and speed estimates continuously at 20-second intervals. Breakdowns in traffic flow at both Site “A” and Site “B” are common during weekday peak periods.

Archived data for 17 weekdays were obtained from Site “A” and for 20 weekdays from Site “B.” The sampling periods ranged from eight to 24 hours per day. The data were closely examined for erroneous detector readings and summarized using a spreadsheet. In addition, the speed and flow rate data were cross-checked with data at adjacent detector stations to ensure that all subsequent analyses reflected site-imposed capacity constraints, as opposed to merely queue spillback resulting from downstream congestion. The driver population in the vicinity during the study intervals typically consists of urban commuters and other drivers familiar with the area’s transportation system. The percentage of heavy vehicles in the traffic stream in the area of the study sites is approximately 10 percent, according to the Ministry of Transportation of Ontario.
Figure 1: Schematic of Site "A"

SITE “A”

Highway 401
“express” lanes

Highway 427
ramp lanes

Paired detectors (Station 1)

Note: Drawing not to scale

Figure 2: Schematic of Site "B"

SITE “B”

Paired detectors (Station 2)

Highway 401
“collector” lanes

Highway 401
“express” lanes

Note: Drawing not to scale
DATA ANALYSES

Data analysis began with a detailed examination of data collected before, during and after transitions from non-congested to congested flow at the two freeway bottleneck sites described in the previous section. For each day of data obtained, time series plots of flow and speed were generated to examine and quantify changes in these variables. The following analyses were undertaken:

1. Identify and quantify each transition interval from non-congested to congested flow, i.e., breakdown event, and document the corresponding breakdown flow. In a previous study with data from Queen Elizabeth Freeway in Toronto, Canada, Lorenz and Elefteriadou (2001) identified and defined breakdown as a speed drop below 90 km/hr, with duration of at least 15 minutes. In this paper, the same definition of breakdown is used, since the same two sites are studied. (Another study on weaving areas (Lertworawanich and Elefteriadou, 2001) showed that the breakdown speed threshold exists at these sites as well, but has a different value (80 km/h). Weaving areas have typically lower speeds and this may be the reason that the speed threshold at breakdown is lower. Thus, the definition of breakdown should ultimately be either segment type-specific, or be generalized using other free-flow speed information, or may be a function of some other factor.) In this research, the breakdown flow is defined as the 5-min. flow (for 5-min aggregation intervals) or 15-minute flow (for 15-min. aggregation intervals) immediately prior to the breakdown.

2. Identify and document the maximum pre-breakdown flow. This flow is the maximum observed at the site prior to the occurrence of congestion. As for breakdown flow analysis, both 5-minute and 15-minute aggregation intervals were examined.

3. Identify and document the maximum discharge flow. This flow is the maximum observed at the site after the occurrence of breakdown, and prior to recovery to non-congested conditions.

Figure 3 illustrates these three variables in a single time-series plot (corresponding to a single breakdown event) of flows and speeds. As shown, these flows differ in magnitude. It has been assumed in the past that the first two values are the same, i.e., occur at the same time, which is not the case.
of these three flow rates (the maximum pre-breakdown, the breakdown flow, and the maximum discharge flow) were obtained for each breakdown event. These flows were obtained for 5-min and for 15-min analysis intervals. Next, frequency diagrams were developed for each site and for each of the three flow parameters. Figure 4 includes the histograms of the three parameters for Site “A”, while Figure 5 includes the histograms for Site “B”, both aggregated in 5-min intervals. Similar histograms were developed for 15-min aggregation intervals but are not shown here because of space constraints. Table 1 presents the minimum and maximum values, as well as the mean and standard deviation for each flow parameter for both 5-min and 15-min aggregation intervals, at the two sites.
FIGURE 4: Observed Flow Rates with 5-Minute Aggregation - Site “A”

