CHAPTER 22

FREEWAY FACILITIES

CONTENTS

Exhibits

I. INTRODUCTION

Background and concepts for this chapter are in Chapter 13

Freeway facilities are composed of connected segments consisting of basic freeway segments, ramp segments, and weaving segments. When several of these segments occur in sequence, they form a freeway facility. A freeway facility is the fundamental unit of analysis in this chapter. A freeway facility is analyzed by direction, and the independent analysis of both directions constitutes the analysis of a two-direction freeway facility. The reader is referred to Chapter 13 for discussion of freeway concepts.

SCOPE OF METHODOLOGY

In Chapters 23, 24, and 25, freeway components are addressed as isolated segments that are assumed to have no significant interaction. A procedure that integrates the methodologies of Chapters 23, 24, and 25 is provided in this chapter, subject to several limitations. The freeway facility has spatial and time dimensions subject to defined limits. The spatial dimension consists of continuous connected segments of defined length, type, and width. The segments could include basic freeway segments, on-ramp junction segments, off-ramp junction segments, or weaving area segments. Free-flow conditions must exist at the upstream and downstream ends of the freeway facility. The maximum length of a freeway facility that should be considered is on the order of 15 to 20 km, so that traffic enters and leaves the freeway in the same time interval.

The temporal dimension consists of connected time intervals. Undersaturated conditions must occur in the first and last time interval. The analysis period to be considered is divided into 15-min time intervals. The material developed for this chapter resulted from research sponsored by the Federal Highway Administration (*1*).

LIMITATIONS TO METHODOLOGY

A complete discussion of freeway control systems or even the analysis of the performance alternatives is beyond the scope of this chapter. The reader should consult references identified in a later section of this chapter. The methodology does not account for delays caused by vehicles using alternate routes or vehicles leaving before or after the study time duration.

Certain freeway traffic conditions cannot easily be analyzed by the methodology. Multiple overlapping bottlenecks are an example. Therefore, other tools may be more appropriate for specific applications beyond the capabilities of the methodology. Refer to Part V of this manual for a discussion of simulation and other models.

User demand responses such as spatial, temporal, modal, or total demand responses caused by traffic management strategies are not automatically incorporated within the methodology. On viewing the facility traffic performance results, the analyst can modify the demand input manually to analyze the effect of user demand responses or traffic growth. The accuracy of the results depends on the accuracy of the estimation of the user demand responses.

The freeway facility methodology is limited to the extent that it can accommodate demand in excess of capacity. The procedures address only local oversaturated flow situations, not systemwide oversaturated flow conditions.

