CHAPTER 36
CONCEPTS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 36 is the supplemental chapter for Volume 1, Concepts, of the *Highway Capacity Manual* (HCM).

Section 2 supplements material in Chapter 7, Interpreting HCM and Alternative Tool Results. It provides information on the recommended number of significant digits to use in presenting results and guidance on presenting analysis results to decision makers, the public, and practitioners.

Sections 3 and 4 supplement material in Chapter 4, Traffic Operations and Capacity Concepts. Section 3 provides guidance on measuring travel time reliability in the field, and Section 4 presents travel time reliability values for selected freeway and arterial facilities as an aid to analysts in interpreting travel time reliability performance measures.

Section 5 supplements Chapters 4 and 7. It provides expanded guidance on the use of vehicle trajectory analysis as a means by which performance measures can be consistently estimated by various alternative analysis tools.

Section 6 supplements Chapter 1, HCM User's Guide. Section 5 of Chapter 1 presented the changes to the HCM made in the Sixth Edition, along with the research basis for those changes. Section 6 of this chapter identifies the new material in the HCM 2010 that was not changed in the Sixth Edition, along with the research basis for that material.

2. PRESENTATION OF RESULTS

GUIDANCE ON THE DISPLAY OF HCM RESULTS

Tabular values and calculated results are displayed in a consistent manner throughout the HCM. Analyst adherence to these conventions is suggested. A key objective is to use the number of significant digits that is reasonable, to indicate to users, decision makers, and other viewers that the results are not extremely precise but take on the precision and accuracy associated with the input variables used. This guidance applies primarily to inputs and final outputs; intermediate results in a series of calculations should not be rounded unless specifically indicated by a particular methodology.

Input Values

Following is a list of representative (not exhaustive) input variables and the suggested number of digits for each.

- Volume (whole number);
- Grade (whole number);
- Lane width (one decimal place);
- Percentage of heavy vehicles (whole number);
- Peak hour factor (two decimal places);
- Pedestrian volume (whole number);
- Bicycle volume (whole number);
- Parking maneuvers (whole number);
- Bus stopping (whole number);
- Green, yellow, all-red, and cycle times (one decimal place);
- Lost time/phase (whole number); and
- Minimum pedestrian time (one decimal place).

Adjustment Factors

Factors interpolated from tabular material can use one more decimal place than is presented in the table. Factors generated from equations can be taken to three decimal places.

Service Volume Tables

When volumes for service volume tables are rounded, the precision used should be no greater than the nearest 10 vehicles or passenger cars for hourly tables and no greater than the nearest 100 vehicles or passenger cars for daily tables.

Free-Flow Speed

For a base free-flow speed (FFS), show the value to the nearest 1 mi/h. If the FFS has been adjusted for various conditions and is considered an intermediate calculation, show speed to the nearest 0.1 mi/h.

Speeds

For threshold values that define level of service (LOS), show speed to the nearest 1 mi/h. For intermediate calculations of speed, use one decimal place.

Volume-to-Capacity and Demand-to-Capacity Ratios

Show volume-to-capacity and demand-to-capacity ratios with two decimal places.

Delay

In computing delay, show results with one decimal place. In presenting delay as a threshold value in LOS tables, show a whole number.

Density

Show density results with one decimal place.

Pedestrian Space

Show pedestrian space values with one decimal place.

Occurrences and Events

For all event-based items, use values to a whole number. These items include parking maneuvers, buses stopping, and passing and meeting events along a pedestrian or bicycle path.

General Factors

In performing all calculations on a computer, the full precision available should be used. Intermediate calculation outputs should be displayed to three significant digits throughout. For the measure that defines LOS, the number of significant digits presented should exceed by one the number of significant digits shown in the LOS table.

PRESENTING RESULTS TO FACILITATE INTERPRETATION

Performance measures selected should be related to the problem being addressed.

Several performance measures can result from HCM analyses. Determination of the appropriate measures will depend on the transportation need being studied. However, decision-making situations generally can be divided into those involving the public (e.g., city councils and community groups) and those involving technicians (e.g., state and local engineering and planning staff).

The HCM is highly technical and complex. The results of the analyses can be difficult for people to interpret for decision making unless the data are carefully organized and presented. In general, the results should be presented as simply as possible. The presentation might use a small set of performance measures and provide the data in an aggregate form without losing the ability to relate to the underlying variations and factors that generated the results.

The LOS concept was created, in part, to make presentation of results easier than if numerical values of service measures were reported directly. In many cases, analysts and decision makers prefer to see one service measure rather than multiple performance measures. At the same time, relying solely on LOS results in making recommendations or decisions can lead to important information available from other performance measures being overlooked. Despite the limitations to its usefulness, the LOS concept remains a part of the HCM because of its acceptance by the public and decision makers.

Decision makers who represent the public usually prefer measures that their constituents can understand. The public can relate to LOS results, which describe relative differences in highway operations. Unit delay (e.g., seconds per vehicle) and travel speed are also readily understood. However, volume-to-capacity ratio, density, percent time-spent-following, and vehicle hours of travel are not measures to which the public easily relates. In the selection of measures to present, recognition by the analyst of the orientation of the decision maker and the context in which the decision will be made is important. In general, these measures can be differentiated as system user or system manager oriented.

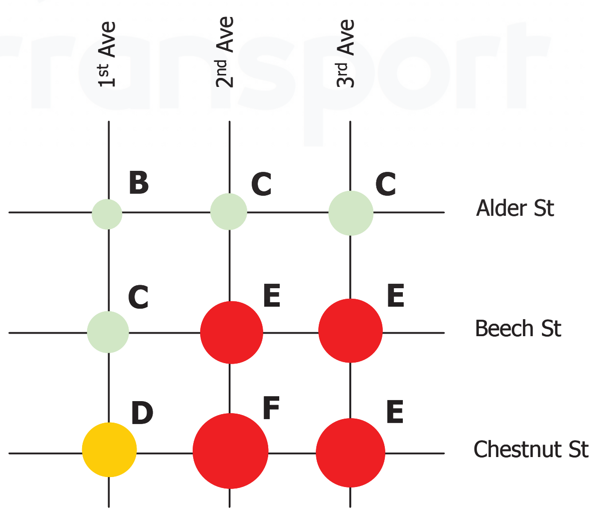
GRAPHIC REPRESENTATION OF RESULTS

Historically, data and analysis results have been presented primarily in tables. However, results may be best presented as pictures and supplemented only as necessary with the underlying numbers in some situations. Graphs and charts should be conceived and fashioned to aid in interpretation of the meaning behind the numbers (*1*).

Present results to make them very plain (obvious) to the audience.

Most performance measures in the HCM are quantitative, continuous variables. However, LOS values result from step functions and do not lend themselves to graphing. When they are placed on a scale, LOS results must be given an equivalent numeric value, as shown in Exhibit 36-1, which presents the LOS for a group of intersections. The LOS letter is indicated, and shaded (or colored) areas indicate intersections that are below, at, or above the analysis objective of LOS D. The size of the indicator at each intersection shows the relative control delay value for the indicated LOS.

Exhibit 36-1
Example of a Graphic Display of LOS



The issue is whether the change in value between successive LOS values (i.e., the interval) should be equal. For example, is conversion of LOS A to F to a scale of 0 through 5 appropriate? Should the numerical equivalent assigned to the difference of the thresholds between LOS A and B be the same as the difference between LOS E and F? These questions have not been addressed in research, except in the area of traveler perception models. Furthermore, LOS F is not given an upper bound. Therefore, a graph of LOS should be considered ordinal, not interval, because the numeric differences between the levels would not appear significant.

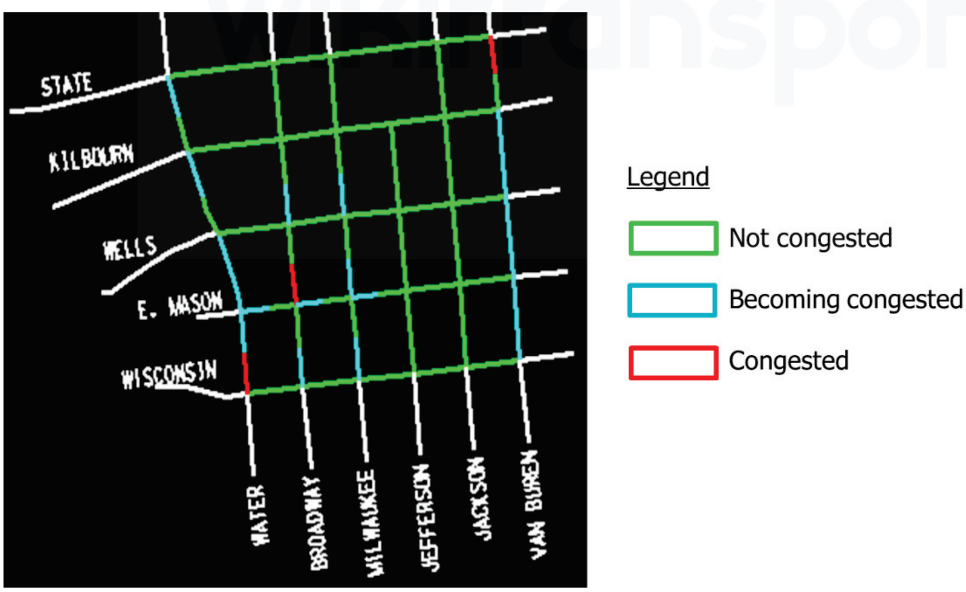
However, it is difficult to refrain from comparing the differences. A scale representing the relative values of the LOS letters would have to incorporate the judgment of the analyst and the opinions of the public or decision makers—a difficult task. A thematic graphic presentation avoids this issue. In Exhibit 36-2, for example, shading is used to highlight time periods and basic freeway segments that do not meet the objective LOS (in this case, D).

Exhibit 36-2
Example of a Thematic Graphic Display of LOS

Start Time	Segment 1	Segment 2	Segment 3	Segment 4
5:00 p.m.	A	B	B	A
5:15 p.m.	B	B	D	A
5:30 p.m.	B	B	F	A
5:45 p.m.	B	D	F	A
6:00 p.m.	B	F	F	A
6:15 p.m.	D	F	E	A
6:30 p.m.	D	E	C	A
6:45 p.m.	B	B	B	A

Further simplification of the presentation can be achieved by converting LOS letters into general descriptors of conditions. For example, Exhibit 36-3 shows a map of a portion of a downtown area, where street segments have been labeled by the analyst as "not congested" (e.g., LOS A, B, or C), "becoming congested" (e.g., LOS D or E), or "congested" (e.g., LOS F). (Note that these represent the analyst's choice of how to interpret and present the results; the HCM does not define specific levels of congestion.) This type of presentation is particularly useful for planning applications where many inputs into the HCM method have been defaulted and therefore the results may be less precise.

Exhibit 36-3
Example Presentation of Planning Analysis Results



Source: City of Milwaukee.

The HCM provides valuable assistance in making transportation management decisions in a wide range of situations. It offers the user a selection of performance measures to meet a variety of needs. The analyst should recognize that using the HCM involves mixing art with science. Sound judgment is needed not only for interpreting the values produced but also for summarizing and presenting the results.

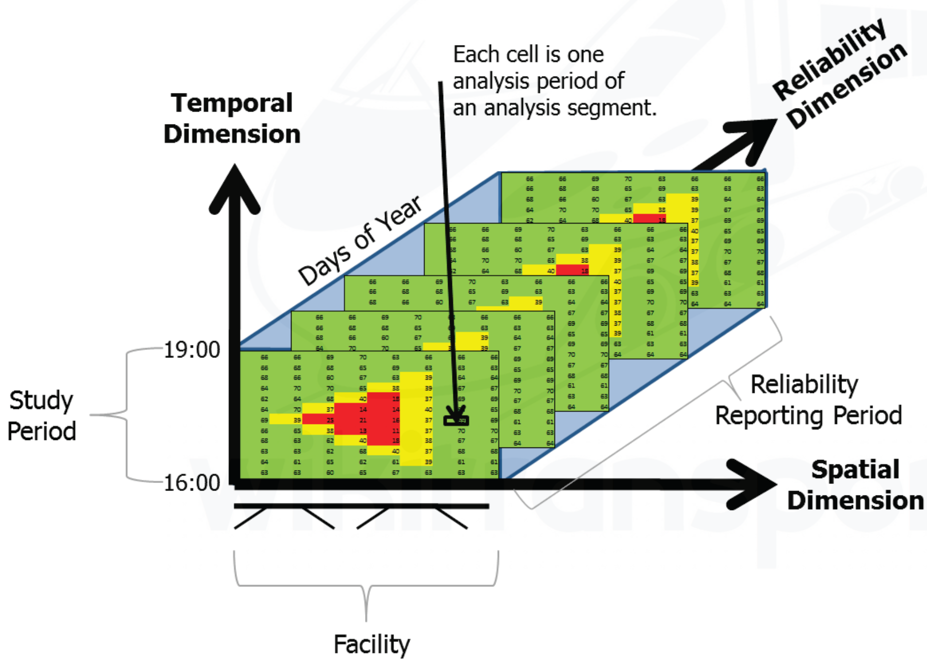
3. MEASURING TRAVEL TIME RELIABILITY IN THE FIELD

This section provides a recommended method for measuring travel time reliability in the field. The intent is to provide a standardized method for gathering and reporting travel time reliability for freeways and arterials directly from field sensors, which can be used for validating estimates of reliability produced by the HCM method and for consistently comparing reliability across facilities.

MEASUREMENT OF TRAVEL TIME RELIABILITY

Measuring travel time reliability in the field involves the development of the three-dimensional reliability box. The three dimensions of reliability are the study section of the facility, the daily study period, and the reliability reporting period (Exhibit 36-4). For example, travel time reliability can be computed for a 1-mi length of freeway during the afternoon peak hour for all nonholiday weekdays in a year.

Exhibit 36-4
Three-Dimensional Reliability Box



Source: Zegeer et al. (*2*).

DATA SOURCES FOR TRAVEL TIME RELIABILITY

Travel time reliability (and travel times generally) may be measured by recording a sample of the vehicle travel times over a fixed length of facility (probe vehicle method) or by recording the spot speeds of all vehicles as they pass over a set of stationary detectors. The latter method will be called for convenience the “spot measurement detector method”; many technologies are available (loops, radar, video, etc.) for measuring spot speeds.

Measuring reliability is all about measuring variability, so the larger the sample (in terms of number of vehicles and hours of the year), the more confidence one can have in the result.

Travel time, like demand, exhibits strong daily and weekly cyclic patterns. There may also be strong seasonal patterns to both demand and travel time. To obtain a useful estimate of the travel time distribution for any given hour of the day or day of the week, a sufficient sample of that hour and that day (and that season, if seasonality is significant) must be obtained to estimate the mean and the standard deviation of the travel time for that hour (and day of the week) within an acceptable range of accuracy. A reference provides details and examples of computing the required sample size to estimate the mean of the travel time distribution for the hour (*3*).

Estimating the standard deviation of the travel time distribution generally requires a much larger sample than estimating the mean to the same precision. To estimate the standard deviation of a normal distribution to within 10% of its true value at the 95% confidence level will require on the order of 200 samples of travel time for the hour (close to a year's worth of nonholiday, weekday data). Only 50 samples are needed to estimate the standard deviation to within 20% of its true value at a 95% confidence level (*4*).

Note that travel time is not normally distributed, so the minimum sample sizes described here should be considered as providing lower confidence levels than the 95% confidence level cited from the literature for the normal distribution.

Roadway-Based Spot Measurement Detectors

Spot measurement detectors can be as close as ⅓ to ½ mi apart, but they can be much farther apart. However, as detector spacing increases, the assumption that speeds are constant over the entire distance becomes more problematic. While an upper limit on spacing has not been established by research, detector spacing of ½ mi or less is greatly preferred.

Single detectors will measure the time a vehicle spends within the detector's detection zone and will divide this time by the estimated average vehicle length (supplied by the operator) to arrive at the estimated speed of the vehicle.

Pairs of detectors will measure the lag between the time the leading edge of the vehicle arrives at the first detector and the time the leading edge arrives at the second detector. The distance between the two detectors is divided by the time difference between the arrival of the leading edge of the vehicle at the upstream detector and its arrival at the downstream detector to obtain the vehicle speed for the short distance between the two detectors.

Probe Vehicles

Electronic toll tag or Bluetooth readers can be deployed at certain segments of freeway so that time stamps of vehicles crossing at these locations can be tracked. When a vehicle with a toll tag or a discoverable Bluetooth device crosses locations with readers, identification of the same vehicle can be matched with different time stamps and corresponding locations. Then the travel time between a pair of toll tag reader locations can be obtained.

In addition, "crowd-sourced" data may be available. To obtain such data, the movements of vehicles and people carrying various GPS-equipped telecommunication devices are monitored anonymously. The observed point speed data or the point-to-point travel times are filtered, converted into average travel times, and archived for later retrieval. The Federal Highway Administration's (FHWA's) National Performance Management Research Data Set is one example of a crowd-sourced database of travel times (*5*).

For point-to-point measurements of travel time, the analyst will need to develop and apply a filtering algorithm that removes vehicles from the sample that take an excessive amount of time to appear at the downstream detector because they have left the facility to stop for errands between the two detectors. The closer together the two readers, the tighter the filtering criterion can be.

Comparison of Sampling Methods

Spot detectors (e.g., loops) take a vertical sample of the facility time–space diagram, while probe vehicle (e.g., electronic toll collection) detectors take a diagonal sample of the facility time–space diagram (compare Exhibit 36-5 and Exhibit 36-6).

At the time of writing, the probe data available from vendors resemble detector data more closely than true probe data. The data may have started out as recorded positions of selected vehicles traveling on a facility, but the processed data that analysts receive are speeds on a link. Consequently, vendor-supplied data at present do not look at all like the Bluetooth or toll tag data collected by agencies.

Exhibit 36-5
Spot Speed (Vertical) Sampling of Loop Detectors

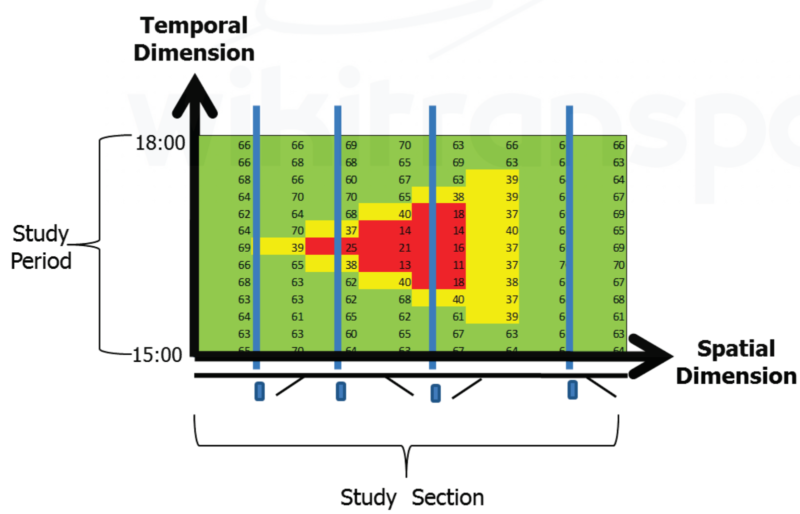
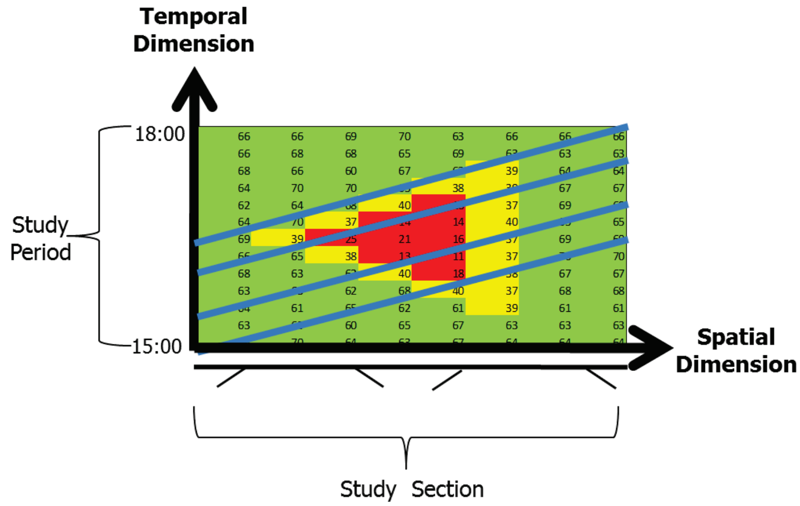
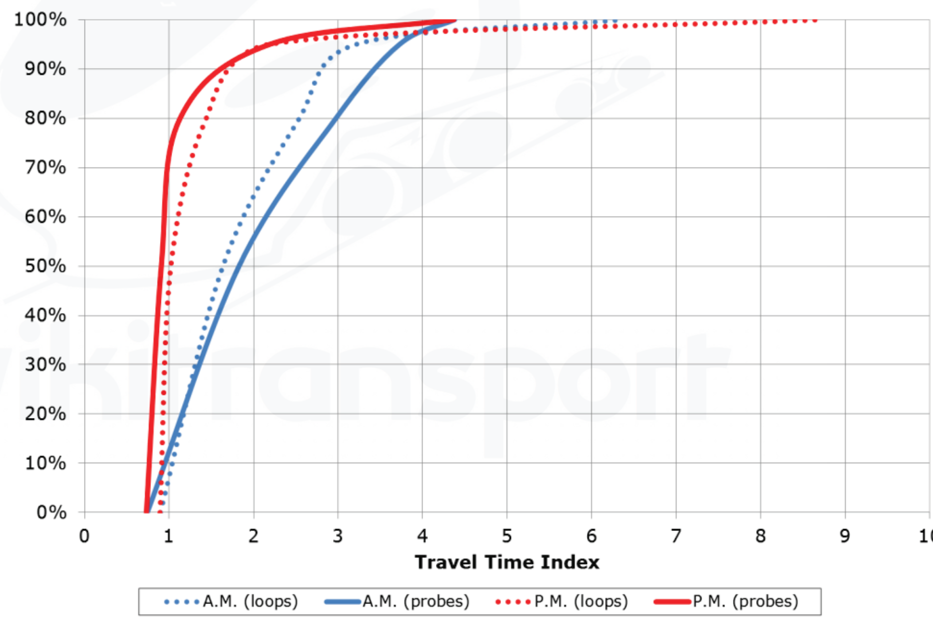


Exhibit 36-6
Time–Space (Diagonal) Sampling of Probe Vehicle Detectors



Since the two measurement methods sample the three-dimensional reliability space differently, they will produce slightly different estimates of the travel time reliability distribution, as illustrated for one freeway in Exhibit 36-7. However, the differences between the methods will generally be less than the differences in reliability between different peak periods.