FIGURE 5: Observed Flow Rates with 5-Minute Aggregation at Site “B”
TABLE 1 Descriptive Statistics of Observed Flow Rates at Sites A and B

<table>
<thead>
<tr>
<th></th>
<th>SITE “A”</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max Pre-Breakdown Flow (veh/h/l)</td>
<td>Breakdown Flow Rate (veh/h/l)</td>
<td>Maximum -Discharge Flow (veh/h/l)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5-min Aggregation</td>
<td>15-min Aggregation</td>
<td>5-min Aggregation</td>
<td>15-min Aggregation</td>
<td>5-min Aggregation</td>
<td>15-min Aggregation</td>
</tr>
<tr>
<td>Number of Observations</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>35</td>
</tr>
<tr>
<td>Min Value</td>
<td>1116</td>
<td>959</td>
<td>1099</td>
<td>958</td>
<td>1411</td>
<td>1431</td>
</tr>
<tr>
<td>Max Value</td>
<td>2014</td>
<td>1930</td>
<td>1915</td>
<td>1842</td>
<td>1975</td>
<td>1860</td>
</tr>
<tr>
<td>Mean</td>
<td>1763</td>
<td>1650</td>
<td>1657</td>
<td>1596</td>
<td>1791</td>
<td>1722</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>157</td>
<td>168</td>
<td>194</td>
<td>173</td>
<td>111</td>
<td>96</td>
</tr>
</tbody>
</table>

|                | SITE “B” |                  |                  |                  |                  |                  |
|                | Max Pre-Breakdown Flow (veh/h/l) | Breakdown Flow Rate (veh/h/l) | Maximum Discharge Flow (veh/h/l) |
|                | 5-min Aggregation | 15-min Aggregation | 5-min Aggregation | 15-min Aggregation | 5-min Aggregation | 15-min Aggregation |
| Number of Observations | 39 | 39 | 39 | 39 | 39 | 37 |
| Min Value      | 1772 | 1739 | 1524 | 1004 | 1680 | 1457 |
| Max Value      | 2456 | 2351 | 2356 | 2351 | 2368 | 2241 |
| Mean           | 2239 | 2108 | 2082 | 2025 | 2128 | 2049 |
| Std. Dev.      | 132  | 129  | 210  | 234  | 147  | 140  |
As shown, the range of values for each of these flow parameters is several hundred veh/h/l. The range of maximum discharge flows at site “A” is relatively narrower. For site “B” however the ranges of the three parameters are similar. At both sites the highest flows were observed under non-congested conditions: The cell 2000-2100 vphpl for site “A” and the cell 2400-2500 vphpl for site “B” contains data for maximum pre-breakdown flows only. In addition, for both sites the lowest flows reported were breakdown flows.

Statistical analyses were conducted to evaluate the distributions obtained, and to compare their parameters. Chi-square tests and Kolmogorov-Smyrnov (K-S) tests were conducted to compare these distributions to the normal distribution. It was concluded that all distributions fit the normal at the 5% level of significance. The t-test was used to compare the mean values of each of the flow rates. The results of the tests are as follows:

For site “A” and at the 5% level of significance:

5 – min aggregation: Max. Discharge Flow = Max. Pre-Breakdown > Breakdown Flow
15 – min aggregation: Max. Discharge Flow > Max. Pre-Breakdown = Breakdown Flow

For site “B” and at the 5% level of significance:

15 – min aggregation: Max. Pre-Breakdown = Max. Discharge Flow = Breakdown Flow

For both sites the breakdown flow is the lowest of the three. Comparisons of the maximum pre-breakdown and maximum discharge values give conflicting results between the two sites. A possible explanation for this difference between the two sites may be that geometric characteristics and sight distance result in different operations under high- and low-speed conditions. As expected, the larger aggregation interval tends to alleviate any differences among the three flow parameters, and the statistical comparisons for these result in equalities in most of the cases examined.