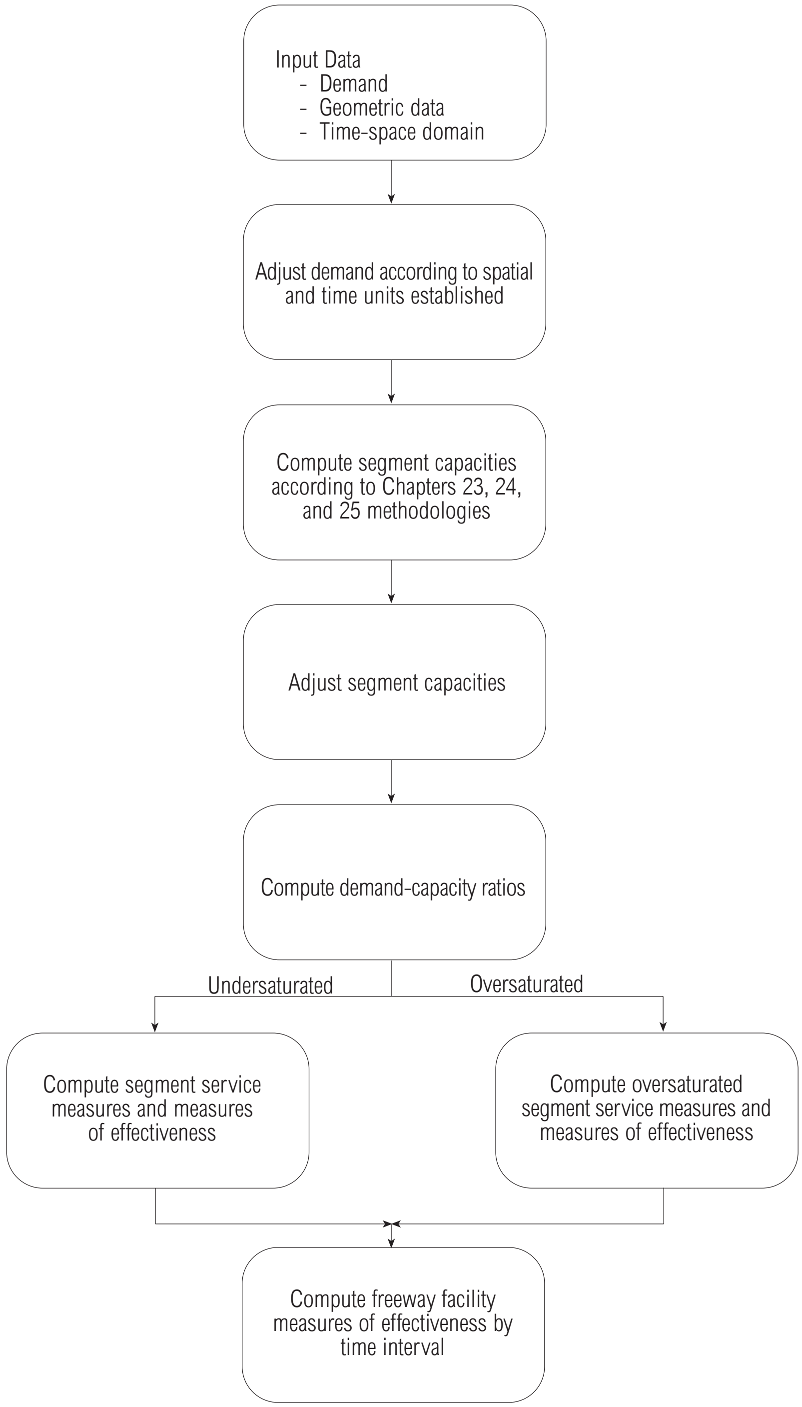
The completeness of the analysis will be limited if freeway segments in the first time interval, the last time interval, and the first freeway segment do not all have demand-to-capacity ratios less than 1.00. The rationale for these limitations is discussed in the section labeled demand-capacity ratio.

The analyst can, given enough time, analyze a completely undersaturated time-space domain manually, although this is difficult. It is not expected that analysts will ever manually analyze a time-space domain that includes oversaturation. For heavily congested freeway facilities with interacting bottleneck queues, the analyst may wish to review Part V of this manual before undertaking this methodology.

II. METHODOLOGY

Exhibit 22-1 summarizes the methodology for analyzing freeway facilities. The methodology integrates the basic freeway segment, ramp segment, and weaving segment procedures into a freeway facility analysis. The methodology adjusts vehicle speeds appropriately to account for effects in adjacent segments. The methodology can analyze freeway traffic management strategies only in cases for which 15-min time intervals are appropriate and for which reliable data for capacity and demand estimates exist.

EXHIBIT 22-1. FREEWAY FACILITY METHODOLOGY



The effect of various demand management techniques can be assessed by varying the demand associated with the technique. The methodology is limited to applications for which data are available to quantify the effects of demand management and whose complexity does not exceed the capabilities of the methodology. This is especially true when periods of oversaturation occur. The analysis should begin and end with no portion of the freeway having oversaturation.

Freeway control is frequently motivated by operational problems such as one or more bottlenecks with significant mainline congestion. Ramp metering is a strategy to reduce the amount of congestion by limiting demand. The effectiveness of a particular ramp-metering strategy in improving freeway performance can be analyzed by the methodology. The ability of the methodology to assess the total effects of the strategy depends on assumptions related to where excess demand would relocate. If the demand is diverted to another time or location within the analysis period, effects can be accounted for.