Exhibit 36-7
Comparison of Loop Detector and Probe Cumulative Travel Time Distributions



Source: Kittelson & Associates, Inc.
Note: I-80 westbound, Contra Costa County, California.

Each method has its strengths and weaknesses, and neither method is always the best. A dense network of loop detectors may produce better estimates than a sparse network of toll tag readers. The reverse may also be true. Thus the choice of method is contingent on the density of the detection available for each method.

Similarly, crowd-sourced data may be superior or inferior to field detector–based measuring methods, depending on the sample size and the gaps in the crowd-sourced data and the density and reliability of the field detectors.

RECOMMENDED METHOD FOR COMPUTING RELIABILITY BY USING ROADWAY-BASED SPOT MEASUREMENT DETECTORS

The recommended method for computing travel time reliability statistics for freeways by using stationary sensors of spot speeds and volumes is described below. Because of the highly varying nature of speeds by distance from signal on urban streets, this method is *not* recommended for urban streets.

1. *Define reliability study bounds.* Select facility direction, length, study period, and reliability reporting period. The analyst should select the reliability reporting period appropriate for the purposes of the analysis. This may be all the nonholiday weekdays of a year (approximately 250 days out of the year) if the analyst is evaluating the reliability of a facility that has regular recurring weekday congestion. It may be the summer or winter weekends of a year if the analyst is evaluating a facility with regular recreational travel congestion.
2. *Download data.* Download lane-by-lane vehicle speeds and volumes aggregated or averaged to 5-min periods for all mainline speed detectors for the selected study direction, within the selected facility length and study period, and for all days included in the reliability reporting period.
3. *Quality check data.*
    a. If the system fills gaps in detector data (e.g., detectors down) with estimates, remove data with less than 70% observed rating.
    b. Remove unrealistic speeds from the data set. Analysts will need to review the data and use local knowledge to determine what is unreasonable. In addition, FHWA provides guidance on quality control for detector data (*6*).
    c. Gaps in data are treated as nonobservations.
4. *Compute 5-min vehicle miles traveled (VMT).*
    a. For each detector station, identify the length of facility represented by the detector. This is usually half the distance to the upstream detector station plus half the distance to the downstream detector, but it can be a different value based on local knowledge of the facility.
    b. Sum volumes across all lanes at the detector station for 5-min time periods.
    c. Neglect periods when the detector is not functioning.
    d. $VMT(t, d) = V(t, d) \times L(d)$, where $VMT(t, d)$ = vehicle miles traveled during time period t measured at detector station d; $L(d)$ = length represented by detector station d (mi), and $V(t, d)$ = sum of lane volumes (veh) measured at detector station d during time period t.
5. *Compute 5-min vehicle hours traveled (VHT).*
    a. $VHT(t, d) = VMT(t, d) / S(t, d)$, where $VHT(t, d)$ = vehicle hours traveled during time period t measured at lane detector station d

and $S(t, d)$ = arithmetic average speed of vehicles (mi/h) measured during time period t at lane detector station d.

b. Neglect periods when the detector is not functioning.

6. *Compute the FFS for the facility.* For a facility analysis, the use of data from continuously operating devices (roadway detectors or probe vehicles) is the preferred method, as described below. However, the analyst should be satisfied with the quality of the data from the suggested time periods before proceeding. For performance monitoring of multiple facilities or complete roadway systems, the analyst may wish to establish FFS in other ways, mainly to establish a consistent base from which to track trends. For example, if monitoring is performed on an annual basis, calculation of FFS every year on a facility may lead to different values for each year. One way to address this problem is to use the empirical method given below in the first year of the monitoring program to set the FFS for all years. Other methods include picking a constant FFS on the basis of agency policy for that facility type or speed limit. The "agency policy" FFS reflects in some way the agency's performance objectives for the facility. Whatever method is used, the analyst should clearly specify it.

   a. Select a nonholiday weekend (or other period known to the analyst to be a light-flow period without congestion).

   b. For each detector, obtain 5-min speeds for 7 to 9 a.m. on a typical *weekend* morning (or other uncongested, light-flow period).

   c. Neglect periods when the detector is not functioning.

   d. Quality control for excessively high speeds or excessively low volumes as discussed earlier.

   e. Identify the average (mean) speed during the observed light-flow period. That is the FFS for the detector.

   f. Convert speed to segment travel times.

   g. Sum segment times to obtain facility free-flow travel times.

7. *Compute the VMT and VHT for each time period.*

$$VMT_t = \sum_d VMT_{t,d}$$

$$VHT_t = \sum_d VHT_{t,d}$$

8. *Compute the travel time index (TTI) for the facility for each time period.*

$$TTI_t = \frac{VHT_t}{VHTFF_t}$$

where $VHTFF_t$ is the VHT that would occur during time period t if all vehicles traveled at the FFS:

$$VHTFF_t = \frac{VMT_t}{FFS}$$

9. *Develop a distribution of the TTI_t values for the facility for the entire analysis period.* Each TTI_t value becomes an observation in the distribution. All performance measures are derived from this distribution. The statistics and percentiles are calculated by using VMT_t as a weight; this is done to account for the fact that the TTIs in each time period are based on a different number of vehicles.

RECOMMENDED METHOD FOR COMPUTING RELIABILITY BY USING PROBE VEHICLES

TMC segments are industry-standard roadway sections used in communicating traffic information to drivers (for example, via a vehicle's navigation system).

The recommended method for computing travel time reliability statistics for freeways and arterials by using probe vehicles and Bluetooth, toll tag, or license plate readers is described below. The instructions assume that the data are obtained from a commercial vendor of historical traffic message channel (TMC) segment speed data.

1. *Define reliability study bounds*. Select the facility direction, length, study period, and reliability reporting period. The analyst should select the reliability reporting period appropriate for the purposes of the analysis. This may be all the nonholiday weekdays of a year (approximately 250 days out of the year) if the analyst is evaluating the reliability of a facility that has regular recurring weekday congestion. It may be the summer or winter weekends of a year if the analyst is evaluating a facility with regular recreational travel congestion.
2. *Download data.* Download TMC segment speeds (or travel times if Bluetooth or toll tag reader data are being used) aggregated or averaged to 5-min (or similar) periods for all mainline segments for the selected study direction and selected facility length, for all study periods and days included in the reliability reporting period.
3. *Quality check data.*
   a. If travel time data (e.g., Bluetooth or toll tag reader data) are being used, convert data to speeds for error-checking purposes.
   b. Remove unrealistic speeds from the data set. Analysts will need to review the data and use local knowledge to determine what is unreasonable.
4. *Compute facility travel times for each analysis period.*
   a. For each TMC (or Bluetooth or toll tag reader) segment, identify its length in miles (to the nearest 0.01 mi).
   b. Divide the segment length by speed to obtain the segment travel time for each analysis period (skip this step if Bluetooth or toll tag travel time data are being used).
   c. Sum the segment travel times to obtain the facility travel time for each time period.

Vendor-supplied urban street reference speeds may include traffic signal delays not included in the HCM definition of FFS.

5. *Compute FFS for the facility.* Steps 5a to 5g below are only applicable to freeway facilities, as urban street segment reference speeds or probe vehicle speeds under low-volume conditions may include traffic signal delays not included in the HCM definition of FFS. For urban street facilities, FFS can be established by use of an alternate method, including (*a*) picking a constant FFS on the basis of agency policy for a given facility type or speed limit; (*b*) establishing FFS on the basis of the actual speed limit (e.g., speed limit plus a constant); and (*c*) measuring speeds at locations not influenced by traffic control or junctions (e.g., midsegment on urban streets).
   a. If the segment reference speed provided by the commercial vendor is reliable, that can be used for the FFS. If it is not reliable, perform the following steps.
   b. Select a nonholiday weekend (or other period known to the analyst to be a light-flow period without congestion).
   c. For each segment, obtain speeds for 5-min time periods for 7 to 9 a.m. on a typical *weekend* morning (or other uncongested, light-flow period).
   d. Quality control for excessively high speeds or travel times as explained earlier.
   e. Identify the average (mean) speed. That is the FFS for the segment.
   f. Convert the segment speed to segment travel times (segment length divided by segment speed).
   g. Sum the segment times to obtain facility free-flow travel times.
6. *Compute the VMT and VHT for each time period.*

$$VMT_t = \sum_d VMT_{t,d}$$

$$VHT_t = \sum_d VHT_{t,d}$$

7. *Compute the TTI for the facility for each time period.*

$$TTI_t = \frac{VHT_t}{VHTFF_t}$$

   where $VHTFF_t$ is the VHT that would occur during time period t if all vehicles traveled at the FFS:

$$VHTFF_t = \frac{VMT_t}{FFS}$$

8. *Develop a distribution of the TTI_t values for the facility for the entire analysis period.* Each TTI_t value becomes an observation in the distribution. All performance measures are derived from this distribution. The statistics and percentiles are calculated by using VMT_t as a weight; this is done to account for the fact that the TTIs in each time period are based on a different number of vehicles.

4. RELIABILITY VALUES FOR SELECTED U.S. FACILITIES

DATA SOURCES

Reliability data for 1 year of nonholiday weekday travel time were obtained from the following sources:

- 2-min traffic speed data in the I-95 corridor for 2010 (*7*), and
- 5-min traffic speed data in California for 2010 (*8*).

The first data set includes freeway and urban street reliability data for states and metropolitan areas in the I-95 corridor (i.e., U.S. East Coast). The average speed of traffic was measured every 2 min for each TMC road segment (*9*). Road segments vary but generally terminate at a decision point for the driver (e.g., intersection, start of left-turn pocket, ramp merge or diverge). Traffic speeds are obtained by monitoring the positions of GPS units in participating vehicles. A "free-flow reference speed" is established for each TMC segment on the basis of empirical observations. It may not correspond exactly to the FFS that would be estimated by the HCM's analytical or field-measurement methods.

The base travel time for freeways was an empirically measured free-flow travel time. For urban streets, the base travel time corresponded to the 85th percentile highest speed observed during off-peak hours. Therefore, the free-flow reference speeds used in these data sets do not correspond exactly to the FFS that an HCM method would produce.

The California data include freeway reliability data for the state's major metropolitan areas, plus reliability data for one urban street in Chula Vista. The data come from two sources: toll tag readers and loop detectors. California's system provides a function for stringing together a series of loop detector station speeds into an estimate of the overall average speed for the facility. The loop detector data used to compute an average speed for each segment of the facility are offset by the time taken by the average vehicle to traverse the upstream segment. Thus for a selected direction of travel, the average speed of vehicles in Segment 1 is used to compute the average travel time t for the selected time period (e.g., 5 min) for that segment starting at time $T = 0$. The mean speed is computed for the next downstream segment for the 5-min period starting at $T = 0 + t$. The resulting mean travel times are then added together to get the average travel time of vehicles for the 5-min period starting their trip at $0 < T < 5$ min.

RELIABILITY STATISTICS FOR A CROSS SECTION OF U.S. FACILITIES

Exhibit 36-8 through Exhibit 36-11 show the distribution of 50th percentile travel time index (TTI_{50}), mean travel time index (TTI_{mean}), and planning time index (PTI or TTI_{95}) observed in the data set of U.S. freeways and urban streets described above, for all time periods combined, the 2-h a.m. peak period, the 2-h midday period, and the 2-h p.m. peak period, respectively. Exhibit 36-11 is an expanded version of Exhibit 11-3 in Chapter 11, Freeway Reliability and Strategy Assessment. The exhibits provide values in 5 percentile increments and include a combined set of values.

Because the free-flow reference speeds used in these data sets do not exactly correspond to the FFS estimates that an HCM analytical method or field-measurement technique would produce, the TTI values presented in these exhibits should be interpreted as being relative to the stated reference speed.

TTIs calculated by using the HCM definition of FFS could be different, but the general patterns observed would be similar.

Exhibit 36-8
Rankings of U.S. Facilities by Mean TTI and PTI (A.M. Peak, Midday, and P.M. Peak Combined)

	Freeways			Urban Streets		
Percentile Rank	**TTI_{50}**	**TTI_{mean}**	**PTI**	**TTI_{50}**	**TTI_{mean}**	**PTI**
Minimum	1.01	1.02	1.07	1.03	1.06	1.23
Worst 95%	1.02	1.05	1.09	1.09	1.12	1.27
Worst 90%	1.02	1.06	1.13	1.13	1.15	1.29
Worst 85%	1.04	1.06	1.14	1.15	1.16	1.32
Worst 80%	1.05	1.08	1.17	1.17	1.20	1.33
Worst 75%	1.05	1.08	1.22	1.19	1.20	1.35
Worst 70%	1.05	1.09	1.25	1.19	1.22	1.36
Worst 65%	1.06	1.10	1.30	1.20	1.22	1.39
Worst 60%	1.07	1.12	1.34	1.20	1.23	1.41
Worst 55%	1.08	1.15	1.39	1.21	1.23	1.42
Worst 50%	1.10	1.16	1.47	1.23	1.26	1.44
Worst 45%	1.11	1.19	1.57	1.24	1.27	1.47
Worst 40%	1.13	1.23	1.73	1.25	1.28	1.49
Worst 35%	1.14	1.30	1.84	1.25	1.29	1.52
Worst 30%	1.17	1.33	1.97	1.26	1.30	1.54
Worst 25%	1.20	1.39	2.24	1.30	1.34	1.60
Worst 20%	1.26	1.43	2.71	1.33	1.36	1.63
Worst 15%	1.31	1.51	2.90	1.35	1.38	1.70
Worst 10%	1.59	1.78	3.34	1.39	1.47	1.84
Worst 5%	1.75	1.97	3.60	1.45	1.54	1.98
Maximum	2.55	2.73	4.73	1.60	1.66	2.55

Source: Derived from directional values in Exhibit 36-12 through Exhibit 36-17. Entries are the lowest value for a category.

Note: TTI_{50} = 50th percentile travel time index (50th percentile travel time divided by base travel time).
TTI_{mean} = mean travel time index (mean travel time divided by base travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
For freeways, the base travel time is the free-flow travel time. For urban streets, the base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

Exhibit 36-9
Rankings of U.S. Facilities by Mean TTI and PTI (A.M. Peak)

	Freeways			Urban Streets		
Percentile Rank	**TTI_{50}**	**TTI_{mean}**	**PTI**	**TTI_{50}**	**TTI_{mean}**	**PTI**
Minimum	1.01	1.02	1.07	1.03	1.06	1.24
Worst 95%	1.01	1.03	1.08	1.08	1.12	1.24
Worst 90%	1.03	1.05	1.12	1.12	1.13	1.27
Worst 85%	1.04	1.06	1.14	1.13	1.15	1.29
Worst 80%	1.04	1.08	1.14	1.14	1.16	1.29
Worst 75%	1.05	1.08	1.17	1.15	1.16	1.31
Worst 70%	1.06	1.09	1.24	1.16	1.17	1.33
Worst 65%	1.07	1.10	1.36	1.18	1.20	1.35
Worst 60%	1.08	1.11	1.40	1.19	1.20	1.37
Worst 55%	1.08	1.16	1.47	1.19	1.21	1.39
Worst 50%	1.09	1.17	1.53	1.20	1.23	1.41
Worst 45%	1.11	1.19	1.58	1.20	1.24	1.42
Worst 40%	1.12	1.21	1.70	1.22	1.26	1.44
Worst 35%	1.13	1.21	1.78	1.24	1.27	1.50
Worst 30%	1.15	1.25	1.89	1.24	1.28	1.52
Worst 25%	1.20	1.42	2.13	1.25	1.29	1.54
Worst 20%	1.28	1.48	2.61	1.26	1.29	1.57
Worst 15%	1.54	1.83	3.17	1.26	1.29	1.66
Worst 10%	1.72	1.93	3.55	1.28	1.31	1.71
Worst 5%	1.95	2.08	3.92	1.35	1.36	1.84
Maximum	2.17	2.73	4.66	1.38	1.49	2.13

Source: Derived from directional values in Exhibit 36-12 through Exhibit 36-17. Entries are the lowest value for a category.

Note: TTI_{50} = 50th percentile travel time index (50th percentile travel time divided by base travel time).
TTI_{mean} = mean travel time index (mean travel time divided by base travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
For freeways, the base travel time is the free-flow travel time. For urban streets, the base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

Exhibit 36-10
Rankings of U.S. Facilities by Mean TTI and PTI (Midday)

	Freeways			Urban Streets		
Percentile Rank	**TTI_{50}**	**TTI_{mean}**	**PTI**	**TTI_{50}**	**TTI_{mean}**	**PTI**
Minimum	1.02	1.03	1.07	1.05	1.07	1.23
Worst 95%	1.02	1.04	1.08	1.08	1.10	1.27
Worst 90%	1.02	1.05	1.11	1.15	1.18	1.28
Worst 85%	1.02	1.06	1.14	1.16	1.18	1.30
Worst 80%	1.03	1.06	1.15	1.18	1.20	1.33
Worst 75%	1.04	1.08	1.17	1.19	1.21	1.34
Worst 70%	1.05	1.08	1.20	1.19	1.22	1.37
Worst 65%	1.05	1.09	1.21	1.20	1.22	1.39
Worst 60%	1.05	1.09	1.24	1.20	1.23	1.41
Worst 55%	1.06	1.11	1.26	1.21	1.23	1.42
Worst 50%	1.06	1.12	1.32	1.22	1.24	1.45
Worst 45%	1.07	1.13	1.34	1.24	1.27	1.47
Worst 40%	1.09	1.15	1.37	1.25	1.29	1.48
Worst 35%	1.09	1.15	1.43	1.25	1.30	1.51
Worst 30%	1.10	1.17	1.51	1.27	1.32	1.53
Worst 25%	1.12	1.26	1.65	1.30	1.34	1.57
Worst 20%	1.14	1.30	1.92	1.31	1.34	1.60
Worst 15%	1.16	1.32	2.41	1.32	1.35	1.63
Worst 10%	1.17	1.42	2.85	1.33	1.38	1.63
Worst 5%	1.21	1.46	3.16	1.35	1.42	1.86
Maximum	1.31	1.76	3.96	1.47	1.55	2.01

Source: Derived from directional values in Exhibit 36-12 through Exhibit 36-17. Entries are the lowest value for a category.