Next, the differences of the flow values for each breakdown event (typically one or two events per day, during the peak periods) to determine whether there is a relationship between the maximum pre-breakdown, breakdown and maximum discharge flows for each event. Tables 2 and 3 provide these values for each aggregation interval, for sites “A” and “B” respectively. The tables also provide 95% confidence intervals on the mean for each average difference. Thus, for example, the average difference between max. pre-breakdown flow and breakdown flow for site “A” at the 5-min. level of aggregation is 106 vphpl, while the 95% confidence interval on this value is between 68 and 144 vphpl. The paired t-test was used to compare the average differences between flow parameters to zero.
### TABLE 2: Comparison of Observed Flow Rates at Site A

<table>
<thead>
<tr>
<th>Parameter</th>
<th>5-Min Aggregation</th>
<th>15-Min Aggregation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Difference Between:</td>
<td>Difference Between:</td>
</tr>
<tr>
<td></td>
<td>(Max Pre-Breakdown)-(Breakdown)</td>
<td>(Max Pre-Breakdown) - (Max. Discharge)</td>
</tr>
<tr>
<td>Number of Observations</td>
<td>41</td>
<td>41</td>
</tr>
<tr>
<td>Min Value</td>
<td>-79</td>
<td>-677</td>
</tr>
<tr>
<td>Max Value</td>
<td>581</td>
<td>238</td>
</tr>
<tr>
<td>Mean</td>
<td>106</td>
<td>-28</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>125</td>
<td>155</td>
</tr>
<tr>
<td>95 % Confidence Interval</td>
<td>[68,144]</td>
<td>[-75,19]</td>
</tr>
</tbody>
</table>

| Number of Observations | 35 | 35 | 35 |
| Min Value | 0 | -738 | -740 |
| Max Value | 383 | 274 | 101 |
| Mean | 53 | -78 | -131 |
| Std. Dev. | 84 | 182 | 159 |
| 95 % Confidence Interval | [25,81] | [-138,-18] | [-184,-78] |
TABLE 3: Comparison of Observed Flow Rates at Site B

<table>
<thead>
<tr>
<th>Parameter</th>
<th>5-Min Aggregation</th>
<th></th>
<th></th>
<th>15-Min Aggregation</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Difference Between:</td>
<td>Difference Between:</td>
<td>Difference Between:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Max Pre-Breakdown)-(Breakdown)</td>
<td>(Max Pre-Breakdown) - (Discharge)</td>
<td>(Breakdown) - (Max. Discharge)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Observations</td>
<td>39</td>
<td>39</td>
<td>39</td>
<td>37</td>
<td>37</td>
<td>37</td>
</tr>
<tr>
<td>Min Value</td>
<td>-20</td>
<td>-156</td>
<td>-596</td>
<td>-15</td>
<td>-212</td>
<td>-1076</td>
</tr>
<tr>
<td>Max Value</td>
<td>804</td>
<td>488</td>
<td>260</td>
<td>1164</td>
<td>733</td>
<td>294</td>
</tr>
<tr>
<td>Mean</td>
<td>156</td>
<td>110</td>
<td>-46</td>
<td>78</td>
<td>73</td>
<td>-5</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>183</td>
<td>151</td>
<td>164</td>
<td>213</td>
<td>168</td>
<td>221</td>
</tr>
<tr>
<td>95 % Confidence Interval</td>
<td>[98,214]</td>
<td>[62,158]</td>
<td>[-97,5]</td>
<td>[9,147]</td>
<td>[19,127]</td>
<td>[-76,66]</td>
</tr>
</tbody>
</table>
The results of these tests are as follows:

For site “A” and at the 5% level of significance:

5 – min aggregation: Max. Discharge = Max. Pre-Breakdown > Breakdown flow
15 – min aggregation: Max. Discharge > Max. Pre-Breakdown > Breakdown flow

For site “B” and at the 5% level of significance:

5 – min aggregation: Max. Pre-Breakdown > Max. Discharge = Breakdown flow
15 – min aggregation: Max. Pre-Breakdown > Max. Discharge = Breakdown flow

As shown, when the differences are tested, the number of inequalities among the three flow parameters increases. The results of the two analysis methods are consistent: at site A, the maximum discharge flows are generally higher, while at site B the maximum pre-breakdown flows are generally higher. For both sites, breakdown flows are generally the lowest.