The use of high-occupancy vehicle (HOV) lanes on freeways raises the issues of the operating characteristics of such lanes and the effects on the remainder of the freeway. The issues are complex because HOV lanes come in many forms including separated facilities, reserved freeway lanes (concurrent flow and contraflow lanes), and priority access (ramp meter bypass lanes). The methodology addresses separated facilities but not the interactions between the HOV lane and the mixed-flow lanes.

Single-lane HOV facilities can be analyzed only when flow rates are less than 1,600 veh/h/ln

There are a number of data requirements for conducting an analysis, and some applications are beyond the capabilities of the methodology. The issue of capacity must be addressed first. This is a difficult issue because data are limited and by design most HOV freeway facilities operate below capacity to maintain a high level of service. Single-lane HOV facilities generally have different speed characteristics because of the lack of passing opportunities. Therefore, it is recommended that single-lane analyses be conducted only when HOV lane demand is less than 1,600 veh/h/ln.

The methodology for analyzing freeway facilities is comprehensive in that it interacts with three other chapters of this manual and incorporates both undersaturated and oversaturated flow analysis capabilities. In this portion of the chapter an overview of the methodology is given. The time-space domain of a freeway facility is described with particular attention to facility geometrics, facility traffic demands, demand-capacity analysis, and optional traffic management strategies. Service measures, levels of service (LOS), and performance measures are also discussed.

The purpose of this section is to describe the computational modules of the methodology. To simplify the presentation, the focus is on the function of and rationale for each module. An expanded version of this section containing all the supporting analytical models and equations is presented in Appendix A.

PERFORMANCE MEASURES

Facilitywide service measures and LOS designations are not incorporated in this chapter, as they are in other chapters in Part III of this manual. This is due to the complexity of assessing freeway facilities when oversaturated flow conditions are encountered. A freeway facility may contain both uncongested and heavily congested segments, and any average service measure for the entire length is likely to be misleading and difficult to classify by LOS.

Systemwide measures can be generated, but no LOS guidelines are given

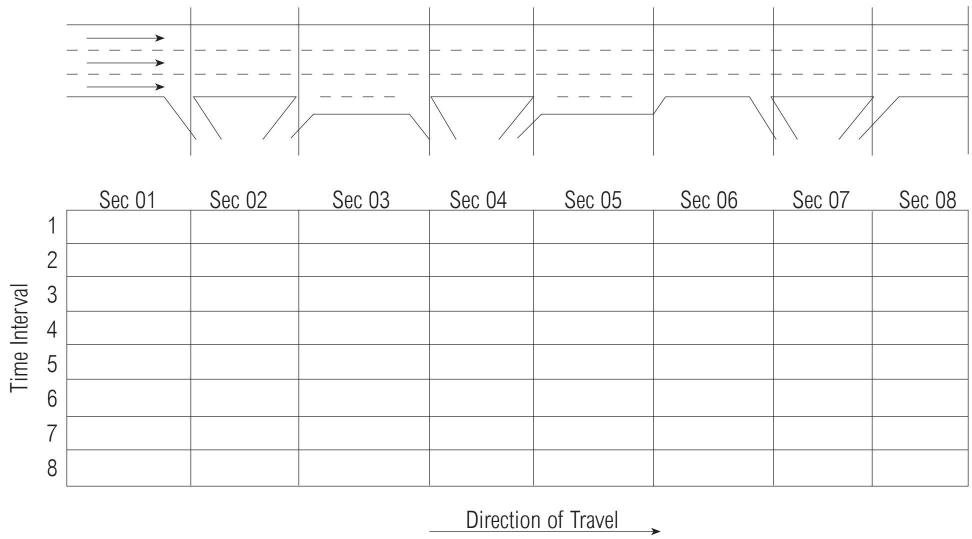
The methodology provides estimates of speed, travel time, density, flow rate (vehicle and person), volume-to-capacity ratio, and congestion status for each cell in the time-space domain. From these estimates, vehicle hours (person-hours) of travel as well as vehicle kilometers (person kilometers) of travel for each cell can be determined.

The previously discussed traffic performance measures can be aggregated by the analyst over the length of the freeway facility, over the study time duration, and over the entire time-space domain.