Note: TTI_{50} = 50th percentile travel time index (50th percentile travel time divided by base travel time).
TTI_{mean} = mean travel time index (mean travel time divided by base travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
For freeways, the base travel time is the free-flow travel time. For urban streets, the base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

Exhibit 36-11
Rankings of U.S. Facilities by Mean TTI and PTI (P.M. Peak)

	Freeways			Urban Streets		
Percentile Rank	**TTI_{50}**	**TTI_{mean}**	**PTI**	**TTI_{50}**	**TTI_{mean}**	**PTI**
Minimum	1.01	1.05	1.10	1.13	1.14	1.32
Worst 95%	1.03	1.06	1.14	1.13	1.15	1.35
Worst 90%	1.04	1.06	1.22	1.18	1.21	1.35
Worst 85%	1.05	1.08	1.24	1.20	1.22	1.36
Worst 80%	1.05	1.09	1.28	1.20	1.22	1.37
Worst 75%	1.06	1.10	1.31	1.21	1.23	1.40
Worst 70%	1.07	1.14	1.32	1.22	1.23	1.41
Worst 65%	1.11	1.16	1.38	1.23	1.25	1.42
Worst 60%	1.14	1.23	1.59	1.24	1.26	1.44
Worst 55%	1.14	1.30	1.72	1.24	1.27	1.47
Worst 50%	1.17	1.31	1.85	1.25	1.28	1.49
Worst 45%	1.20	1.34	1.94	1.25	1.29	1.50
Worst 40%	1.21	1.36	2.06	1.31	1.33	1.52
Worst 35%	1.23	1.38	2.25	1.34	1.36	1.59
Worst 30%	1.26	1.41	2.46	1.35	1.38	1.64
Worst 25%	1.29	1.48	2.62	1.39	1.44	1.68
Worst 20%	1.35	1.57	2.77	1.41	1.49	1.78
Worst 15%	1.61	1.71	2.93	1.41	1.52	1.83
Worst 10%	1.70	1.86	3.26	1.49	1.56	1.88
Worst 5%	1.76	1.99	3.54	1.56	1.60	2.10
Maximum	2.55	2.73	4.73	1.60	1.66	2.55

Source: Derived from directional values in Exhibit 36-12 through Exhibit 36-17. Entries are the lowest value for a category.

Note: TTI_{50} = 50th percentile travel time index (50th percentile travel time divided by base travel time).
TTI_{mean} = mean travel time index (mean travel time divided by base travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
For freeways, the base travel time is the free-flow travel time. For urban streets, the base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

Exhibit 36-12 through Exhibit 36-14 present the source freeway data for the a.m. peak, midday, and p.m. peak periods, respectively. Exhibit 36-15 through

Exhibit 36-17 present the source urban street data for the a.m. peak, midday, and p.m. peak periods, respectively.

Exhibit 36-12
Freeway Reliability Values: Weekday A.M. Peak Period

Location	Freeway	Length (mi)	FFRS (mi/h)	Direction	Avg. Travel Time (min)	TTI_{mean}	PTI
Delaware	I-495	11.5	65	NB	11.0	1.03	1.08
Delaware	I-495	11.6	65	SB	11.1	1.03	1.07
Delaware	I-95	13.4	60	NB	14.6	1.10	1.37
Delaware	I-95	13.1	61	SB	13.5	1.05	1.13
Los Angeles	I-10	4.6	64	EB	4.5	1.06	1.12
Los Angeles	I-10	4.6	65	WB	4.5	1.08	1.14
Los Angeles	I-210	4.6	66	EB	4.9	1.17	1.57
Los Angeles	I-210	4.6	69	WB	4.6	1.16	1.57
Maryland	I-495 ES	26.5	63	SB	28.0	1.10	1.42
Maryland	I-495 ES	26.7	62	NB	31.1	1.20	1.71
Maryland	I-495 WS	15.4	60	NB	18.3	1.19	1.68
Maryland	I-495 WS	15.3	61	SB	26.9	1.78	2.71
Pennsylvania	I-76	3.7	51	EB	4.7	1.08	1.22
Pennsylvania	I-76	3.6	49	WB	6.5	1.49	3.06
Philadelphia	I-76	3.7	51	EB	4.7	1.08	1.22
Philadelphia	I-76	3.6	49	WB	6.5	1.79	3.06
Sacramento	US-50	6.0	69	EB	5.7	1.10	1.27
Sacramento	US-50	6.0	71	WB	6.2	1.21	1.78
Sacramento	I-80	12.4	68	EB	11.5	1.06	1.14
Sacramento	I-80	12.4	67	WB	12.0	1.09	1.17
San Diego	I-5	10.6	71	NB	11.1	1.23	1.81
San Diego	I-5	10.6	72	SB	9.1	1.02	1.07
San Diego	I-15	3.9	70	NB	4.7	1.41	2.10
San Diego	I-15	3.9	69	SB	7.3	1.58	3.38
San Francisco	I-880	4.6	71	NB	4.6	1.17	1.47
San Francisco	I-880	4.8	67	SB	8.2	1.92	3.57
San Francisco	I-680	4.2	66	NB	4.8	1.26	1.92
San Francisco	I-680	4.7	65	SB	5.2	1.21	1.49

Notes: FFRS = free-flow reference speed, calculated empirically; may not exactly match the HCM-defined FFS.
TTI_{mean} = mean travel time index (mean travel time divided by free-flow travel time).
PTI = planning time index (95th percentile travel time divided by free-flow travel time).
NB = northbound, SB = southbound, EB = eastbound, WB = westbound, ES = east side, WS = west side.

Exhibit 36-13
Freeway Reliability Values: Weekday Midday Periods

Location	Roadway	Length (mi)	FFRS (mi/h)	Direction	Avg. Travel Time (min)	TTI_{mean}	PTI
Delaware	I-495	11.5	65	NB	11.0	1.03	1.07
Delaware	I-495	11.6	65	SB	11.3	1.05	1.11
Delaware	I-95	13.4	60	NB	13.9	1.05	1.20
Delaware	I-95	13.1	61	SB	13.8	1.08	1.34
Los Angeles	I-10	4.6	64	EB	4.5	1.06	1.15
Los Angeles	I-10	4.6	65	WB	4.5	1.08	1.14
Los Angeles	I-210	4.6	66	EB	4.8	1.16	1.32
Los Angeles	I-210	4.6	69	WB	4.4	1.10	1.18
Maryland	I-495 ES	26.5	63	SB	27.2	1.07	1.31
Maryland	I-495 ES	26.7	62	NB	28.2	1.09	1.42
Maryland	I-495 WS	15.4	60	NB	20.5	1.34	2.69
Maryland	I-495 WS	15.3	61	SB	19.8	1.30	2.26
Pennsylvania	I-76	3.7	51	EB	5.0	1.13	1.39
Pennsylvania	I-76	3.6	49	WB	6.2	1.43	2.95
Philadelphia	I-76	3.7	51	EB	5.0	1.13	1.39
Philadelphia	I-76	3.6	49	WB	6.2	1.72	2.95
Sacramento	US-50	6.0	69	EB	5.8	1.11	1.20
Sacramento	US-50	6.0	71	WB	5.9	1.15	1.47
Sacramento	I-80	12.4	68	EB	11.8	1.09	1.25
Sacramento	I-80	12.4	67	WB	11.9	1.08	1.14
San Diego	I-5	10.6	71	NB	9.3	1.03	1.07
San Diego	I-5	10.6	72	SB	9.5	1.06	1.21
San Diego	I-15	3.9	70	NB	3.8	1.13	1.23
San Diego	I-15	3.9	69	SB	4.1	1.24	1.61
San Francisco	I-880	4.6	71	NB	4.5	1.17	1.53
San Francisco	I-880	4.8	67	SB	5.6	1.31	1.96
San Francisco	I-680	4.2	66	NB	4.4	1.15	1.34
San Francisco	I-680	4.7	65	SB	5.0	1.15	1.26

Notes: FFRS = free-flow reference speed, calculated empirically; may not exactly match the HCM-defined FFS.
TTI_{mean} = mean travel time index (mean travel time divided by free-flow travel time).
PTI = planning time index (95th percentile travel time divided by free-flow travel time).
NB = northbound, SB = southbound, EB = eastbound, WB = westbound, ES = east side, WS = west side.

Exhibit 36-14
Freeway Reliability Values: Weekday P.M. Peak Period

Location	Roadway	Length (mi)	FFRS (mi/h)	Direction	Avg. Travel Time (min)	TTI_{mean}	PTI
Delaware	I-495	11.5	65	NB	11.4	1.06	1.23
Delaware	I-495	11.6	65	SB	12.0	1.10	1.39
Delaware	I-95	13.4	60	NB	14.6	1.10	1.29
Delaware	I-95	13.1	61	SB	16.8	1.30	1.83
Los Angeles	I-10	4.6	64	EB	5.1	1.20	1.31
Los Angeles	I-10	4.6	65	WB	4.9	1.16	1.28
Los Angeles	I-210	4.6	66	EB	4.5	1.08	1.35
Los Angeles	I-210	4.6	69	WB	4.2	1.06	1.15
Maryland	I-495 ES	26.5	63	SB	33.3	1.31	1.85
Maryland	I-495 ES	26.7	62	NB	33.7	1.31	1.98
Maryland	I-495 WS	15.4	60	NB	41.8	2.73	4.73
Maryland	I-495 WS	15.3	61	SB	30.6	2.02	3.67
Pennsylvania	I-76	3.7	51	EB	6.0	1.36	1.94
Pennsylvania	I-76	3.6	49	WB	7.7	1.78	3.29
Philadelphia	I-76	3.7	51	EB	6.0	1.36	1.94
Philadelphia	I-76	3.6	49	WB	7.7	1.78	3.29
Sacramento	US-50	6.0	69	EB	7.0	1.35	2.12
Sacramento	US-50	6.0	71	WB	7.7	1.51	2.74
Sacramento	I-80	12.4	68	EB	13.9	1.28	1.84
Sacramento	I-80	12.4	67	WB	12.1	1.09	1.31
San Diego	I-5	10.6	71	NB	9.4	1.05	1.22
San Diego	I-5	10.6	72	SB	13.1	1.47	2.45
San Diego	I-15	3.9	70	NB	4.7	1.18	2.97
San Diego	I-15	3.9	69	SB	3.8	1.14	1.50
San Francisco	I-880	4.6	71	NB	7.7	1.96	3.43
San Francisco	I-880	4.8	67	SB	5.8	1.34	1.73
San Francisco	I-680	4.2	66	NB	6.1	1.59	2.74
San Francisco	I-680	4.7	65	SB	5.0	1.15	1.25

Notes: FFRS = free-flow reference speed, calculated empirically; may not exactly match the HCM-defined FFS.
TTI_{mean} = mean travel time index (mean travel time divided by free-flow travel time).
PTI = planning time index (95th percentile travel time divided by free-flow travel time).
NB = northbound, SB = southbound, EB = eastbound, WB = westbound, ES = east side, WS = west side.

Exhibit 36-15
Urban Street Reliability Values: Weekday A.M. Peak Period

Location	Roadway	Length (mi)	FFRS (mi/h)	Direction	Avg. Travel Time (min)	TTI_{mean}	PTI
California	Telegraph Canyon Rd.	4.4	45	EB	6.19	1.06	1.24
California	Telegraph Canyon Rd.	4.4	45	WB	6.57	1.12	1.42
Delaware	US-202	3.8	42	NB	6.97	1.28	1.55
Delaware	US-202	3.9	44	SB	6.52	1.20	1.41
Maryland	Hwy 175	7.4	38	NB	13.92	1.20	1.32
Maryland	Hwy 175	7.4	38	SB	14.00	1.21	1.35
Maryland	Hwy 193	5.9	33	EB	13.75	1.26	1.45
Maryland	Hwy 193	5.9	33	WB	13.72	1.27	1.52
Maryland	Hwy 198	10.1	42	EB	16.51	1.13	1.24
Maryland	Hwy 198	10.2	41	WB	16.95	1.15	1.27
Maryland	Hwy 355	4.2	30	NB	10.37	1.23	1.38
Maryland	Hwy 355	4.2	30	SB	12.57	1.49	2.13
Maryland	Randolph Rd.	6.7	35	EB	14.13	1.22	1.36
Maryland	Randolph Rd.	6.7	35	WB	15.28	1.31	1.71
Maryland	US-40	4.1	41	EB	7.00	1.16	1.29
Maryland	US-40	4.2	39	WB	8.50	1.29	1.85
Pennsylvania	US-1	8.0	33	NB	19.68	1.36	1.67
Pennsylvania	US-1	7.6	32	SB	18.18	1.29	1.52
Philadelphia	Hwy 611	3.4	20	NB	13.26	1.29	1.58
Philadelphia	Hwy 611	3.3	19	SB	12.89	1.25	1.41
South Carolina	US-378	5.5	44	EB	8.61	1.16	1.29
South Carolina	US-378	5.4	45	WB	8.37	1.16	1.31

Notes: FFRS = free-flow reference speed, calculated empirically; may not exactly match the HCM-defined FFS.
TTI_{mean} = mean travel time index (mean travel time divided by free-flow travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
NB = northbound, SB = southbound, EB = eastbound, WB = westbound.
The base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

Exhibit 36-16
Urban Street Reliability Values: Weekday Midday Periods

Location	Roadway	Length (mi)	FFRS (mi/h)	Direction	Avg. Travel Time (min)	TTI_{mean}	PTI
California	Telegraph Canyon Rd.	4.4	45	EB	6.27	1.07	1.23
California	Telegraph Canyon Rd.	4.4	45	WB	6.46	1.10	1.28
Delaware	US-202	3.8	42	NB	7.28	1.34	1.63
Delaware	US-202	3.9	44	SB	6.93	1.28	1.47
Maryland	Hwy 175	7.4	38	NB	13.93	1.20	1.33
Maryland	Hwy 175	7.4	38	SB	14.17	1.23	1.38
Maryland	Hwy 193	5.9	33	EB	14.29	1.31	1.52
Maryland	Hwy 193	5.9	33	WB	13.99	1.29	1.49
Maryland	Hwy 198	10.1	42	EB	17.13	1.18	1.29
Maryland	Hwy 198	10.2	41	WB	17.47	1.18	1.27
Maryland	Hwy 355	4.2	30	NB	12.02	1.42	1.87
Maryland	Hwy 355	4.2	30	SB	13.07	1.55	2.01
Maryland	Randolph Rd.	6.7	35	EB	14.22	1.23	1.36
Maryland	Randolph Rd.	6.7	35	WB	14.62	1.25	1.42
Maryland	US-40	4.1	41	EB	7.44	1.23	1.47
Maryland	US-40	4.2	39	WB	8.01	1.22	1.42
Pennsylvania	US-1	8.0	33	NB	19.23	1.33	1.53
Pennsylvania	US-1	7.6	32	SB	19.02	1.35	1.58
Philadelphia	Hwy 611	3.4	20	NB	14.12	1.38	1.61
Philadelphia	Hwy 611	3.3	19	SB	13.78	1.34	1.63
South Carolina	US-378	5.5	44	EB	8.88	1.20	1.33
South Carolina	US-378	5.4	45	WB	8.78	1.22	1.40

Notes: FFRS = free-flow reference speed, calculated empirically; may not exactly match the HCM-defined FFS.
TTI_{mean} = mean travel time index (mean travel time divided by free-flow travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
NB = northbound, SB = southbound, EB = eastbound, WB = westbound.
The base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

Exhibit 36-17
Urban Street Reliability Values: Weekday P.M. Peak Period

Location	Roadway	Length (mi)	FFRS (mi/h)	Direction	Avg. Travel Time (min)	TTI_{mean}	PTI
California	Telegraph Canyon Rd.	4.4	45	EB	6.71	1.14	1.35
California	Telegraph Canyon Rd.	4.4	45	WB	6.73	1.15	1.35
Delaware	US-202	3.8	42	NB	7.42	1.36	1.62
Delaware	US-202	3.9	44	SB	6.84	1.26	1.43
Maryland	Hwy 175	7.4	38	NB	14.20	1.23	1.36
Maryland	Hwy 175	7.4	38	SB	14.81	1.28	1.49
Maryland	Hwy 193	5.9	33	EB	16.39	1.50	1.83
Maryland	Hwy 193	5.9	33	WB	15.67	1.45	1.69
Maryland	Hwy 198	10.1	42	EB	18.53	1.27	1.50
Maryland	Hwy 198	10.2	41	WB	17.81	1.21	1.32
Maryland	Hwy 355	4.2	30	NB	14.03	1.66	2.11
Maryland	Hwy 355	4.2	30	SB	13.47	1.60	1.89
Maryland	Randolph Rd.	6.7	35	EB	16.11	1.39	1.65
Maryland	Randolph Rd.	6.7	35	WB	14.33	1.23	1.36
Maryland	US-40	4.1	41	EB	9.40	1.56	2.55
Maryland	US-40	4.2	39	WB	8.04	1.22	1.41
Pennsylvania	US-1	8.0	33	NB	19.63	1.36	1.53
Pennsylvania	US-1	7.6	32	SB	21.31	1.52	1.80
Philadelphia	Hwy 611	3.4	20	NB	13.22	1.29	1.48
Philadelphia	Hwy 611	3.3	19	SB	13.19	1.28	1.46
South Carolina	US-378	5.5	44	EB	9.22	1.24	1.41
South Carolina	US-378	5.4	45	WB	8.81	1.22	1.39

Notes: FFRS = free-flow reference speed, calculated empirically; may not exactly match the HCM-defined FFS.
TTI_{mean} = mean travel time index (mean travel time divided by free-flow travel time).
PTI = planning time index (95th percentile travel time divided by base travel time).
NB = northbound, SB = southbound, EB = eastbound, WB = westbound.
The base travel time corresponds to the 85th percentile highest speed observed during off-peak hours.

RELIABILITY STATISTICS FOR FLORIDA FREEWAYS

Exhibit 36-18 presents reliability statistics for a cross section of Florida freeways (*10*). The data were gathered and reported for the p.m. peak period (4:30 to 6:00 p.m.) and are *not* aggregated over the length of the facility. The data consist of spot speeds that have been inverted into travel time rates (min/mi).

The reliability statistics for Florida are reported separately from the rest of the United States because Florida was testing a variety of definitions of FFS in the research from which these data were obtained (*10*). Florida usually sets the FFS for its freeways as the posted speed limit plus 5 mi/h. However, a speed of 5 mi/h less than the posted speed limit and a policy speed of 40 mi/h were also being tested for reliability computation purposes. The following statistics are presented:

- Four different TTIs (50th, 80th, 90th, and 95th percentile TTIs) based on a definition of FFS of the posted speed plus 5 mi/h;
- Two policy indices, one based on the 50th percentile speed and a target speed of the posted speed minus 5 mi/h, the other based on the 50th percentile speed and a speed of 40 mi/h;
- A buffer time index based on the 95th percentile speed and the mean speed; and
- A misery index based on the average of the highest 5% of travel times and a free-flow travel time derived from the posted speed plus 5 mi/h.

Exhibit 36-18
Florida Freeway Reliability Statistics

Location	TTI_{50}	TTI_{80}	TTI_{90}	TTI_{95} (PTI)	Policy Index Alt. 1	Policy Index Alt. 2	Buffer Time Index	Misery Index
I-95 NB at NW 19th St.	1.00	1.36	1.69	2.01	1.27	1.75	2.02	2.22
I-95 SB at NW 19th St.	1.08	1.19	1.58	2.01	1.27	1.75	1.86	2.48
I-95 NB, S of Atlantic Blvd.	1.03	1.28	1.73	2.23	1.27	1.75	2.16	2.74
I-95 SB, S of Atlantic Blvd.	1.10	1.36	1.89	2.37	1.27	1.75	2.15	2.93
SR 826 NB at NW 66th St.	2.40	2.82	3.07	3.35	1.33	1.50	1.39	3.69
SR 826 SB at NW 66th St.	1.01	1.28	2.63	4.06	1.33	1.50	4.02	4.62
SR 826 WB, W of NW 67th Ave.	1.04	1.08	1.21	1.77	1.33	1.50	1.70	2.10
SR 826 EB, W of NW 67th Ave.	0.98	1.00	1.02	1.04	1.33	1.50	1.07	1.10
I-4 EB, W of World Dr.	0.97	1.04	1.06	1.08	1.27	1.75	1.12	1.12
I-4 WB, W of World Dr.	1.02	1.09	1.49	1.90	1.27	1.75	1.86	2.22
I-4 EB, W of Central Florida Pkwy.	1.06	1.13	1.18	1.31	1.27	1.75	1.24	1.56
I-4 WB, W of Central Florida Pkwy.	1.05	1.36	1.63	1.81	1.27	1.75	1.72	2.03
I-275 NB, N of MLK Jr Blvd.	1.45	1.71	1.91	2.16	1.33	1.50	1.49	2.58
I-275 SB, N of MLK Jr Blvd.	0.97	1.01	1.04	1.12	1.33	1.50	1.15	1.28
I-275 NB, N of Fletcher Blvd.	1.05	1.07	1.11	1.21	1.33	1.50	1.16	1.35
I-275 SB, N of Fletcher Blvd.	0.96	0.98	0.99	1.00	1.33	1.50	1.04	1.01
I-10 EB, E of Lane Ave.	0.93	0.96	0.98	0.99	1.33	1.50	1.07	1.01
I-10 WB, E of Lane Ave.	0.97	1.10	1.24	1.46	1.33	1.50	1.51	1.87
I-95 NB, S of Spring Glen Rd.	1.04	1.09	1.26	1.77	1.27	1.75	1.70	2.00
I-95 SB, S of Spring Glen Rd.	1.16	1.30	1.42	1.60	1.27	1.75	1.38	1.88
Minimum	0.93	0.96	0.98	0.99	1.27	1.50	1.04	1.01
Average	1.11	1.26	1.51	1.81	1.30	1.63	1.64	2.09
Maximum	2.40	2.82	3.07	4.06	1.33	1.75	4.02	4.62

Source: Adapted from Kittelson & Associates, Inc. (*10*).