CONCLUSIONS

Three flow parameters were examined in this paper, to determine which one would be more appropriate for use in defining the “capacity” of a freeway:

*Breakdown and Breakdown Flow:* Breakdown was defined as the traffic condition when the average speed of all lanes on the section dropped below 90 km/h for at least a 15-minute period. The breakdown flow was defined as the 5-minute (or 15-minute) flow immediately prior to the breakdown.

*Observed Maximum Pre-Breakdown Flow:* The maximum sustained 5-minute (or 15-minute) flow observed prior to the breakdown.

*Observed Maximum Discharge Flow:* The maximum flow observed after the breakdown, under congested conditions. This is the 5-minute (or 15-minute) aggregated flow rate under congested conditions, which were defined here as corresponding to speeds below 90 km/h at the study site for at least a 15-minute period.

It was concluded that:

- The numerical value of each of these three parameters varies, and their range is relatively large, in the order of several hundred veh/h/l.
- These parameters are normally distributed for both sites and both analysis aggregation intervals.
- The numerical value of breakdown flows is almost always lower than both the maximum pre-breakdown and the maximum discharge flows.
- The maximum pre-breakdown flow tends to be lower than the maximum discharge flow in Site “A”, but the opposite is observed in Site “B”. A possible
explanation for this difference may be that geometric characteristics and sight
distance may result in different operations under high- and low-speed conditions.

RECOMMENDATIONS

It is important that the HCM makes a clear distinction in the definition of the three parameters. This is particularly important for the definitions of breakdown flow and maximum pre-breakdown flow, which have wrongly been thought of as occurring at the same time. As shown above, this is not the case. Furthermore, the first was shown to be almost always lower than the second for the eighty breakdown events studied.

Breakdown can be defined using a speed threshold, which may be different for different types of freeway segments, and be determined as a function of geometric characteristics and free flow speed at the site. In addition to the speed threshold, or speed drop there should be an associated duration of that speed drop to ensure that breakdown has occurred. Consistent with previous research, duration of 15-minutes was found to be a reasonable interval for the sites studied in this paper.

Regarding the definition of capacity, the authors recommend that the breakdown flow be used. There are four advantages to using breakdown flow in the capacity definition: a) it was shown to be at both sites the lowest value of the three, thus it would be a conservative estimate of the traffic-carrying ability of a freeway; b) breakdown must occur before one can identify and measure the other two parameters, when conducting field studies; c) it is consistent with the current implication that capacity is the boundary between non-congested and congested conditions; and d) breakdown flow can be associated to the probability of breakdown, or probability that congestion would occur, which would be of interest in traffic management and control.

It is recommended that distributions of capacity (i.e., breakdown flow) are developed and incorporated into the HCM. To accomplish this, additional research should be conducted to establish whether breakdown flow distributions are different for different types of segments. It is possible that differences in the shape and numerical value of the breakdown flow distributions exist as a function of design parameters. The identification of design factors and the role they may play in capacity distributions is an important consideration in maximizing the operational efficiency of a freeway segment. Lane-by-lane flow distributions and their effect on breakdown should also be examined.

It is also recommended that distributions of discharge flow are developed and incorporated into the HCM, as they would be valuable in the analysis of congested conditions. It is particularly important to investigate whether geometric conditions affect the differences in maximum flows observed prior and after the breakdown.

Lastly, it is recommended that the HCM incorporate guidelines for obtaining each of these flow variables in the field to ensure consistency in traffic observations and analyses.
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