SEGMENTING FREEWAY FACILITIES

The time-space domain of a freeway facility is used to provide an overview of the methodology. A typical time-space domain is shown in Exhibit 22-2.

EXHIBIT 22-2. TIME-SPACE DOMAIN OF A FREEWAY FACILITY



Freeway facilities up to 20 km long can be analyzed with the methodology

The horizontal scale indicates the distance along the freeway facility. Traffic moves from left to right, and the scale is divided into freeway sections. A freeway section boundary occurs wherever there is a change in traffic demand (i.e., on-ramp or off-ramp) or a change in segment capacity (i.e., lane drop or lane addition). Freeway facilities up to 15 to 20 km long can be analyzed by this methodology. Estimates of traffic demand on longer freeway facilities cannot be reliably developed with the methodology, because the travel time between some origins and destinations will exceed the standard time interval (15 min).

The vertical scale indicates the study time duration. Time extends down the time-space domain, and the scale is divided into 15-min intervals. The study time duration can include any number of contiguous 15-min intervals. The number of section-based cells in the time-space domain is the product of the number of sections and the number of 15-min intervals. In Exhibit 22-2 there are 64 section-based cells.

The boundary conditions of the time-space domain are extremely important since the time-space domain will be analyzed as an independent freeway facility having no interactions with the upstream or downstream portions of any connecting facilities (including freeways and surface streets) or with time periods before or after the study time duration. This means that no congestion should occur along the four boundaries of the time-space domain. The cells located along the four boundaries should all have demands less than capacities and should contain undersaturated flow conditions.

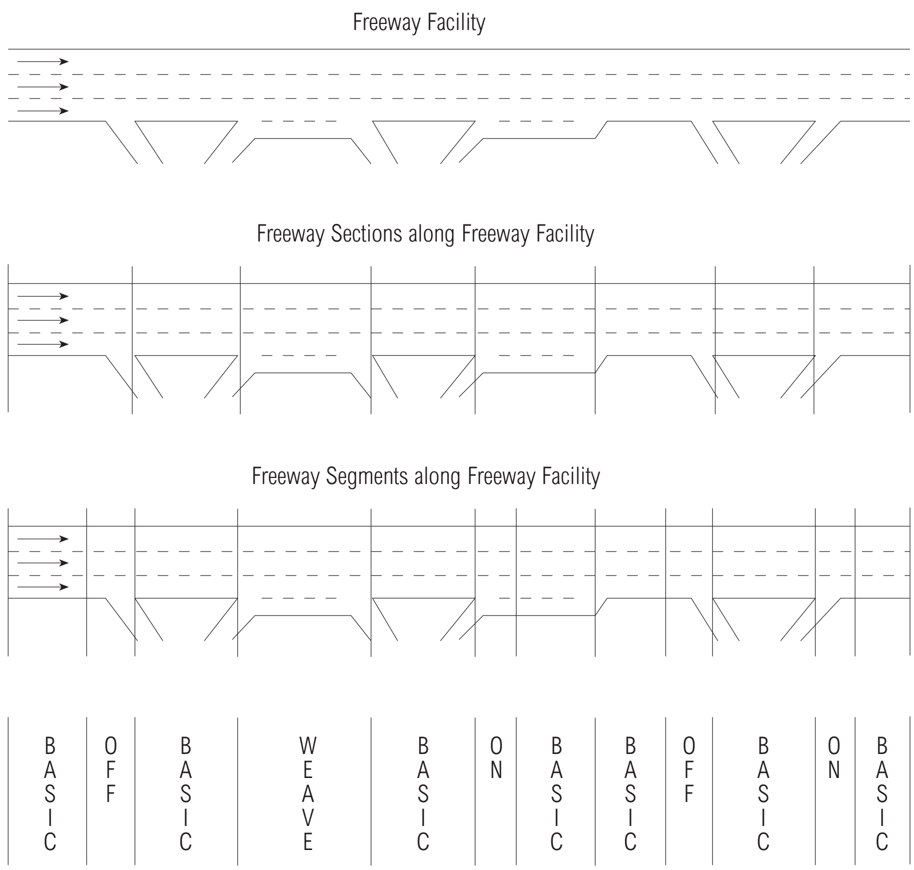
Exhibit 22-2 shows the division of the freeway facility into connected freeway sections. However, to use the predictions of capacity and performance measures from the basic freeway, ramp, and weaving segment chapters, the sections must be further divided into segments. Each section contains one or more segments depending on the freeway geometrics.