Notes: TTI_{xx} = travel time index based on the percentile speed indicated in the subscript and a free-flow speed defined as the posted speed plus 5 mi/h.
PTI = planning time index.
Policy Index Alternative 1 = index based on the 50th percentile speed and a target speed of the posted speed minus 10 mi/h.
Policy Index Alternative 2 = index based on the 50th percentile speed and a target speed of 40 mi/h.
Buffer time index = index based on the ratio of the 95th percentile and mean travel speeds.
Misery index = index based on the ratio of (*a*) the average of the highest 5% of travel times and (*b*) a free-flow travel time defined as the posted speed plus 5 mi/h.
N = north, S = south, E = east, W = west, NB = northbound, SB = southbound, EB = eastbound, WB = westbound.

5. VEHICLE TRAJECTORY ANALYSIS

INTRODUCTION

Overview

This section contains expanded guidance for the use of alternative traffic analysis tools (mostly microsimulation tools) in assessing the performance of highway facilities. An important part of the guidance deals with the use of vehicle trajectory analysis as the "lowest common denominator" for comparing performance measures from different tools. Material on vehicle trajectory analysis is also included in the following chapters:

- *Chapter 4, Traffic Operations and Capacity Concepts*, introduces the concept of individual vehicle trajectory analysis. A growing school of thought suggests that comparing results between traffic analysis tools and methods is possible only through analyzing vehicle trajectories as the "lowest common denominator." Vehicle trajectories can be used to develop performance measures that are consistent with HCM definitions, with field measurement techniques, and with each other. Examples of vehicle trajectory plots were shown that illustrate the visual properties of vehicle trajectories.
- *Chapter 7, Interpreting HCM and Alternative Tool Results*, explores the use of vehicle trajectory analysis in defining and estimating consistent performance measures. First, it introduces the mathematical properties of trajectories as an extension of the visual properties. Next, it identifies the performance measures that can be computed from trajectories and explores their compatibility with the performance measures estimated by the computational procedures presented throughout the HCM.

Chapter 7 presents general guidelines for defining and comparing measures from different traffic analysis tools. Those guidelines are expanded in this section through presentation of more specific trajectory analysis procedures by which consistent performance measures can be estimated. The trajectory analysis procedures described in this section were developed and tested by postprocessing the external trajectory files produced by a typical simulation tool. The postprocessor features and the process by which the procedures were developed are described elsewhere (*11*).

Several examples of the analysis of vehicle trajectories on both interrupted- and uninterrupted-flow facilities are presented here. These examples demonstrate the complexities that can arise, for example, in multilane situations, multiphase operations, situations in which the demand exceeds the capacity, and situations in which vehicles are unable to access a desired lane because of congestion. Specific procedures are then proposed and demonstrated with additional examples.

Mathematical Properties of Vehicle Trajectories

As was pointed out in Chapter 7, an analysis of vehicle trajectories requires a mathematical representation that includes a set of properties associated with each vehicle at specific points in time and space. Some of the material on mathematical properties of vehicle trajectories presented in this section is also included in Chapter 7. It is repeated here to provide a convenient introduction to the topic of vehicle trajectory analysis. A graphic representation of the path of an individual vehicle in space and time is also repeated here as Exhibit 36-19.

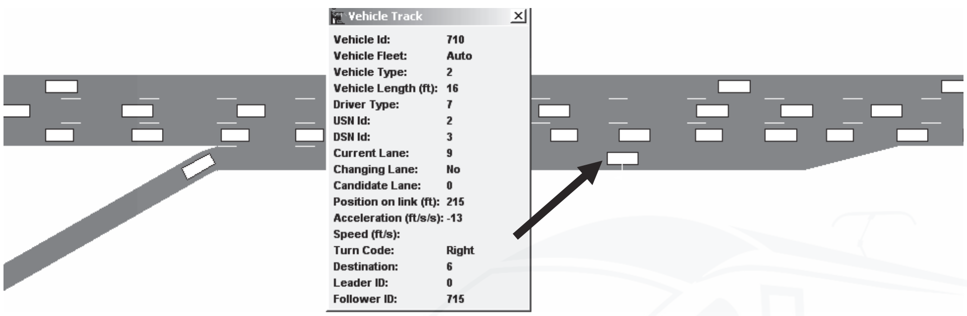


Exhibit 36-19
Vehicle Data Stored for Each Time Step

Many properties can be associated with a specific vehicle at a point in time. Some properties are required for the accurate determination of performance measures from trajectories. Others are used for different purposes, such as safety analysis.

Basic Trajectory Properties

The basic trajectory properties from which all the required performance measures can be estimated include the following information for each vehicle within the facility boundaries and for each time step within the analysis period:

- *Vehicle identification*: Vehicle identification is required to distinguish a specific vehicle from all other vehicles within the facility boundaries.
- *Position*: This property is the most basic of all, and many other properties may be derived from it. A one-dimensional position is sufficient to produce performance measures. Some question remains about a universal representation of position, because different tools specify the position in different ways. A common reference point for position needs to be established. A reference point that indicates the relative position of the vehicle in the link would be desirable to enable developers to produce uniform measures.
- *Link or segment*: A link or segment is required to associate performance measures with a specific link or analysis segment for reporting purposes.
- *Lane*: In multilane facilities, knowledge of the lane in which the vehicle is traveling is important because headways, densities, and other measures must be estimated by lane. It is also necessary for identifying lane changes.

Static Vehicle and Facility Parameters

Some required properties can be derived from the basic properties with knowledge of certain parameters that are constant with respect to time:

- *Vehicle length*: Required to convert headways to gaps, and
- *Link end positions*: Required to determine the position of the vehicle with respect to the upstream or downstream end of the link.

Some simulation tools repeat this static information in each record to avoid the need for an external parameter file.

Derived Trajectory Properties

The remainder of the required trajectory properties can be derived from the basic properties as follows:

- *Instantaneous speed*: This property can be determined from the relative positions of the vehicle at time t and time $t - \Delta t$ on the assumption of a constant acceleration during Δt. However, since most tools update vehicle positions from the speeds, speed is commonly included as a basic trajectory property.
- *Instantaneous acceleration*: This property can be determined from the relative speeds of the vehicle at time t and time $t - \Delta t$ on the assumption of a constant acceleration during Δt. However, since most tools update vehicle speeds from the acceleration, acceleration is commonly included as a basic trajectory property.

TRAJECTORY ANALYSIS EXAMPLES

This section demonstrates the ability of alternative analysis tools to quantify trajectory properties. Several examples are presented for both uninterrupted- and interrupted-flow facilities.

Basic Signalized Intersection

The first example is very basic. The intersection configuration involves two single-lane, one-way streets as shown in Exhibit 36-20. To simplify the situation even more, the simulation parameters are adjusted to enforce a uniform operation. Essentially, all the randomness inherent in simulation is removed. A simulation of uniform conditions would not normally produce useful results, but this example provides a good starting point for illustrating the nature of vehicle trajectory plots.

A trajectory plot showing two cycles of simulated operation for this example is presented in Exhibit 36-21(a). This form is the classic one that appears often in the literature to support discussion related to queue accumulation and discharge. A copy of the exhibit used in Chapter 31, Signalized Intersections: Supplemental, to illustrate the basic traffic signal principles is also included as Exhibit 36-21(b). The two figures are different in that the first was produced directly from the vehicle trajectory data while the second was drawn by hand. The ability to reproduce the classic representation from controlled conditions will provide a measure of confidence in the validity of future examples involving much more complicated situations.

Exhibit 36-20
Basic Signalized Intersection Example

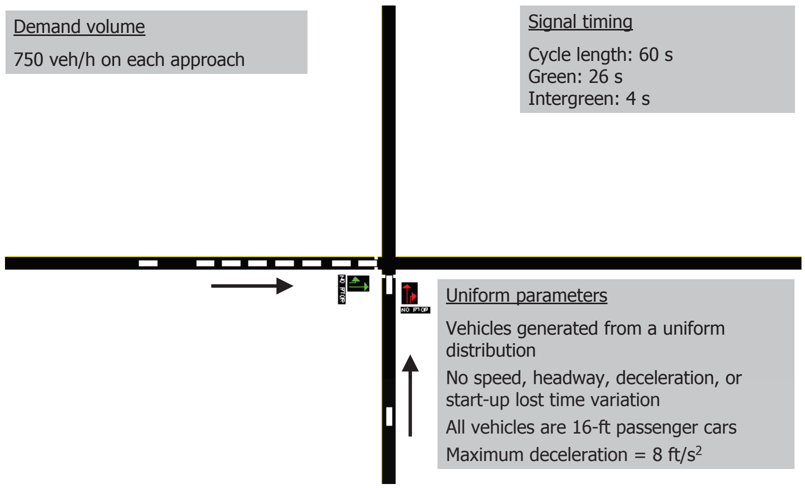
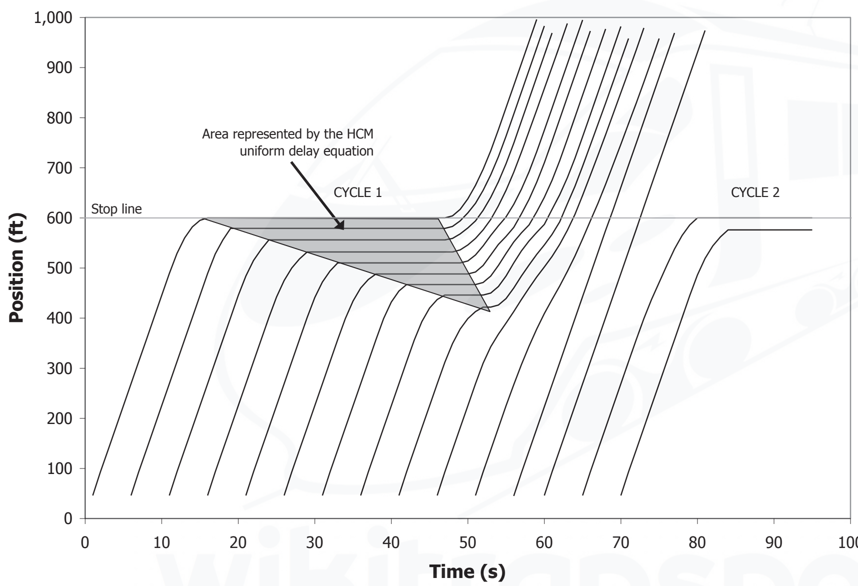
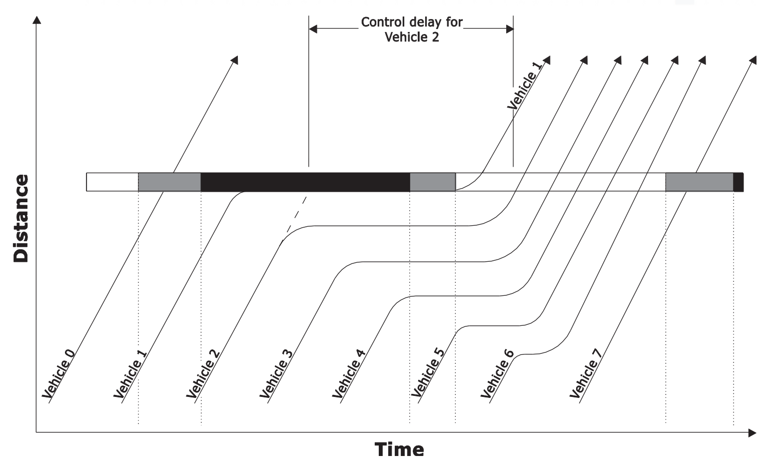


Exhibit 36-21
Trajectory Plots for Uniform Arrivals and Departures

Note the similarity between the trajectories obtained from the file (above) and those developed manually in Chapter 31 (below) to illustrate the basic principles of signalized intersection operation.



(a) Plot Produced from Simulation



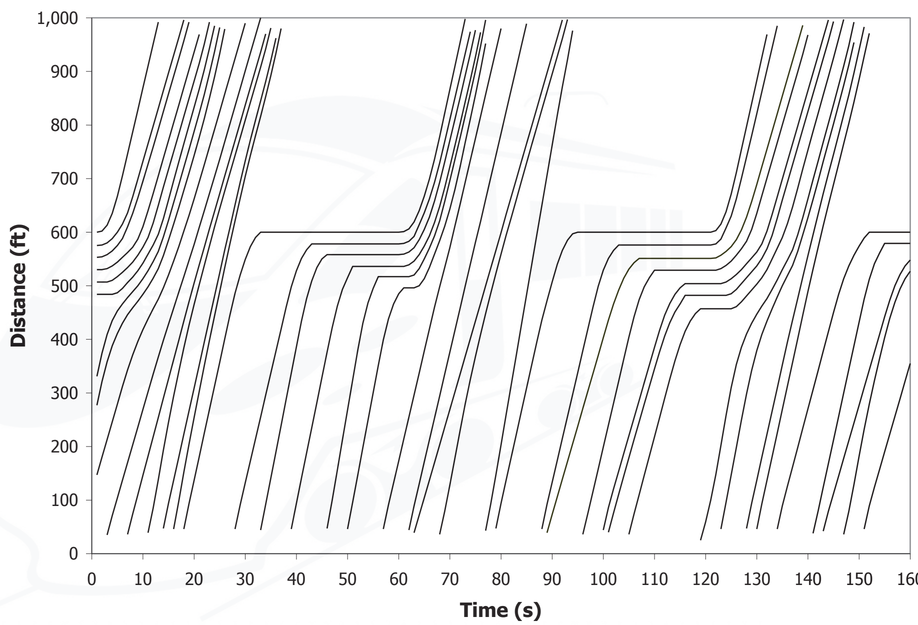
(b) Plot Produced by Hand

Restoring Randomness to the Simulation

To simplify the discussion, the first example was presented with all randomness removed from the operation. Subsequent examples are more realistic in their treatment of traffic flow. Vehicles are generated at entry points from a Poisson distribution, and the simulation tool's default parameters for randomizing driver behavior are applied.

Exhibit 36-22 shows a sample trajectory plot for the same operation depicted in Exhibit 36-21. As expected, the individual trajectories follow the same pattern as the uniform case, except that some spacings and speeds are not as consistent. The trajectory lines do not cross each other in this example because the example uses a single-lane approach and overtaking is not possible.

Exhibit 36-22
Introducing Randomness into the Simulation



Vehicle Trajectories for Oversaturated Operation

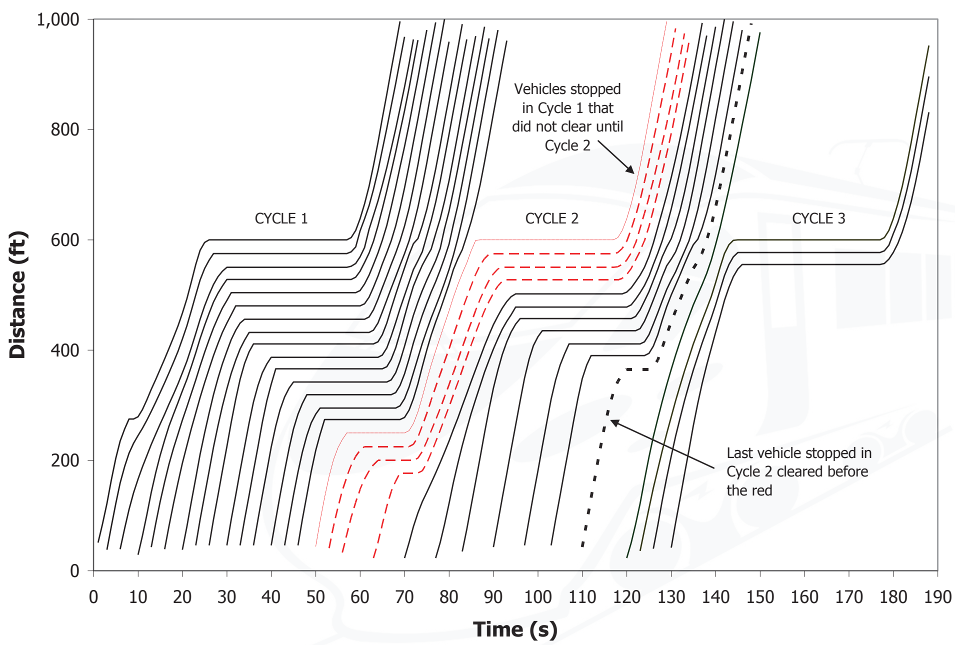
Up to this point, the examples have involved volume-to-capacity ratios less than 1.0, in which all vehicles arriving on a given cycle were able to clear on the same cycle. Saturation levels close to and above 1.0 present a different picture. Three cases are presented here:

1. *Cycle failure,* occurring when saturation approaches 1.0 and residual queues build on one cycle but are resolved on the next cycle;
2. *Oversaturated operation,* a situation in which the link has a demand volume exceeding the link's capacity and queues extend throughout the approach link; and
3. *Undersaturated operation,* in which queues extend to an upstream link for a part of a cycle because of closely spaced intersections.

Cycle Failure

A cycle failure example is presented in Exhibit 36-23. This trajectory plot shows a situation in which some vehicles arriving in Cycle 1 were unable to clear until Cycle 2. This condition is identified from the trajectory plot for four stopped vehicles (i.e., horizontal trajectory lines) that were forced to stop again before reaching the stop line. These vehicles became the first four vehicles in the queue for Cycle 2. Fortunately, the arrivals during Cycle 2 were few enough that all stopped vehicles were able to clear the intersection before the beginning of the red phase. A closer inspection of Exhibit 36-23 shows that one more vehicle, which was not stopped, was also able to clear.

Exhibit 36-23
Cycle Failure Example

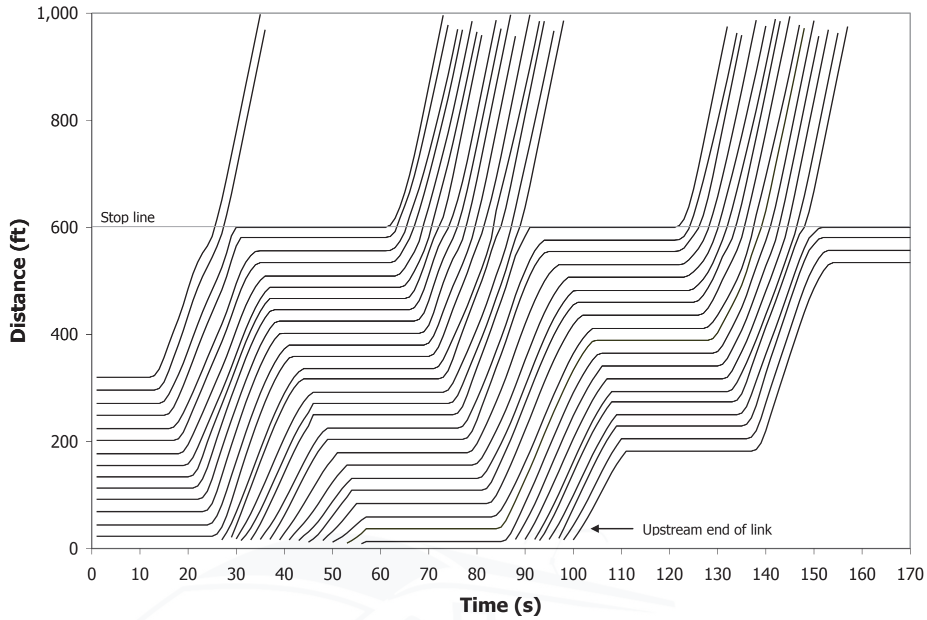


Severely Oversaturated Operation

Oversaturated operation was produced by increasing the demand volume to the point where it exceeded the capacity of the approach. The increased demand produced a queue that extended the length of the link. Inspection of the animated graphics showed that the queue did, in fact, back up beyond the link entry point.

The vehicle trajectory plot for this operation is presented in Exhibit 36-24. The move-up process is represented in the trajectories. Vehicles entering the link require up to three cycles to clear the intersection. The implications for control delay computations when the queue occupies a substantial proportion of the link are discussed in Chapter 7.

Exhibit 36-24
Oversaturated Signal Approach



A larger question is what to do with the vehicles denied entry during the analysis period. The answer is that, as indicated in Chapter 19, Signalized Intersections, the analysis period must be long enough to include a period of uncongested operation at each end. The delay to vehicles denied entry to this link will be accounted for in upstream links during the period. The upstream links must include a holding area outside the system. Some tools include the delay to vehicles denied entry and some do not. If a tool is used that does not include denied-entry delay, fictitious links must be built into the network structure for that purpose.

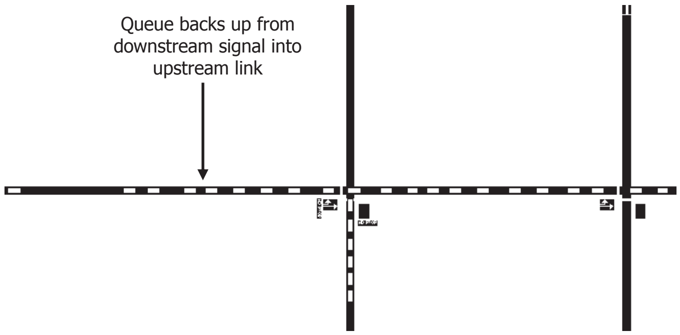
Queue Backup from a Downstream Signal

Even when an approach is not fully saturated, queues might back up from a downstream signal for a portion of the cycle. This happens when intersections are closely spaced. An example of queue backup within a cycle is shown in Exhibit 36-25.

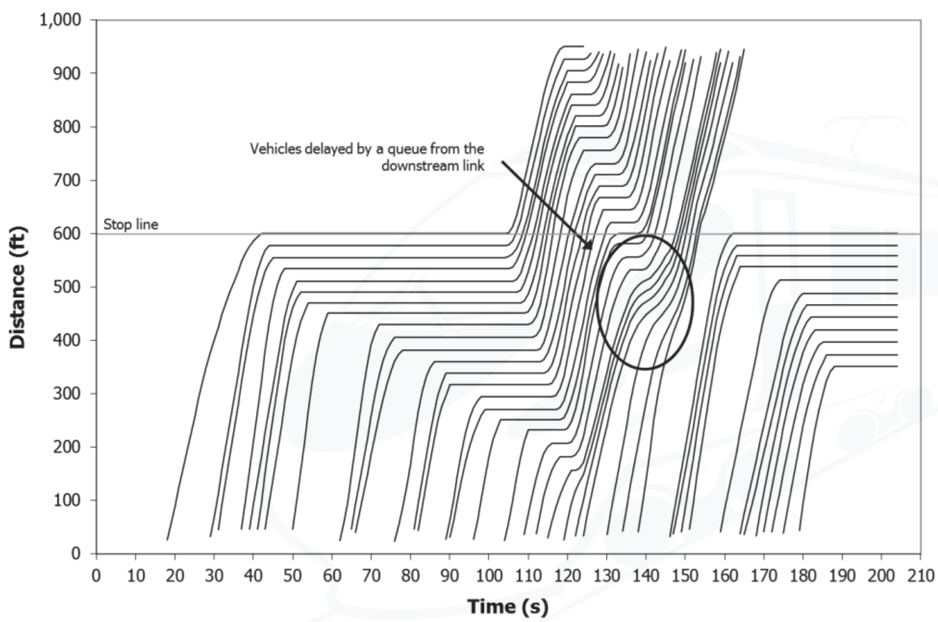
The two-intersection configuration for this example is shown in Exhibit 36-25(a). The graphics screen capture shows that vehicles that would normally pass through the upstream link are prevented from doing so by queues that extend beyond the end of the downstream link for a portion of the cycle. The question is how to treat the resulting delay.

By the definitions given to this point, the delay in the upstream link would be assigned to the upstream link, even though the signal on the downstream link was the primary cause. The important thing is not to overlook any delay and to assign all delay somewhere and in a consistent manner. With simulation modeling, the only practical place to assign delay consistently is the link on which the delay occurred. Subtle complexities make it impractical to do otherwise. For example, the root cause of a specific backup might not be the immediate downstream link. The backup might be secondary to a problem at some distant location in the network at some other point in time.

Exhibit 36-25
Queue Backup from a Downstream Signal



(a) Simulation Graphics Representation



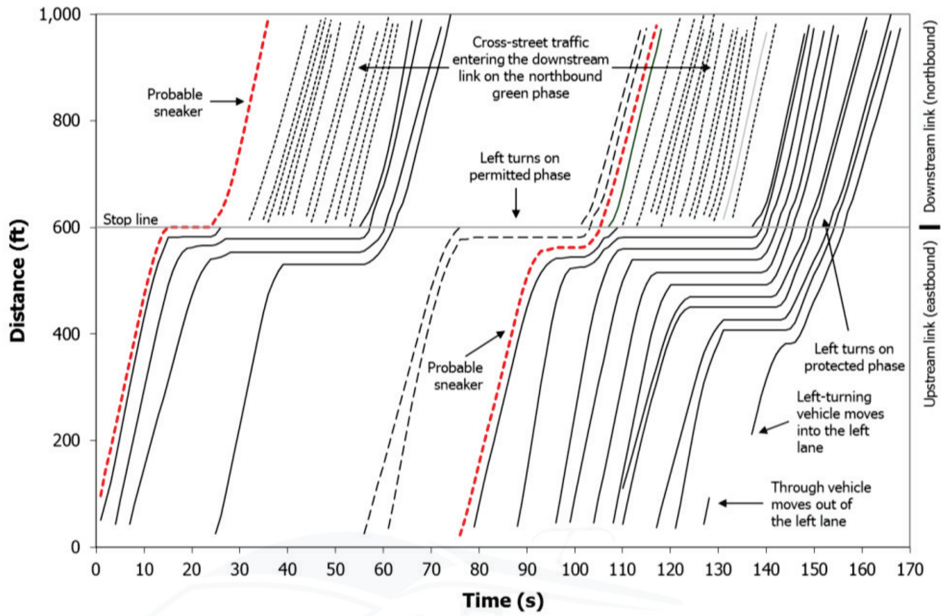
(b) Vehicle Trajectory Representation

More Complex Signal Phasing

Up to this point only simple signal phasing has been considered. Many applications involve simulating more complex phasing on urban streets. As an example of a more complex situation, a left turn moving on both a protected and a permitted phase is examined.

Exhibit 36-26 shows the trajectory plot for an eastbound left-turn movement from an exclusive lane controlled by a signal with both protected and permitted phases. In this case, the upstream link is the eastbound approach to the intersection and the downstream link is the northbound approach to the next intersection. Because the distance on a trajectory plot is one-dimensional, the distance scale is linear, even though the actual route takes a right-angle bend.

Exhibit 36-26
Trajectory Plot for More Complex Signal Phasing



Even with an undersaturated operation, this trajectory plot is substantially more involved than the previous ones. Several phenomena are identified in the exhibit, including the following:

1. Cross-street traffic entering the downstream link on the northbound phase: These vehicles do not appear on the upstream link because they are on a different link. They enter the downstream link at the stop line on the red phase for the left-turn movement of interest.
2. Left turns on the protected phase, shown as solid lines on the trajectory plot: The protected left-turn phase takes place immediately after the red phase. The left-turning vehicles begin to cross the stop line at that point.
3. Left turns on the permitted phase, shown as broken lines on the trajectory plot: The permitted left-turn phase takes place immediately after the protected phase. There is a gap in the trajectory plot because the left-turning vehicles must wait for oncoming traffic to clear.
4. Left-turn “sneakers”: Explicit identification of a sneaker on the trajectory plot is not possible; however, the last left turn to clear the intersection on the permitted phase is probably a sneaker if it enters at the end of the permitted phase.
5. Left-turn vehicles that enter the link in the through lane and change into the left lane somewhere along the link: These vehicles are identified by trajectories that begin in the middle of the link.
6. Through vehicles that enter the link in the left-turn lane and change into the through lane somewhere along the link: These vehicles are identified by trajectories that end abruptly in the middle of the link.

The trajectory plot shown for this example is more complex than the previous plots; however, performance can be analyzed in the same way.

Freeway Examples

Freeway trajectories follow the same definitions as surface street trajectories, but the queuing patterns differ because they are created by car-following phenomena and not by traffic signals. The performance measures of interest also differ. There is no notion of control delay on freeways because there is no control. The level of service on uninterrupted-flow facilities is based on traffic density expressed in units of vehicles per mile per lane. In some cases, such as merging segments, the density in specific lanes is of interest.

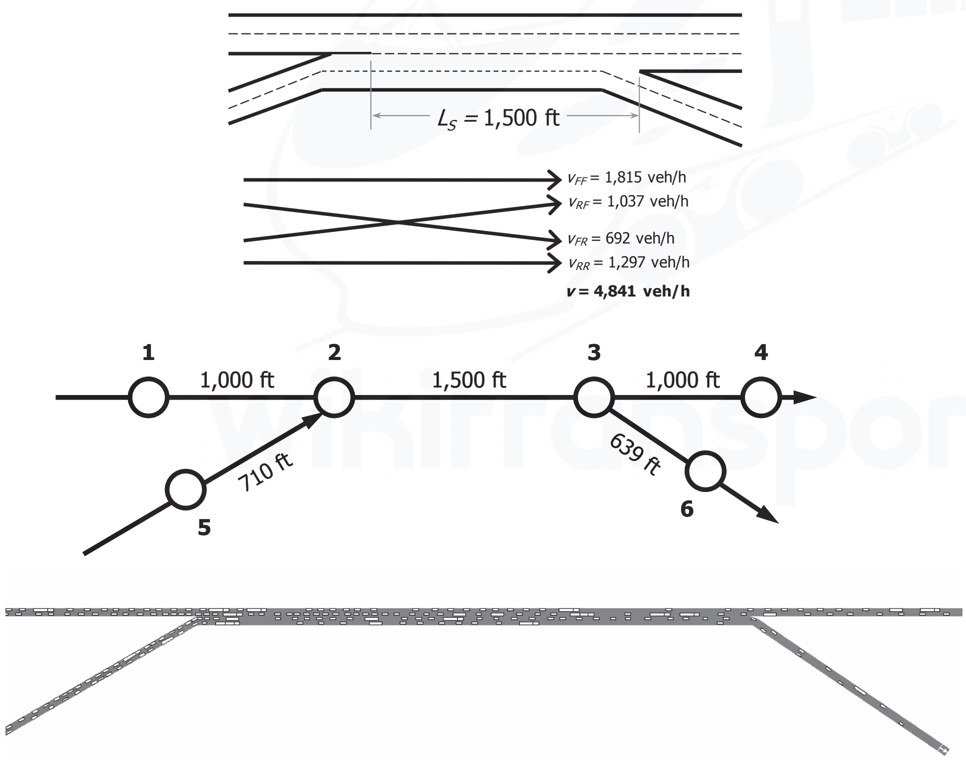
Two cases are examined. The first deals with a weaving segment, and the second deals with merging at an entrance ramp.

Weaving Segment Example

Simulation Network Structure

The problem description, link–node structure, and animated graphics view for the weaving segment example are shown in Exhibit 36-27. The scenario is the same as that used in Example Problem 1 in Chapter 27, Freeway Weaving: Supplemental. There are two lanes on the freeway and on each ramp. The two ramp lanes are connected by full auxiliary lanes.

Exhibit 36-27
Weaving Segment Description and Animated Graphics View



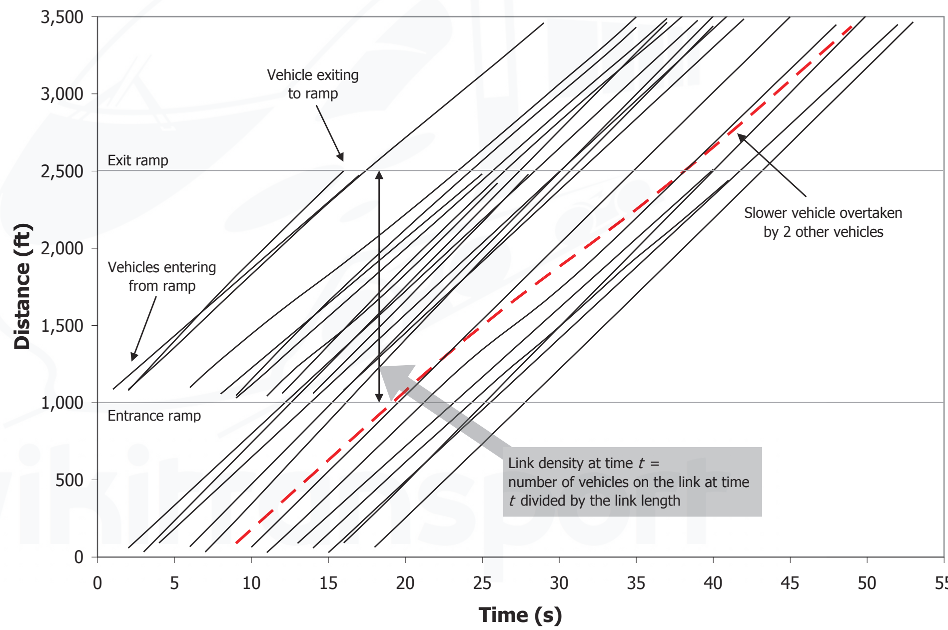
Note: L_S = length of segment, V_{FF} = vehicles entering from freeway and leaving to freeway, V_{RF} = vehicles entering from ramp and leaving to freeway, V_{FR} = vehicles entering from freeway and leaving to ramp, V_{RR} = vehicles entering from ramp and leaving to ramp, veh/h = vehicles per hour.

Vehicle Trajectories for the Freeway Lanes

The vertical (i.e., distance) axis of the trajectory plot provides a linear one-dimensional representation of a series of connected links. The links can follow any pattern as long as some of the vehicles leaving one link flow into the next link. The analysis tool accommodates a maximum of eight connected links. When multiple links are connected to a node (as is usually the case), different combinations of links may be used to construct a multilink trajectory analysis. The route configuration must be designed with the end product in mind. Sometimes multiple routes must be examined to obtain a complete picture of the operation.

There are two entry links and two exit links to the weaving segment, giving four possible routes for analysis. Two routes are examined in this example. The first route, which is represented in Exhibit 36-28, shows the traffic entering the weaving segment from the freeway and leaving to the freeway (V_{FF} in Exhibit 36-27), represented by Links 1–2–3–4. The second route will be examined in the next subsection.

Exhibit 36-28
Trajectory Plot for Freeway Links



In this multilane plot, in contrast to previous plots, some of the trajectory lines might cross each other because of different speeds in different lanes. One such instance is highlighted in Exhibit 36-28. This figure also shows vehicles that enter and leave the weaving segment on the ramps. Because the ramps are not part of the selected route, the ramp vehicles appear on the trajectory plot only on the link that represents the weaving segment. Examples of ramp vehicles are identified in the figure.

The definition of link density (vehicles per mile) is also indicated in Exhibit 36-28. Density as a function of time t is expressed in vehicles per mile and is determined by counting the number of vehicles within the link and dividing by the link length in miles. Average lane density (vehicles per mile per lane) on the link may then be determined by dividing the link density by the number of lanes. To obtain individual lane densities, the trajectory analysis must be performed on each lane. The analysis must also be performed on a per lane basis to examine individual vehicle headways.

Vehicle Trajectories for the Entrance and Exit Ramps

By specifying the links on the route as 5–2–3–6 instead of 1–2–3–4, the trajectories for vehicles entering and leaving the weaving segment on the ramps (V_{RR} in Exhibit 36-27) can be examined. This trajectory plot is shown in Exhibit 36-29. This figure is similar to Exhibit 36-28, except that the vehicles that do not appear outside the weaving segment are those on the freeway links instead of the ramp links.

Two other routes can also be constructed, one for vehicles entering from the freeway and leaving to the exit ramp, V_{FR}, as 1–2–3–6, and one for those entering from the ramp and leaving to the freeway, V_{RF}, as 5–2–3–4. These plots are not included here.

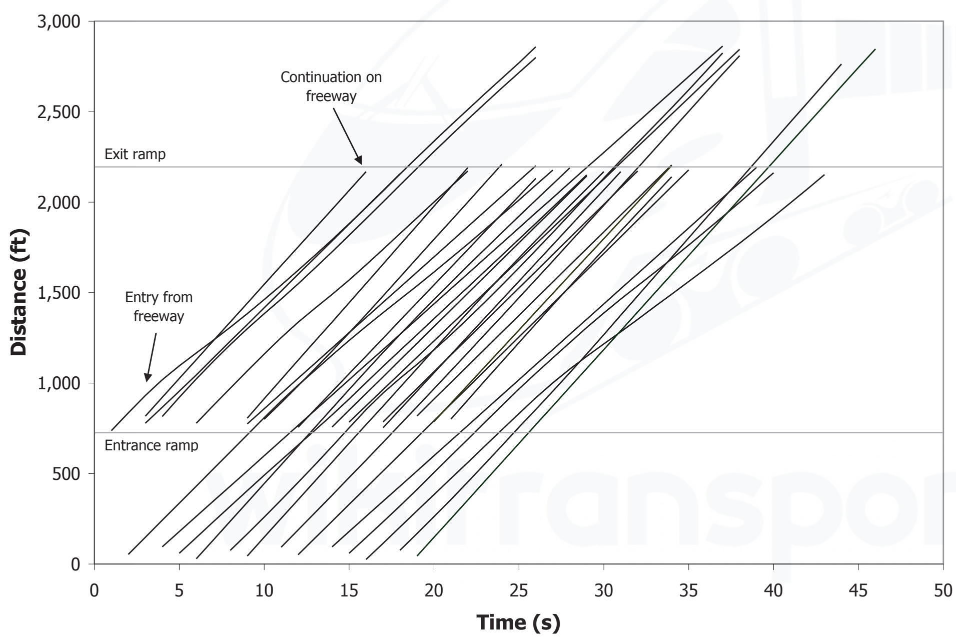


Exhibit 36-29
Trajectory Plot for Entrance and Exit Ramp Links

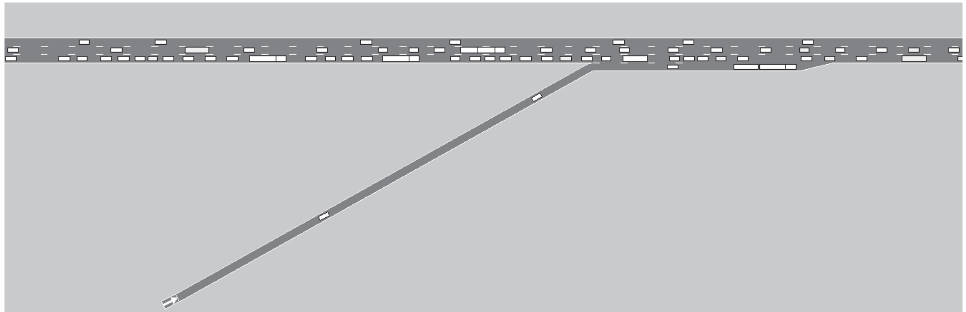
Entrance Ramp Merging Example

Merging segments provide another good example of vehicle trajectory analysis on a freeway. The merging vehicles affect freeway operation differently in each lane, so each lane must be examined independently.

Simulation Network Structure

The same node structure used in the weaving segment example is used here. The lane configuration has been changed to be more representative of a merge operation. Three lanes have been assigned to the freeway and one lane to the entrance ramp. The demand volumes have been specified to provide a near-saturated operation to observe the effects of merging under these conditions. A graphic view of the operation is presented in Exhibit 36-30.

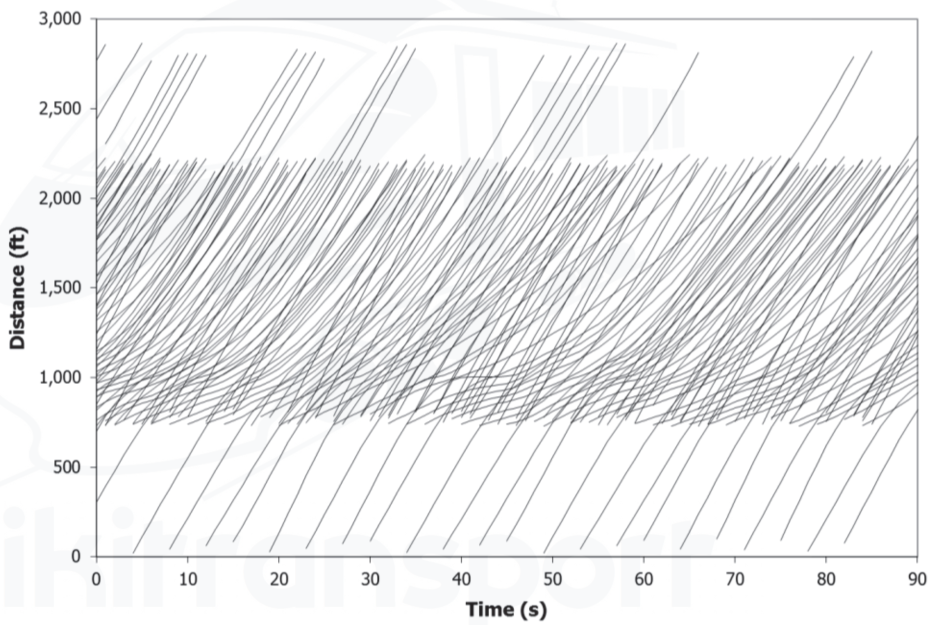
Exhibit 36-30
Entrance Ramp Merging Segment Graphics View



Trajectory Plots for All Lanes

Exhibit 36-31 shows a trajectory plot for all freeway lanes combined within the merge area. The operation is clearly heterogeneous, with a mixture of fast and slow speeds. Many trajectory lines cross each other, and not much can be done in the way of analysis with these data.