First, any weaving segment, as defined in Chapter 24, Freeway Weaving, is labeled as a weaving segment. Next, any on-ramp or off-ramp segment, as defined in Chapter 25, Ramps and Ramp Junctions, is labeled as an on-ramp or off-ramp segment. The remaining portions of the freeway facility are labeled as basic freeway segments (Chapter 23).

Guidelines for handling long and short sections

Special labeling of segments may be required under certain circumstances. For example, a long freeway section between an on-ramp and an off-ramp can be subdivided into three segments: on-ramp, basic freeway, and off-ramp. A complication may occur when a short freeway section contains an on-ramp followed by an off-ramp without an auxiliary lane between the two ramps. The problem arises if the length of the freeway section is insufficient to meet the requirements of the sum of the lengths of the on-ramp segment and the off-ramp segment as stated in Chapter 25. In that case, the overlapping freeway segment is analyzed both as an on-ramp segment and an off-ramp segment, and the more restrictive option is selected. Similarly, weaving sections with lengths exceeding 750 m can be analyzed as basic segments with the added auxiliary lane and ramp demands. Other special types of freeway sections requiring special attention may be encountered. In those cases, Chapters 23, 24, and 25 should be consulted. The transformation of freeway sections into freeway segments for the freeway facility of Exhibit 22-2 is shown in Exhibit 22-3. The estimated segment capacities and traffic performance algorithms filter down through the time-space domain so that each cell has an estimated capacity and an algorithm for predicting traffic performance measures.

EXHIBIT 22-3. CONVERSION OF FREEWAY SECTIONS INTO FREEWAY SEGMENTS



FREEWAY FACILITY DEMANDS AND ESTIMATION OF TRAFFIC DEMAND

Traffic counts at each entrance to and exit from the freeway facility (including the mainline entrance and the mainline exit) for each time interval serve as input to the methodology. Whereas entrance counts are considered to represent the current entrance demands for the freeway facility (provided that there is not a queue on the freeway entrance), the exit counts may not represent the current exit demands for the freeway facility because of freeway congestion.

For planning applications, estimated traffic demands at each entrance to and exit from the freeway facility for each time interval serve as input to the methodology. The sum of the input demands must be equal to the sum of the output demands in every time interval.

Once the entrance and exit demands are calculated, the demands for each cell in every time interval can be estimated. The segment demands can be thought of as filtering across the time-space domain and filling each cell in the time-space matrix.

Estimation of traffic demand requires careful differentiation of volume, as counted, and demand

Demand estimation is required if the methodology uses actual freeway counts. If demand flows are known or can be projected, they are used directly. The demand estimation module converts the input set of freeway exit 15-min traffic counts into a set of freeway exit 15-min traffic demands. Freeway exit demand is defined as the number of vehicles that desire to exit the freeway in a given 15-min time interval. This demand may not be represented by the 15-min exit count because of upstream freeway congestion within the facility.

Time interval scale factor defined

The procedure followed is to sum the freeway entrance demands along the entire freeway facility (including the freeway mainline entrance) and to compare it with the sum of the freeway exit counts along the entire directional freeway facility (including the freeway mainline exit) for each time interval. The ratio of the total freeway entrance demands to the freeway exit counts in each time interval is called the time interval scale factor. Theoretically, the scale factor should approach 1.00 when the freeway exit counts are, in fact, freeway exit demands.

Scale factors greater than 1.00 indicate increasing levels of congestion within the freeway facility, with exit traffic counts underestimating actual freeway exit demands. Scale factors less than 1.00 indicate decreasing levels of congestion, with exit traffic counts exceeding actual freeway exit demands. To provide an estimate of freeway exit demand, each freeway exit count is multiplied by the time interval scale factor.