Exhibit 36-31
Trajectory Plot for All Freeway Lanes in the Merge Area



Trajectory Plots for Individual Lanes

Clearly, each lane must be examined individually. Exhibit 36-32, Exhibit 36-33, and Exhibit 36-34 show selected trajectories for Lanes 1, 2, and 3, respectively, from a later point in time in the simulation. Because these plots represent individual lanes, the trajectory lines do not cross each other. The effect of the merging operation is observable (and predictable) in these three figures.

In Lane 1, freeway speeds are low upstream of the merge point. Merging vehicles enter the freeway slowly but pick up speed rapidly downstream of the merge point bottleneck. The merging vehicles enter the freeway from the acceleration lane, which begins at 1,000 ft on the distance scale. The merging vehicle trajectories before entry onto the freeway are not shown in Exhibit 36-32 because those vehicles are either on a different link or in a different lane.

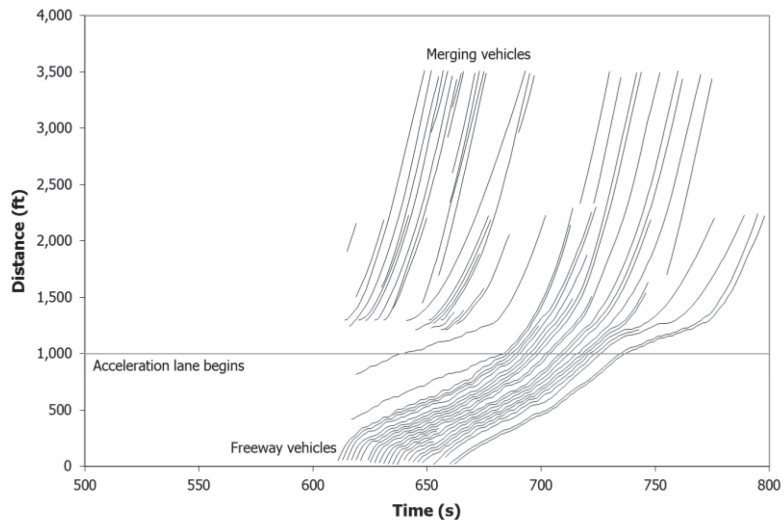


Exhibit 36-32
Trajectory Plot for Freeway Lane 1 (Rightmost) in the Merge Area

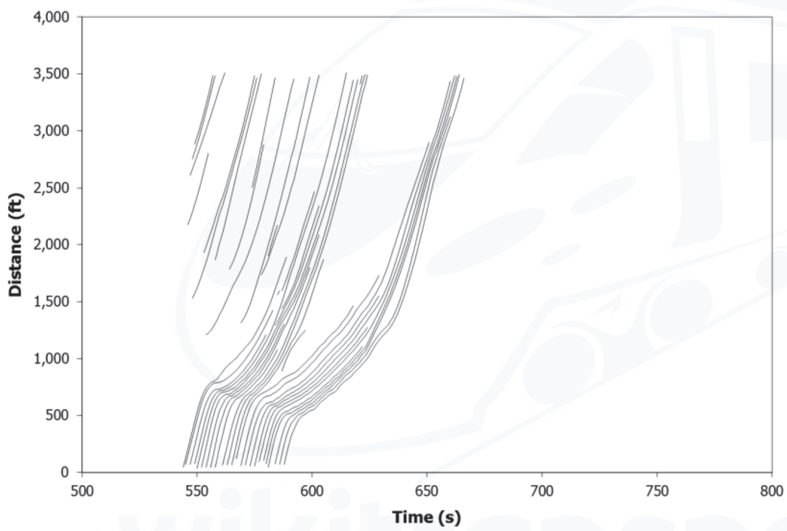


Exhibit 36-33
Trajectory Plot for Freeway Lane 2 (Center) in the Merge Area

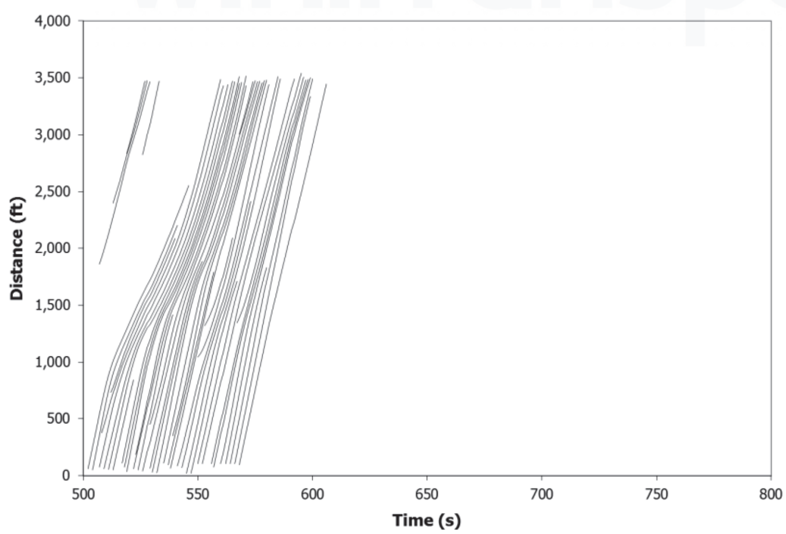


Exhibit 36-34
Trajectory Plot for Freeway Lane 3 (Leftmost) in the Merge Area

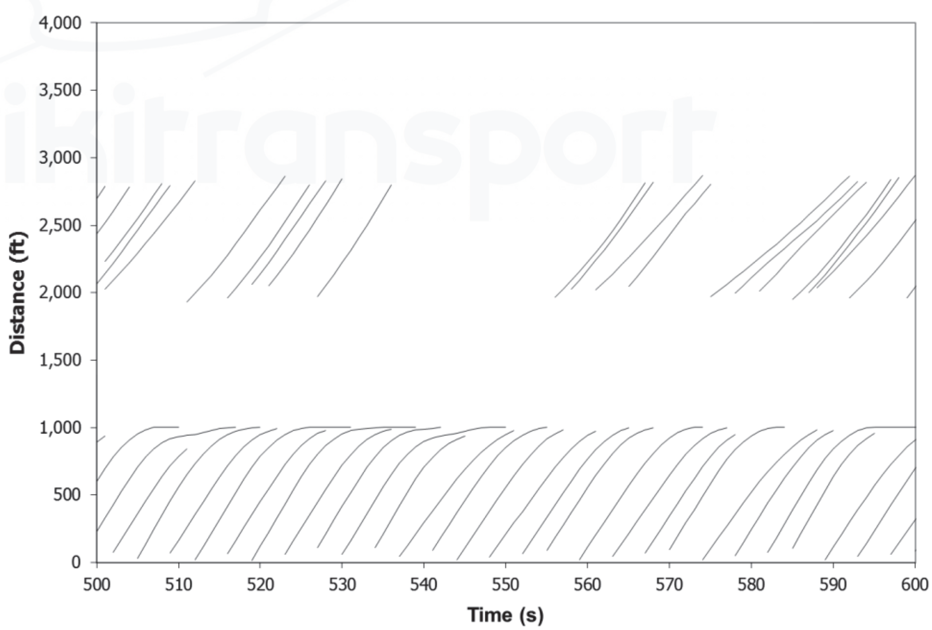
In Lane 2, the freeway speeds are higher but still well below the FFS, indicating that the merge operation affects the second lane as well. Some vehicles enter Lane 2 in the vicinity of the acceleration lane, but they are generally vehicles that have left Lane 1 to avoid the friction. Both Lane 1 and Lane 2 show several discontinuous trajectories that indicate lane changes. The Lane 3 operation is much more homogeneous and speeds are higher, indicating a much smaller effect of the merging operation.

Trajectory Plots for Ramp Vehicles

To configure a trajectory route covering the entrance ramp vehicles, the ramp and acceleration lane, which were not represented in Exhibit 36-32 through Exhibit 36-34, must be selected in place of the upstream freeway link. The acceleration lane number must first be identified from the simulation tool's output. Because of the selected tool's unique and somewhat creative lane numbering scheme, the acceleration lane will be Lane 9. To cover both the ramp and the acceleration lane, Lane 9 must be selected on the freeway link (2–3).

The trajectory plot for this route is shown in Exhibit 36-35. The results are not what might be anticipated. Vehicles are observed on the ramp and in the acceleration lane, but they disappear as soon as they enter the freeway. More vehicles eventually appear toward the end of the freeway link. The vehicles disappear because Lane 9 was selected for the freeway link, so vehicles in Lane 1 do not show up on the plot. The vehicles that reappear at the end of the link are those leaving the freeway at the downstream exit. They reappear at that point because the deceleration lane at the end of the link is also assigned as Lane 9. This plot is not particularly useful, except that it illustrates the complexities of trajectory analysis.

Exhibit 36-35
Trajectory Plot for Acceleration and Deceleration Lanes



To obtain a continuous plot of ramp vehicles, nodes must be added to the network at the points where the acceleration and deceleration lanes join the freeway. These nodes are shown as Nodes 7 and 8 in Exhibit 36-36. A continuous route may then be configured as 5–2–7–8–3–4. Selected trajectories from the trajectory plot for this route are shown in Exhibit 36-37. This plot shows the entering vehicles on the ramp as they pass through the acceleration lane onto the freeway. There are some discontinuities in the trajectories because of the different point at which vehicles leave the acceleration lane.

Exhibit 36-36
Addition of Intermediate Nodes for Continuous Trajectory Plots

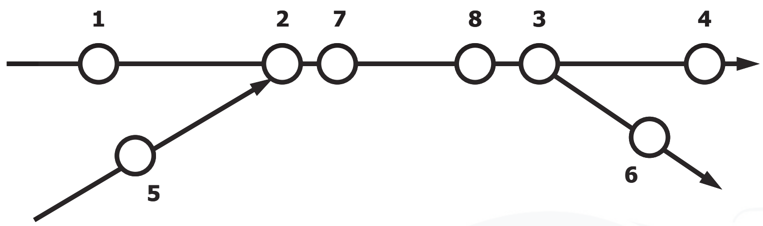
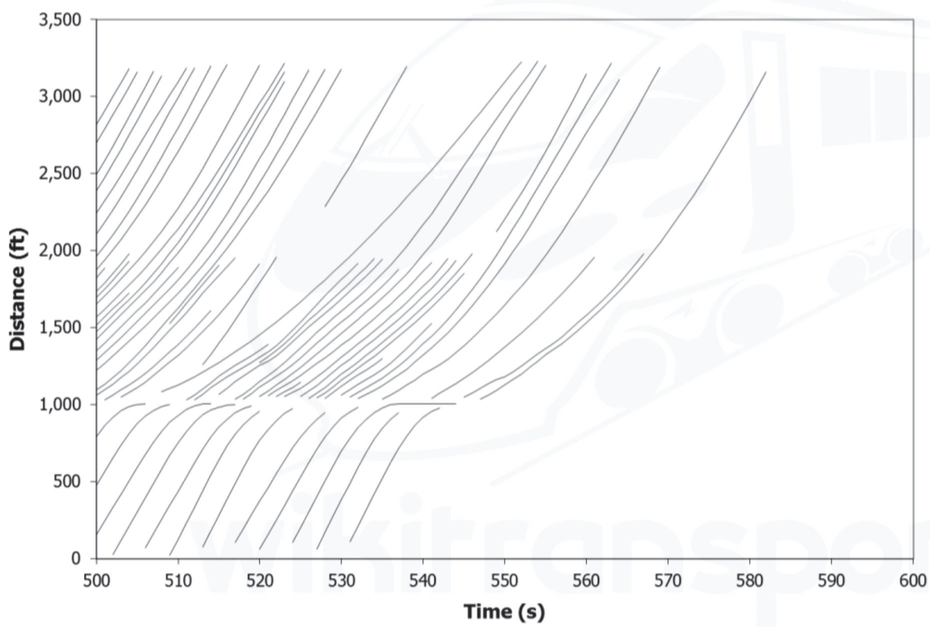


Exhibit 36-37
Trajectory Plot for Acceleration Lane and Freeway Lane 1



ESTIMATING PERFORMANCE MEASURES FROM VEHICLE TRAJECTORY DATA

The preceding subsections demonstrated that the production of vehicle trajectory plots that can be interpreted and analyzed is possible. This subsection focuses on computation of the performance measures from a mathematical analysis of the data represented in these plots.

Trajectory Analysis Procedures Overview

One development goal for the HCM 2010 was the creation of a set of computational procedures by which developers of simulation tools could produce performance measures that are consistent among different tools and, to the extent possible, compatible with the HCM's deterministic procedures. The procedures presented here were designed to be implemented easily by using the common trajectory properties described previously and illustrated by examples. Developers of simulation tools are encouraged to implement these procedures, and users of simulation tools are encouraged to consider the extent to which the procedures have been implemented in the traffic analysis tool selection process described in Chapter 6, HCM and Alternative Analysis Tools.

Requirements for Trajectory Analysis Algorithm Development

A basic set of guidelines for computing uniform performance measures from vehicle trajectory analysis was introduced in Chapter 7, Interpreting HCM and Alternative Tool Results. Since these requirements are also incorporated into the specific computational procedures proposed in this chapter, they are repeated here to promote a better understanding of the procedures. The general guidelines suggested in Chapter 7 include the following:

1. The trajectory analysis procedures are limited to analysis of trajectories produced by the traffic flow model of each simulation tool. The nature of the procedures must not suggest the need for developers to change their driver behavior or traffic flow modeling logic.
2. If the procedures for estimating a particular measure cannot be satisfactorily defined to permit a valid comparison between the HCM and other modeling approaches, such comparisons should not be made.
3. All performance measures that accrue over time and space should be assigned to the link and time interval in which they occur. Subtle complexities make it impractical to do otherwise. For example, the root cause of a specific delay might not be within the link or the immediate downstream link. The delay might be secondary to a problem at some distant location in the network and in a different time interval.
4. The spatial and temporal boundaries of the analysis domain must include a period that is free of congestion on all sides. This principle is also stated in Chapter 10, Freeway Facilities Core Methodology, and in Chapter 19, Signalized Intersections, for multiperiod signalized intersection analysis. To ensure that delays to vehicles denied entry to the system during a given period are properly recognized, creation of fictitious links outside the physical network to hold such vehicles might be necessary. A more detailed discussion of spatial and temporal boundaries is provided in Chapter 7.
5. It is important to ensure that the network has been properly initialized or "seeded" before trajectory analysis is performed. When the warm-up periods are set and applied, simulation tools typically start with an empty network and introduce vehicles until the vehicular content of the network stabilizes. Trajectory analysis should not begin until stability has been achieved. If the simulation period begins with oversaturated conditions, stability may never be achieved. See the discussion in Chapter 7 on temporal and spatial boundaries.

In addition to the general guidelines, some requirements must be addressed here to promote the development of trajectory analysis procedures that can be applied in a practical manner by the developers of simulation tools. The following requirements are suggested:

1. The algorithms must be suitable for computation "on the fly." They must not require information from a future time step that would complicate the data handling within the simulation process.
2. Arbitrary thresholds for determining parameters should be kept to a minimum because of the difficulty of obtaining acceptance throughout the user community for specific thresholds. When arbitrary thresholds cannot be avoided, they should be justified to the extent possible by definitions in the literature, and above all, they should be applied consistently for different types of analysis.
3. Computationally complex and time-consuming methods should be avoided to minimize the additional load on the model. Methods should be developed to simplify situations with many special cases because of the difficulty of enumerating all special cases.
4. The same definitions, thresholds, and logic should be used for determination of similar parameters in different computational algorithms for longitudinal and spatial analysis.

Summary of Computational Procedures

Several performance measures were examined in Chapter 7, and general guidelines for comparing measures produced by different tools were presented. Previous material in this section has demonstrated the potential for development of uniform measures by individual vehicle trajectory analysis and has proposed some requirements for development of the analysis procedures. Specific procedures for analyzing vehicle trajectories are now presented and demonstrated with additional examples.

Thresholds for Computation of Performance Measures

Elimination of arbitrary and user-specified values is an important element of standardization. Avoidance of arbitrary thresholds was identified earlier as a requirement for the development of trajectory analysis procedures. Avoidance of all arbitrary thresholds is desirable. If thresholds cannot be avoided, they should be justified in terms of the literature. When no such justification exists, they should at least be established on the basis of consensus and applied consistently. The following thresholds cannot be avoided in vehicle trajectory analysis.

Car Length

The following is stated in Chapter 31, Signalized Intersections: Supplemental:

> A vehicle is considered as having joined the queue when it approaches within one car length of a stopped vehicle and is itself about to stop. This definition is used because of the difficulty of keeping track of the moment when a vehicle comes to a stop.

So, for estimation of queue-related measures, a value that represents one car length must be chosen. For the purposes of this section, a value of 20 ft is used.

Stopped-Vehicle State

One example of an arbitrary threshold is the speed at which a vehicle is considered to have come to a stop. Several arbitrary thresholds have been applied for this purpose. To maintain consistency with the definition of the stopped state applied in other chapters of the HCM, a speed less than 5 mi/h is used here for determining when a vehicle has stopped.

Moving-Vehicle States

Other states in addition to the stopped state that must be defined consistently for vehicle trajectory analysis include the following:

- The uncongested state, in which a vehicle is moving in a traffic stream that is operating below its capacity;
- The congested state, in which the traffic stream has reached a point that is at or slightly above its capacity, but no queuing from downstream bottlenecks is present; and
- The severely constrained state, in which downstream bottlenecks have affected the operation.

These states apply primarily to uninterrupted flow. A precise definition would require complex modeling algorithms involving capacity computations or “look ahead” features, both of which would create a computational burden. Therefore, an easily applied approximation must be sought. Threshold speeds are a good candidate for such an approximation.

These states can be thought of conveniently in terms of speed ranges. To avoid specifying arbitrary speeds as absolute values, use of the target speed of each vehicle as a reference is preferable. The target speed is the speed at which the driver prefers to travel. It differs from the FFS in the sense that most simulation tools apply a “driver aggressiveness” factor to the FFS to determine the target speed. In the absence of accepted criteria, three equal speed ranges are applied for the purposes of this section. Thus, the operation is defined as uncongested if the speed is above two-thirds of the target speed. It is defined as severely constrained when the speed is below one-third of the target speed, and it is considered congested in the middle speed range. This stratification is used to produce performance measures directly (e.g., percent of time severely constrained). It is also used in computing other performance measures (e.g., release from a queue).

Computational Procedures for Stop-Related Measures

The two main stop-related measures are number of stops and stopped delay. The beginning of a stop is defined in the same way for both measures. The end of a stop is treated differently for stopped delay and number of stops. For stopped delay, the end of a stop is established as soon as the vehicle starts to move (i.e., its speed reaches 5 mi/h or greater). For determining the number of stops, some hysteresis is required. For purposes of this section, after a vehicle is stopped a subsequent stop is not recognized until it leaves the severely constrained state (i.e., its speed reaches one-third of the target speed).

Because subsequent stops are generally made from a lower speed, they can be expected to have a smaller impact on driver perception, operating costs, and safety. Recognizing this fact, the National Cooperative Highway Research Program (NCHRP) 03-85 project proposed a "proportional stop" concept (*11*), in which the proportion of a subsequent stop is based on the relative kinetic energy loss and is therefore proportional to the square of the speed from which the stop was made. Thus, each time a vehicle speed drops below 5 mi/h, the number of stops is incremented by $(S_{max}/S_{\text{target}})^2$, where S_{max} is the maximum speed attained since the last stop and S_{target} is the target speed.

This procedure has not been applied in practice. It is mentioned here because it offers an interesting possibility for the use of simulation to produce measures that could be obtained in the field but could not be estimated by the macroscopic deterministic models described in the HCM. The procedure is illustrated by an example later in this section.

Computational Procedures for Delay-Related Measures

The procedures for computing delay from vehicle trajectories involve aggregating all delay measures over each time step. Therefore, the results take the form of aggregated delay and not unit delay, as defined in Chapter 7. To determine unit delays, the aggregated delays must be divided by the number of vehicles involved in the aggregation. Partial trips made over a segment during the time period add some complexity to the unit delay computations.

The following procedures should be used to compute the various delay-related measures from vehicle trajectories:

- *Time step delay*: The delay on any time step is, by definition, the length of the time step minus the time the vehicle would have taken to cover the distance traveled in the step at the target speed. This value is easily determined and is the basis for the remainder of the delay computations.
- *Segment delay*: Segment delay is the time actually taken to traverse a segment minus the time that would have been taken to traverse the segment at the target speed. The segment delay on any step is equal to the time step delay. Segment delays accumulated over all time steps in which a vehicle is present on the segment represent the segment delay for that vehicle.
- *Queue delay*: Queue delay is equal to the time step delay on any step in which the vehicle is in a queued state; otherwise, it is zero. Queue delays are accumulated over all time steps while the vehicle is in a queue.
- *Stopped delay:* Stopped delay is equal to the time step delay on any step in which the vehicle is in a stopped state; otherwise, it is zero. Because a vehicle is considered to be "stopped" if it is traveling at less than a threshold speed, a consistent definition of stopped delay requires that the travel time at the target speed be subtracted. Time step delays

accumulated over all time steps in which the vehicle was in the stopped state represent the stopped delay.

- *Control delay*: Control delay is the additional travel time caused by operation of a traffic control device. It cannot be computed directly from the vehicle trajectories in a manner consistent with the procedures given in Chapters 19 and 31 for signalized intersection analysis. However, it is an important measure because it is the basis for determining the level of service on a signalized approach.

Queue delay computed from trajectory analysis provides the most appropriate representation of control delay.

The queue delay computed from vehicle trajectories provides a reasonable approximation of control delay when the following conditions are met:

1. The queue delay is caused by a traffic control device, and
2. The identification of the queued state is consistent with the definitions provided in this section.

Computational Procedures for Queue-Related Measures

Procedures for computing queue-related measures begin with determining whether each vehicle in a segment is in a queued state. A vehicle is in a queued state if it has entered a queue and has not yet left it. The beginning of a queued state occurs when

- The gap between a vehicle and its leader is less than or equal to 20 ft,
- The vehicle speed is greater than or equal to the leader speed, and
- The vehicle speed is less than or equal to one-third of the target speed (i.e., the speed is severely constrained).

A separate case must be created to accommodate the first vehicle to arrive at the stop line. If the link is controlled (interrupted-flow case), the beginning of the queued state also occurs when

- No leader is present on the link,
- The vehicle is within 50 ft of the stop line, and
- The vehicle is decelerating or has stopped.

These rules have been found to cover all the conditions encountered.

The ending of the queued state also requires some rules. For most purposes, the vehicle should be considered to remain in the queue until it leaves the link. The analysis is done on a link-by-link basis. In the case of queues that extend over multiple links, a vehicle leaving a link immediately enters the queue on the next link. Experience with trajectory analysis has shown that other conditions need to be applied to supplement this rule. Thus, the end of the queued state also occurs when

- The vehicle has reached two-thirds of the target speed (i.e., uncongested operation), and
- The leader speed is greater than or equal to the vehicle speed or the vehicle has no leader in the same link.

The additional conditions cover situations in which, for example, a vehicle escapes a queue by changing lanes into an uncongested lane (e.g., through vehicle caught temporarily in a turn bay overflow).

Chapters 19 and 31 offer the following guidance on estimating queue length:

1. The maximum queue reach (i.e., back of queue, or BOQ) is a more useful measure than the number of vehicles in the queue, because the BOQ causes blockage of lanes. The maximum BOQ is reached when the queue has almost dissipated (i.e., has zero vehicles remaining).
2. A procedure is prescribed to estimate average maximum BOQ on a signalized approach.

Because of its macroscopic nature, the HCM queue estimation procedure cannot be applied directly to simulation. On the other hand, simulation can produce additional useful measures because of its higher level of detail. The first step in queue length determination has already been dealt with by setting up the rules for determining the conditions that indicate when a vehicle is in a queue. The next step is to determine the position of the last vehicle in the queue.

The BOQ at any time step will be determined by the position of the last queued vehicle on the link plus the length of that vehicle.

The BOQ on any step is a relatively simple thing to determine. The trick is to figure out how to accumulate the individual BOQ measures over the entire period. Several measures can be produced.

1. The maximum BOQ at some percentile value—for example, 95%;
2. The maximum BOQ on any cycle at some percentile value—for example, 95%;
3. The historical maximum BOQ (i.e., the longest queue recorded during the period);
4. The probability that a queue will back up beyond a specified point; and
5. The proportion of time that the queue will be backed up beyond a specified point.

Some of these measures are illustrated later in an example.

Computational Procedures for Density-Related Measures

The uninterrupted-flow procedures described in the HCM base their LOS estimates on the density of traffic in terms of passenger cars per mile per lane (pc/mi/ln). In one case (freeway merges and diverges), the density is estimated only for the two lanes adjacent to the ramp.

Density computations do not require a detailed analysis of the trajectory of each vehicle. They are best made by simply counting the number of vehicles in each lane on a given segment, recognizing that the results represent actual vehicles and not passenger cars.

For comparable results, the simulated densities must be converted to pc/mi/ln, especially if simulation tools are used to evaluate the LOS on a segment. Because the effect of heavy vehicles on the flow of traffic is treated microscopically, there is no notion of passenger car equivalence in simulation modeling. In addition, traffic flow models may differ among the various simulation tools in their detailed treatment of heavy vehicles. Therefore, a simple conversion process that will ensure full compatibility with the HCM's LOS estimation procedures cannot be prescribed. One possible method for developing passenger car equivalence conversion factors involves multiple simulation runs:

1. Use the known demand flow rates, v, and truck proportions to obtain the resulting segment density in vehicles per mile per lane (veh/mi/ln), d_1.
2. Use the known demand flow rates, v, with passenger cars only to obtain the resulting segment density in veh/mi/ln, d_2.
3. Determine the heavy vehicle equivalence factor as $f_{HV} = d_2/d_1$.
4. Set the demand flow rates to v/f_{HV} with passenger cars only to obtain the resulting segment density in pc/mi/ln.

This process is more precise because it adheres to the definition of passenger car equivalence. Unfortunately, it is too complicated to be of much practical value. However, two methods could produce a more practical approximation. Both require determining the heavy vehicle adjustment factor, f_{HV}, by the method prescribed in Chapter 12 for basic freeway segments. This method is also referenced and used in the procedural chapters covering other types of freeway segments. The simplest approximation may be obtained by running the simulation with known demand flow rates and truck proportions and then dividing the simulated density by f_{HV}. Another approximation involves dividing the demand flow rates by f_{HV} before running the simulation with passenger cars only. The resulting densities are then expressed in pc/mi/ln. The second method conforms better to the procedures prescribed in Chapters 11 to 13, but the first method is probably easier to apply.

Follower density is an emerging density-based measure for two-lane highways (*12*, *13*). It is defined as the number of followers per mile per lane. A vehicle can be classified as following when

- The gap between the rear and the front ends of the leading and following vehicles, respectively, are shorter than or equal to 3 s; and
- The speed of the following vehicle is not more than 12 mi/h lower than that of the preceding vehicle.

The follower density can be derived from point measurements by means of the following formula:

Equation 36-1

$$\text{Follower density} = \%\ \text{followers} \times \text{flow rate} / \text{time mean speed}$$

Although this performance measure is not computed by the procedures in the HCM, it is mentioned because it has attracted significant international interest and can easily be computed by vehicle trajectory analysis.

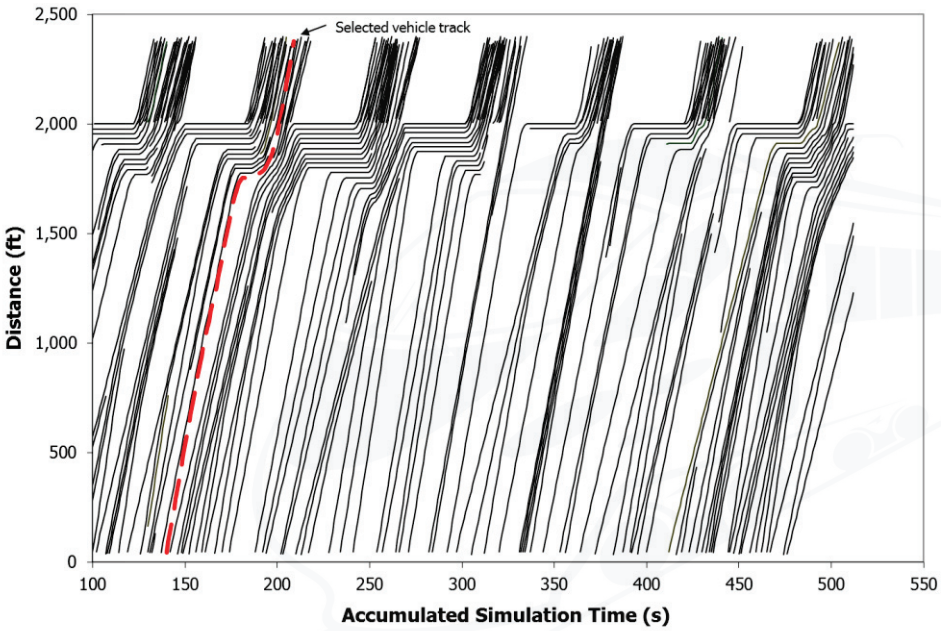
Analysis of a Signalized Approach

The simple approach to a signalized intersection (Exhibit 36-20) is now converted to a two-lane approach with a length of 2,000 ft. A 10-min (600-s) analysis period is used. The cycle length is 60 s, giving 10 cycles for inspection. The analysis period would normally be longer, but 10 min is adequate for demonstration purposes.

Trajectory Plots

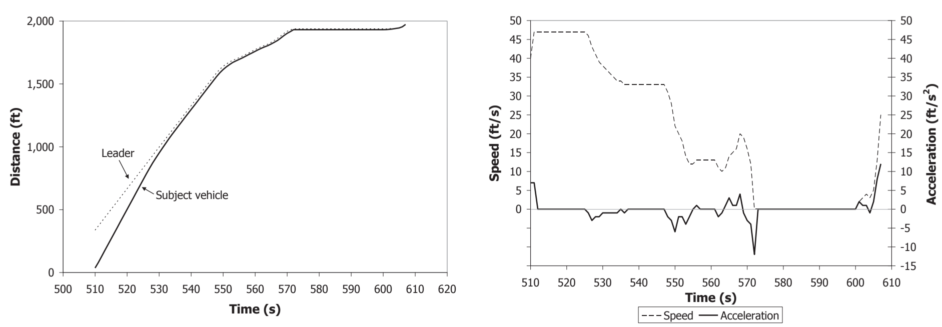
The trajectory plot for the first few cycles is shown in Exhibit 36-38. The vehicle track selected for later analysis is also shown in this exhibit.

Exhibit 36-38
Trajectories for Several Cycles on a Signalized Approach



Two individual trajectory analysis plots are shown in Exhibit 36-39. The first plot shows the trajectories of two vehicles where the progress of the subject vehicle is constrained by its leader. The second plot shows the speed and acceleration profiles for the subject vehicle.

Exhibit 36-39
Example Trajectory Analysis Plots



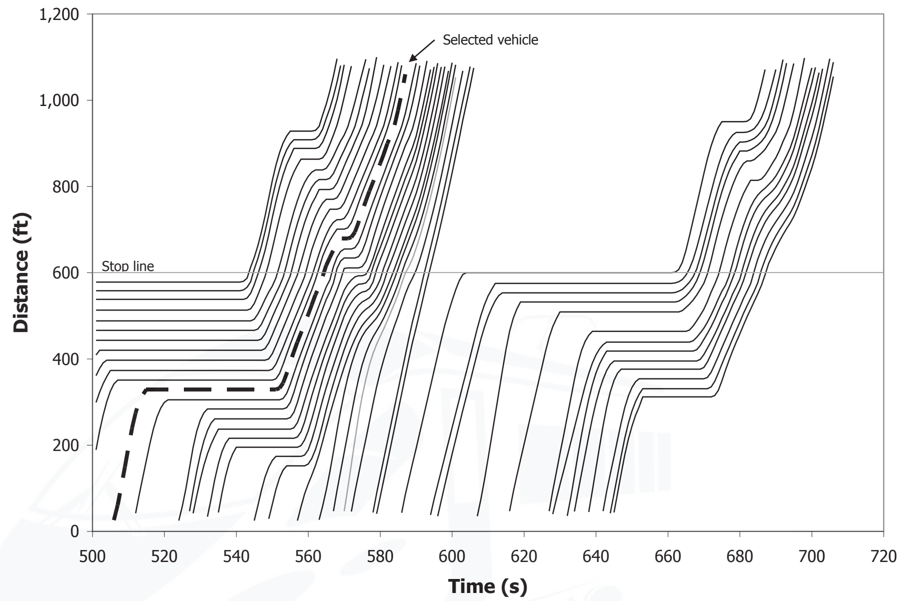
(a) Subject Vehicle and Leader Vehicle Trajectories

(b) Speed and Acceleration Profile of Subject Vehicle

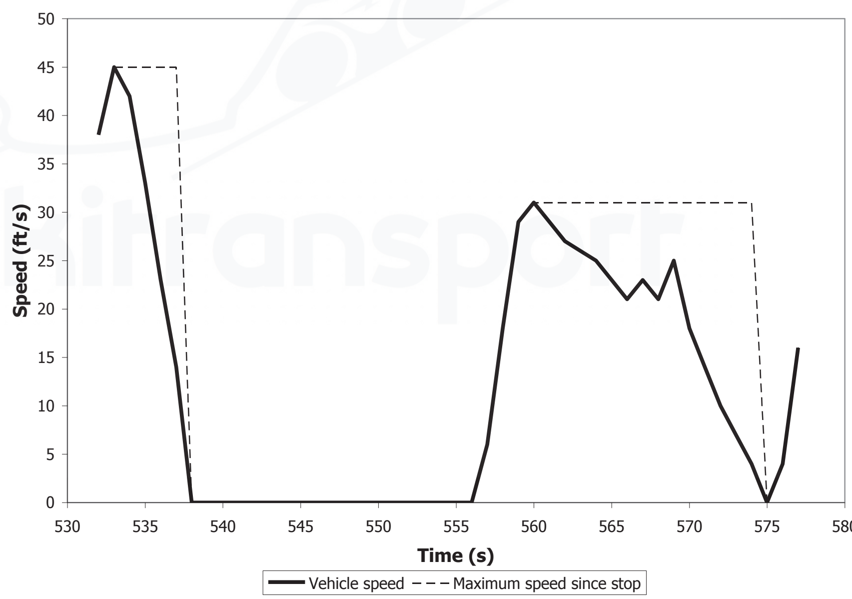
Analysis of Stops

An example of the analysis of a single vehicle selected from the entire trajectory plot is shown in Exhibit 36-40. With the definition of a partial stop based on the NCHRP 03-85 kinetic energy loss concept, the total stop value was 1.81 because the second stop was made from a lower speed.

Exhibit 36-40
Analysis of a Full and a Partial Stop



(a) Vehicle Trajectories



(b) Selected Vehicle Speed

Segment delay	Queue delay	Stop delay	Number of stops
34.64 s	33.23 s	20 s	1.81

(c) Performance Measures for Selected Vehicle

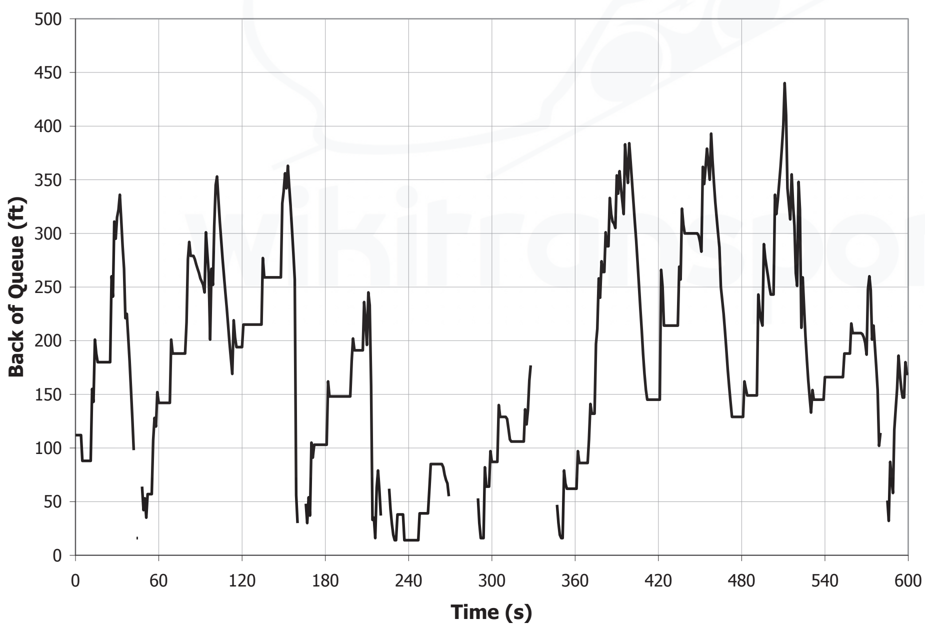
Queuing Analysis

Exhibit 36-41(a) illustrates the queue length (BOQ) per step for one lane of the signalized approach over all the time steps in the period. The 10 cycles are discernible in this figure. Also, a considerable variation in the cyclical maximum BOQ is evident.

The percentile instantaneous BOQ and the percentile maximum BOQ per cycle should be distinguished. For the instantaneous BOQ, the individual observation is the BOQ on any step, so the sample size is the number of steps covered (600 in this case). For cyclical maximum BOQ, the individual observation is the maximum BOQ in any cycle, so the sample size is the number of cycles (10 in this case). The maximum BOQ in any cycle can be determined only by inspecting the plotted instantaneous values. No procedure is proposed here for automatic extraction of the maximum cyclical BOQ from the instantaneous BOQ data.

A statistical analysis showing the average BOQ, the 95th percentile BOQ (based on 2 standard deviations past the average value), and the historical maximum BOQ is presented in Exhibit 36-41(b). One important question is whether the 95% BOQ can be represented statistically on the basis of the standard deviation, assuming a normal distribution. The BOQ histogram showing the distribution of instantaneous BOQ for the 600 observations is shown in Exhibit 36-42. The appearance of this histogram does not suggest any analytical distribution; however, the relationship between the 95% BOQ and the historical maximum appears to be reasonable for this example.

Exhibit 36-41
BOQ Analysis by Time Step

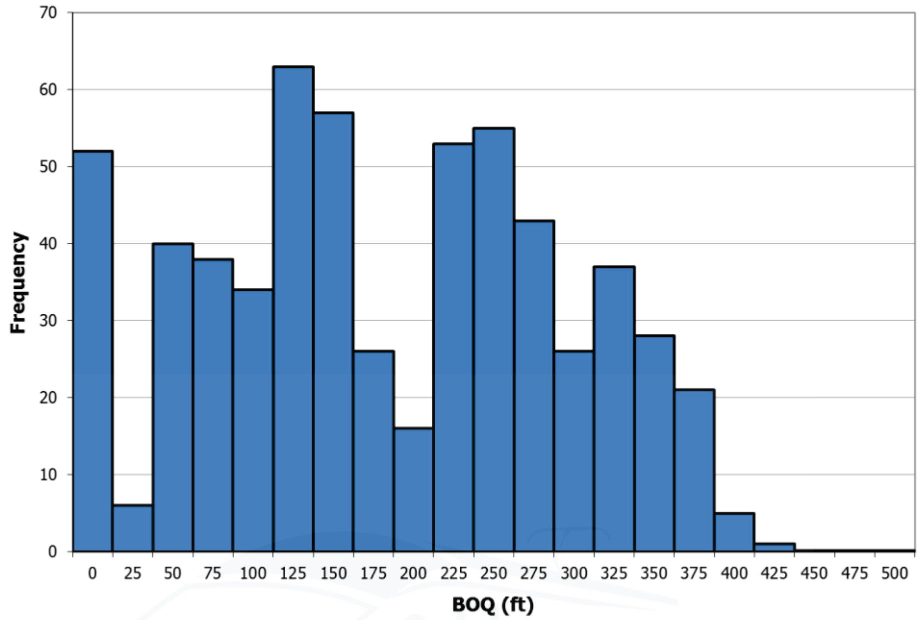


(a) BOQ Plot

Average queue	Standard deviation	95th percentile queue	Maximum queue
174 ft	110 ft	395 ft	440 ft

(b) Queue-Related Performance Measures

Exhibit 36-42
BOQ Histogram



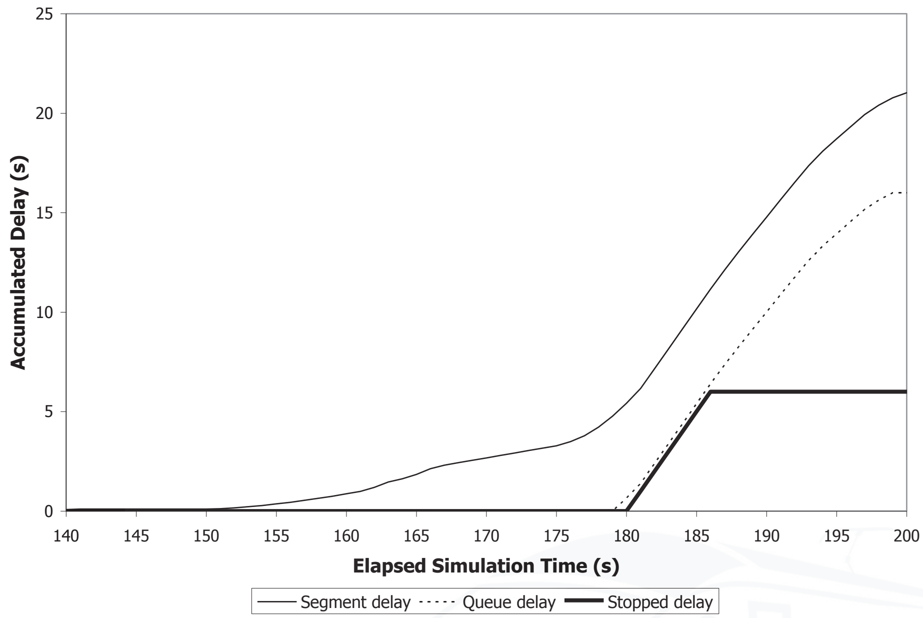
The queue length on an isolated approach that is close to saturation will have a near uniform distribution (i.e., equal probability of all lengths between zero and the maximum). The standard deviation of a uniform distribution is greater than one-half of the mean, so the 95th percentile estimator (mean value plus 2 standard deviations) will be greater than the maximum value. This situation raises some doubt about the validity of basing the 95th percentile BOQ on the standard deviation, especially with cyclical queuing.