Once the entrance and exit demands are determined, the traffic demands for each freeway section in each time interval can be calculated. On the time-space domain diagram, the section demands can be viewed as projecting horizontally across the diagram with each cell containing an estimate of its 15-min demand.

ADJUSTMENTS OF SEGMENT CAPACITY

Segment capacity estimates are determined directly from Chapters 23, 24, and 25 for basic, weaving, and ramp segments, respectively. All estimates of segment capacity should be carefully reviewed and compared with local knowledge and available traffic information for the study site, particularly for known bottleneck segments.

On-ramp and off-ramp roadway capacities are also determined in this module. On-ramp demands may exceed on-ramp capacities and limit the traffic demand entering the facility. Off-ramp demands may exceed off-ramp capacities and cause congestion on the freeway, although that effect is not accounted for in the methodology. Unlike the computations performed in the basic freeway, weaving, and ramp chapters, all capacity computations performed in this chapter are on the basis of vehicles per hour and not passenger cars.

In this chapter, capacity computations are on a veh/h basis

The effect of a predetermined ramp-metering plan can be evaluated in this methodology by overriding the computed ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to the specified metering rate. This feature not only permits the evaluation of a prespecified ramp-metering plan, but also permits the user to improve the ramp-metering plan by experimentation.

Capacities can be selected to reflect a variety of controls or conditions

Freeway design improvements can be evaluated within this methodology by modifying the design features of any portion or portions of the freeway facility. For example, the effect of adding auxiliary lanes at critical locations and full lanes over multiple segments can be assessed.

Reduced-capacity situations can also be investigated. The capacity in any cell of the time-space domain can be reduced to represent incident situations such as construction and maintenance activities, adverse weather, and traffic accidents/vehicular breakdowns. Conversely, capacity can be increased to match field measurements. In analyzing adjusted capacity, use of an alternative speed-flow relationship is important. The computational details for this case are provided later in this chapter.

Permanent Capacity Reduction

A lane drop is in many ways the simplest capacity-reducing situation to deal with. Capacity in both segments, that with the smaller number of lanes and that with the larger number, can be calculated using Chapter 23, 24, or 25 methodologies. So long as the arriving demand is less than the lower capacity, no queue will form upstream of the lane drop. If the arriving demand begins to exceed the reduced capacity, a queue will begin to form immediately upstream of the reduced-capacity section, which will have become a bottleneck. Some results suggest that a poorly designed merge at the lane drop can negatively affect the capacity of the segment with the smaller number of lanes because of the increase in friction and turbulence, but this effect has not yet been quantified.

Construction Activities Capacity Reduction

Capacity reductions due to construction activities can be divided into short-term maintenance work zone lane closures and long-term construction zone closures. One of the primary distinctions between short-term work zones and long-term construction zones is the nature of the barriers used to demarcate the work area. Long-term construction zones generally have portable concrete barriers; short-term work zones use standard channeling devices (traffic cones, drums) in accordance with the *Manual on Uniform Traffic Control Devices* (*2*). Generally, reduction of capacity brought about by reconstruction or major maintenance activities will last for several weeks or even months, although some short-term maintenance activities last only a few hours.

Short-Term Work Zones

Research (*3*) suggests that a capacity of 1,600 pc/h/ln be used for short-term freeway work zones, regardless of the lane closure configurations. For some types of closures, capacity may be higher (*3*).

The base value should be adjusted for other conditions, as follows.

Intensity of work will affect capacity

• Intensity of work activity: The intensity of work activity refers to the number of workers on site, the number and size of work vehicles in use, and the proximity of work to the travel lanes in use. Unusual types of work also contribute to the apparent intensity, simply in terms of the rubbernecking factor. Research data did not result in explicit quantification of these effects, but it is suggested that the capacity of 1,600 pc/h/ln be adjusted by up to ±10 percent for work activity that is more or less intense than normal (*3*). The research did not define what constitutes normal intensity. Hence, this factor should be applied on the basis of professional judgment, recognizing that 1,600 pc/h/ln is an average over a variety of conditions.