Delay Analysis for a Single Trajectory

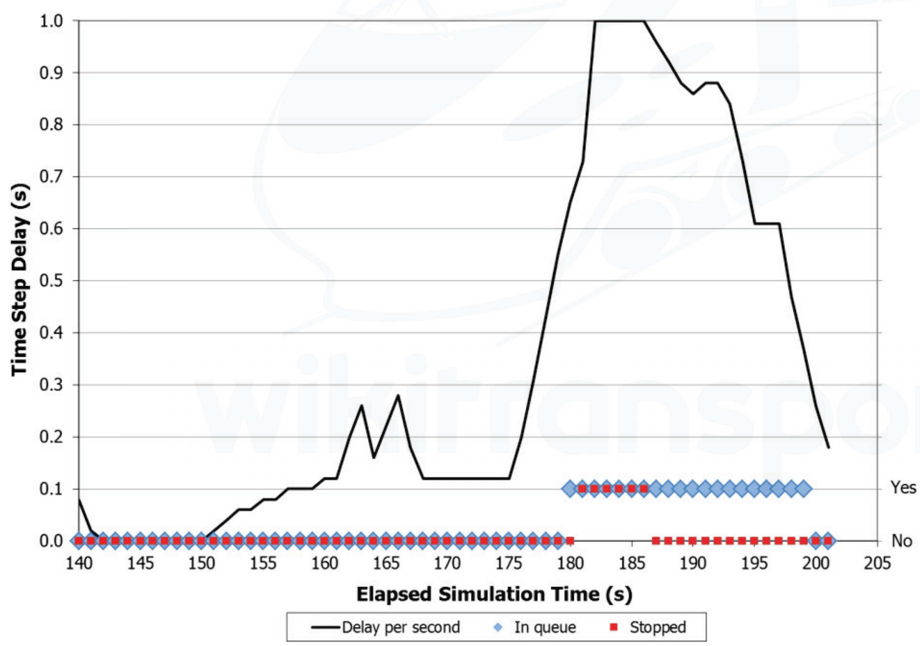
A comparison of the accumulated delay by all definitions for the selected vehicle track indicated in Exhibit 36-38 is presented in Exhibit 36-43(a). The relationships between segment delay, queue delay, and stopped delay are evident in this figure. The segment delay begins to accumulate before the vehicle approaches the intersection because of midsegment interactions that reduce the speed below the target speed. The queue delay begins to accumulate as the vehicle enters the queue, and the stopped delay begins to accumulate a few seconds later. The stopped delay ceases to accumulate as soon as the vehicle starts to move, but the queue delay continues to accumulate until the vehicle leaves the link.

The time step delay analysis plots shown in Exhibit 36-43(b), based on 1-s time steps, provide additional insight into the operation. The time step delay is close to zero as the vehicle enters the segment, indicating that the speed is close to the target speed. Small delays begin to accumulate in advance of the intersection. The accumulation becomes more rapid when the vehicle enters the queue. The periods when the vehicle is in the stopped and the queued state are also shown in this figure.

Exhibit 36-43
Accumulated Delay by Various Definitions



(a) Accumulated Delay



(b) Time Step Delay

As was indicated previously, the value of control delay cannot be determined by simulation in a manner that is comparable with the procedures prescribed in Chapters 19 and 31. Because this segment terminates at a signal, it is suggested that the queue delay would provide a reasonable estimate of control delay because the queue delay offers a close approximation to the delay that would be measured in the field.

Delay Analysis for All Vehicles on the Segment

The preceding example dealt with accumulated delay of a single vehicle traversing the segment. A useful delay measure requires the accumulation of delay to all vehicles traversing the segment during the period. An example is shown in Exhibit 36-44. In keeping with the recommendations offered elsewhere (*14*), only vehicles that traversed the entire link during the period are included in this analysis. Therefore, the number of vehicles analyzed (210) is lower than the number of vehicles that were actually on the link during the period (286).

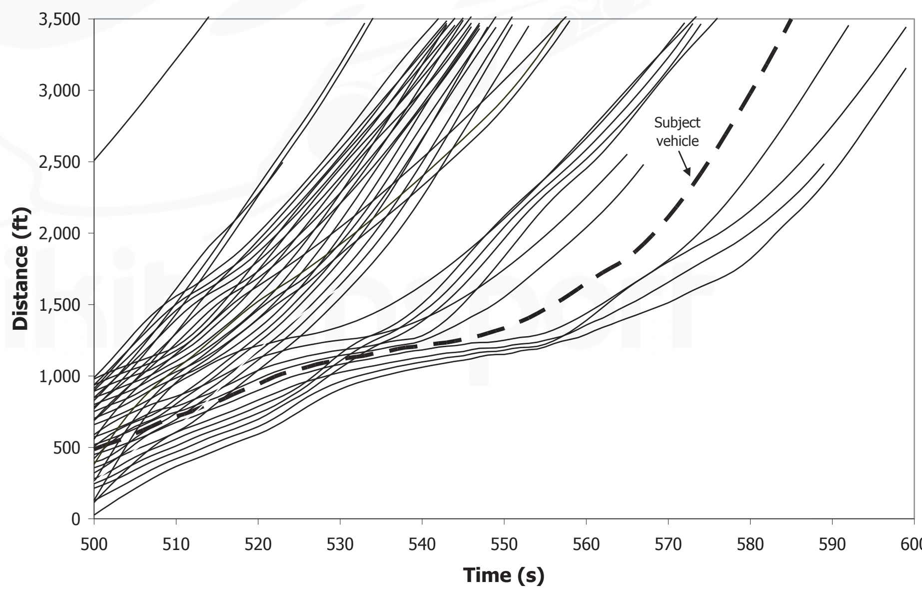
Exhibit 36-44
Delay Analysis for All Vehicles on a Segment

	Segment Delay (s)	Queue Delay (s)	Stop Delay (s)	No. of Stops
Lane 1	3,128	2,562	1,957	95.4
Lane 2	3,400	2,793	2,047	96.2
Total	6,529	5,355	4,004	191.6
Average per vehicle	31.09	25.50	19.07	0.91

Analysis of a Freeway Segment

A performance analysis of the freeway weaving area originally shown in Exhibit 36-27 is presented here. A single vehicle is selected from the trajectory plot and its trajectory is analyzed. The results are shown in Exhibit 36-45. The analysis produced segment delay and queue delay. This segment was very congested, as indicated by the trajectory plot. No stopped delay was produced because the vehicle never actually came to a stop (i.e., its speed stayed above 5 mi/h).

Exhibit 36-45
Longitudinal Analysis of Delay for a Selected Vehicle in a Weaving Area



(a) Vehicle Trajectories

Segment delay	Queue delay	Stopped delay
39.58 s	37.01 s	0 s

(b) Delay-Related Performance Measures for Subject Vehicle

A spatial analysis of the entire segment can also be performed to produce the following measures by lane:

- Average density over the segment,
- Percent slow vehicles (i.e., traveling at less than two-thirds the target speed),
- Percent queued vehicles,
- Average queue length (measured from front of queue to BOQ),
- Average BOQ position,
- Maximum BOQ position, and
- Percent of time steps when the queue overflowed the segment.

The results are presented in tabular form in Exhibit 36-46. The values are presented by lane, and the exhibit note presents combined density values for Lanes 1 and 2 for compatibility with the HCM definition of merge area density.

Exhibit 36-46
Example Spatial Analysis by Lane

	Lane 1	**Lane 2**	**Lane 3**	**Acceleration Lane**
Average density (veh/mi/ln)	73.4	51.0	43.6	9.9
Percent slow vehicles (%)	88.4	68.5	41.5	65.7
Percent queued vehicles (%)	63.4	22.0	2.4	26.7
Average queue length (ft)	600	215	15	40
Average back of queue (ft)	1,471	1,119	135	562
Maximum back of queue (ft)	1,497	1,497	1,492	1,474
Percent overflow	66.1	29.6	0.5	0.17

Note: Average Lane 1 and Lane 2 density is 62.2 veh/mi/ln.

6. SUMMARY OF CHANGES FROM HCM2000 TO HCM 2010

INTRODUCTION

This section documents the major research projects that contributed to the previous edition of the manual, the HCM 2010. The What's New in the HCM Sixth Edition section of Chapter 1, HCM User's Guide, describes the new research incorporated into the present edition of the HCM.

OVERVIEW

Research Basis for the Preupdate HCM 2010

Exhibit 36-47 lists the major research projects that contributed to the HCM 2010. The impacts of these and other projects on individual HCM chapters are described later in this section.

Focus Groups

After the publication of the HCM2000, the Transportation Research Board's Committee on Highway Capacity and Quality of Service sponsored a series of focus groups at various locations around the United States to obtain feedback and to identify desired improvements for the next edition. Committee and subcommittee members also prepared an audit of the HCM in the areas of planning, design and operations, and educational needs (*15*). After the HCM 2010 was funded, the Institute of Transportation Engineers sponsored a web-based survey on HCM usage and desired improvements, and NCHRP Project 03-92 organized several focus groups on those topics. The feedback from these and other sources was considered when decisions were made on the format, content, and organization of the HCM 2010.

Reorganization from the HCM2000

The HCM 2010 consisted of four volumes: (*a*) Volume 1: Concepts, (*b*) Volume 2: Uninterrupted Flow, (*c*) Volume 3: Interrupted Flow, and (*d*) Volume 4: Applications Guide. Material from Parts I to V of the HCM2000 was distributed into Volumes 1 to 4 of the HCM 2010 as follows:

- *Part I: Overview* material appeared in Volume 1.
- *Part II: Concepts* material appeared in Volumes 2 and 3 if used directly in an analysis (e.g., default values and LOS tables) and in Volume 1 otherwise.
- *Part III: Methodologies* material appeared in Volume 2 for uninterrupted-flow chapters and Volume 3 for interrupted-flow chapters. Worksheets and highly detailed descriptions of methodological steps appeared in the Volume 4 chapters.
- *Part IV: Corridor and Areawide* material that is conceptual in nature appeared in Volume 1. More detailed analytical material was removed in favor of guidance in the use of alternative tools for corridor and areawide analyses.

- *Part V: Simulation and Other Models* material was distributed throughout the HCM 2010. Volume 1 contained an overview of alternative tools (Chapter 6) and general guidance on comparing HCM and alternative results (Chapter 7). Specific guidance on when to consider alternative tools was presented in each chapter in Volumes 2 and 3. Selected Volume 4 chapters provided examples of applying alternative tools to situations that cannot be addressed by HCM methodologies.

Exhibit 36-47
Major Research Projects Contributing to the Preupdate HCM 2010

Project	Project Title	Project Objective(s)
NCHRP 03-60	Capacity and Quality of Service of Interchange Ramp Terminals	Develop improved methods for capacity and quality-of-service analysis of interchange ramp terminals for a full range of interchange types.
NCHRP 03-64	*Highway Capacity Manual* Applications Guide	Develop an HCM Applications Guide that shows how to apply HCM methodologies to real-world problems and indicates when other methods may be more appropriate.
NCHRP 03-65	Applying Roundabouts in the United States	Develop methods for estimating the safety and operational impacts of U.S. roundabouts and refine the design criteria used for them.
NCHRP 03-70	Multimodal Level of Service Analysis for Urban Streets	Develop a framework and enhanced methods for determining levels of service for automobile, transit, bicycle, and pedestrian modes on urban streets, in particular with respect to the interaction among the modes.
NCHRP 03-75	Analysis of Freeway Weaving Sections	Develop improved methods for capacity and LOS analysis of freeway weaving sections.
NCHRP 03-79	Measuring and Predicting the Performance of Automobile Traffic on Urban Streets	Develop techniques for measuring the performance of automobile traffic on urban streets for real-time applications; develop procedures for predicting the performance of automobile traffic on urban streets.
NCHRP 03-82	Default Values for Capacity and Quality of Service Analyses	Determine appropriate default values for inputs to HCM analyses; develop a guide to select default values for various applications.
NCHRP 03-85	Guidance for the Use of Alternative Traffic Analysis Tools in Highway Capacity Analyses	Enhance the guidance in the HCM for the selection and use of alternative traffic analysis tools.
NCHRP 03-92	Production of the Year 2010 *Highway Capacity Manual*	Develop the 2010 edition of the HCM.
Federal Highway Administration	Evaluation of Safety, Design, and Operation of Shared-Use Paths (DTFH61-00-R-00070)	Develop an LOS estimation method for shared-use paths to assist path designers and operators in determining how wide to make new or rebuilt paths and whether to separate the different types of users.
Federal Highway Administration	Active Traffic Management Measures for Increasing Capacity and Improving Performance (DTFH61-06-D-00004)	Describe active traffic management techniques and available information and analysis methods for evaluating their effectiveness in increasing highway facility capacity and improving operational performance.

Multimodal Approach

To encourage HCM users to consider all travelers on a facility when they perform analyses and make decisions, the HCM 2010 integrated material on nonautomobile and automobile modes. Thus, there were no stand-alone Pedestrian, Bicycle, and Transit chapters in this edition. Instead, users were referred to the Urban Streets chapter for analysis procedures for pedestrians, bicyclists, and transit users on urban streets; to the Signalized Intersections chapter for procedures relating to signalized intersections; and so on.

In recognition of the companion *Transit Capacity and Quality of Service Manual* (TCQSM) (*16*) and of the difficulty in keeping the two manuals in synch, users were referred to the TCQSM for transit-specific capacity and quality-of-service procedures. However, transit quality of service in a multimodal context continued to be addressed in the HCM.

Traveler Perception Models

Since the 1985 HCM, LOS was defined in terms of measures of operational conditions within a traffic stream (*17*, *18*). HCM methodologies have generally presented a single LOS measure per system element that can be (*a*) directly measured in the field, (*b*) perceived by travelers, and (*c*) affected by facility owners. However, since the publication of the HCM2000, a number of research projects studied whether a single operational factor is sufficient to describe LOS, as well as whether nonoperational factors should also be used (*19*). These projects proposed models that (*a*) incorporated multiple factors of traveler satisfaction and (*b*) set LOS thresholds based on traveler perceptions of service quality. Traveler perception models from two of these studies (*20*, *21*) were incorporated into the Multilane Highways, Two-Lane Highways, Urban Street Facilities, Urban Street Segments, and Off-Street Pedestrian and Bicycle Facilities chapters.

Generalized Service Volume Tables

The HCM2000 provided "example service volume tables" for 10 system elements. The service volume tables were developed by using a single set of default values and were accompanied by cautionary notes that they were illustrative only. The HCM 2010 provided "generalized service volume tables" for facilities that incorporate a range of national default values. These tables could be considered for such applications as statewide performance reporting, areawide (i.e., regional) modeling, and future-year analyses as part of a long-range transportation planning process.

METHODOLOGICAL CHANGES BY SYSTEM ELEMENT

Freeway Facilities

The basic methodology was similar to the one given in the HCM2000 but incorporated a new weaving-segment analysis procedure. A significant change was the addition of LOS thresholds for freeway facilities based on density. Other changes included updates to the material on the impact of weather and work zones on freeway facility capacity, along with new information on the impact of active traffic management measures on freeway operations.

Basic Freeway Segments

The basic methodology was similar to the one given in the HCM2000. The FFS prediction model was improved, and a speed–flow curve for segments with a 75-mi/h FFS was added.

Freeway Weaving Segments

This chapter was completely updated and incorporated the methodology developed by NCHRP Project 03-75. Although the general process for analyzing weaving segments was similar to that given in the HCM2000, the HCM 2010 models was based on an up-to-date set of weaving data. The following are the two major differences in how the methodology is applied: (*a*) a single algorithm for predicting weaving speeds and a single algorithm for predicting nonweaving speeds were provided, regardless of the weaving configuration, and (*b*) the LOS F threshold was changed.

Ramps and Ramp Junctions

The following revisions were made to the HCM2000 methodology:

- Procedures were added to check for unreasonable lane distributions that overload the left or right lane(s) (or both) of the freeway.
- A revision was made to correct an illogical trend involving on-ramps on eight-lane freeways in which density increases as the length of the acceleration lane increases.

Multilane Highways

The multilane highways automobile methodology was essentially the same as that given in the HCM2000. A methodology for calculating bicycle LOS for multilane highways was added.

Two-Lane Highways

The following revisions were made to the HCM2000 automobile methodology:

- The two-direction analysis was dropped: the one-direction methodology is the only one used, with two-direction results obtained by appropriate weighted averaging of the one-direction results.
- Several key curves and tables used in one-direction analyses were adjusted and incorporated into the chapter.

A bicycle LOS methodology for two-lane highways was added.

Urban Street Facilities

This was a new chapter containing guidance to help analysts determine the scope of their analysis (i.e., isolated intersection versus coordinated signal system) and the relevant travel modes (i.e., automobile, pedestrian, bicycle, transit, or a combination). The methodology section described how to aggregate results from the segment and point levels of analysis into an overall facility assessment. Information on the impact of active traffic management measures on urban street performance was added.

Urban Street Segments

This chapter was completely rewritten. The work of NCHRP Project 03-79 was incorporated into the chapter, providing improved methods for estimating urban street FFS and running times, along with a new method for estimating the stop rate along an urban street. In addition, the work of NCHRP Project 03-70 was incorporated, providing a multimodal LOS methodology that could be used to evaluate trade-offs in how urban street right-of-way is allocated among the modes using the street.

Signalized Intersections

The following revisions were made to the HCM2000 methodology:

- A new incremental queue accumulation method was added to calculate the d_1 delay term and the Q_1 length term. It was equivalent to the HCM2000 method for the idealized case but was more flexible to accommodate nonideal cases, including coordinated arrivals and multiple green periods with differing saturation flow rates (i.e., protected-plus-permitted left turns and sneakers).
- An actuated controller operation modeling procedure was added.
- A left-turn lane overflow check procedure was added.
- Pedestrian and bicycle LOS methodologies relating to signalized intersections were moved into this chapter.

Unsignalized Intersections

The HCM2000's Unsignalized Intersections chapter was split into three chapters: two-way STOP-controlled intersections, all-way STOP-controlled intersections, and roundabouts.

Two-Way STOP-Controlled Intersections

The two-way STOP-controlled intersection methodology for the automobile mode was essentially the same as the one given in the HCM2000, except gap-acceptance parameters for six-lane streets were added. Furthermore, pedestrian and bicycle LOS methodologies relating to two-way STOP-controlled intersections were moved into this chapter.

All-Way STOP-Controlled Intersections

The all-way STOP-controlled intersection methodology was essentially the same as the one given in the HCM2000. A queue-estimation model was added.

Roundabouts

This chapter replaced the HCM2000 roundabout content. It was based on the work of NCHRP Project 03-65, which developed a comprehensive database of U.S. roundabout operations and new methodologies for evaluating roundabout performance. A LOS table for roundabouts was added.

Interchange Ramp Terminals

Material on interchange ramp terminals was completely updated on the basis of NCHRP Project 03-60.

Off-Street Pedestrian and Bicycle Facilities

The pedestrian path procedures were essentially the same as those of the HCM2000, but guidance was provided on how to apply the procedures to a wider variety of facility types. The bicycle path procedures, which were based on Dutch research in the HCM2000, were updated on the basis of results of an FHWA study to calibrate the Dutch model for U.S. conditions and increase the number of path user groups (e.g., inline skaters and runners) addressed by the procedures.

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