Heavy vehicles should be accounted for

• Effects of heavy vehicles: It is recommended that the heavy-vehicle adjustment factor, f_{HV}, found elsewhere in the manual be used to account for the effect of heavy vehicles in the traffic stream moving through the work zone, as shown in Equation 22-1.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} \tag{22-1}$$

where

f_{HV} = heavy-vehicle adjustment factor,
P_T = proportion of heavy vehicles, and
E_T = passenger-car equivalent for heavy vehicles.

The value of E_T can be taken from Chapter 23, Basic Freeway Segments.

Entrance ramps within 150 m of a lane closure will affect capacity

- Presence of ramps: If there is an entrance ramp within the taper area approaching the lane closure or within 150 m downstream of the beginning of the full lane closure, the ramp will have a noticeable effect on the capacity of the work zone for handling mainline traffic. This arises in two ways. First, the ramp traffic will generally force its way in, so it will directly reduce the amount of mainline traffic that can be handled. Second, the added turbulence in the merging area due to the entrance ramp may itself reduce the capacity slightly. If at all possible, ramps should be located at least 450 m upstream from the beginning of the full closure to maximize the total work zone throughput. If that cannot be done, then either the ramp volume should be added to the mainline volume to be served or the capacity of the work zone should be decreased by the ramp volume (up to a maximum of half of the capacity of one lane, on the assumption that at very high volumes mainline and ramp vehicles will alternate). Equation 22-2 is used to compute the resulting reduced capacity.

$$c_a = (1{,}600 + I - R) * f_{HV} * N \qquad (22\text{-}2)$$

where

c_a = adjusted mainline capacity (veh/h);
f_{HV} = adjustment for heavy vehicles as defined in Equation 22-1;
I = adjustment factor for type, intensity, and location of the work activity, as discussed above (ranges from –160 to +160 pc/h/ln);
R = adjustment for ramps, as described in the preceding paragraph; and
N = number of lanes open through the short-term work zone.

Long-Term Construction Zones

For long-term construction zones, capacity values are given in Exhibit 22-4. If traffic crosses over to lanes that are normally used by the opposite direction of travel, the capacity is close to the 1,550 veh/h/ln value in Exhibit 22-4 (*5*). If no crossover is needed, but only a merge down to a single lane, the value is typically higher and may average about 1,750 veh/h/ln (*6*).

EXHIBIT 22-4. SUMMARY OF CAPACITY VALUES FOR LONG-TERM CONSTRUCTION ZONES

No. of Normal Lanes	Lanes Open	Number of Studies	Range of Values (veh/h/ln)	Average per Lane (veh/h/ln)
3	2	7	1780–2060	1860
2	1	3	–	1550

Source: Dudek (*4*).

Lane Width Consideration

An additional adjustment factor can be added to the long-term and short-term reduction model for the effect of lane width (*7*). For traffic with passenger cars only, headways increase by about 10 percent in going from 3.5-m widths to 3.25- or 3.0-m widths and by an additional 6 percent in going to 2.75-m widths. These increases in headways translate to 9 and 14 percent drops in capacity for the narrower lane widths within construction zones.

Adverse Weather Capacity Reduction

There have been several research studies on the effect of rain, snow, and fog. It has become clear that adverse weather can significantly reduce not only capacity but also operating speeds. The following sections discuss the effects of each of these weather conditions and address the issue of when and how to take these effects into account in applying the methodology.

Rain

Research found that speeds are not particularly affected by wet pavement until visibility is also affected (*8*). This result suggests that light rain will not have much effect on speeds (and presumably not on capacities) unless it is of such extended duration that there is considerable water on the pavement. Heavy rain, on the other hand, affects visibility immediately and can be expected to have a noticeable effect on traffic flow.

This expectation is borne out by studies of freeway traffic. Research found minimal reductions in maximum observed flows for light rain but significant reductions for heavy rain (*9*). Likewise, the research found a small effect on operating speeds for light rain and larger effects for heavy rain. These changes in operating speeds are important because they directly affect traffic performance.

For light rain, a reduction in free-flow speeds of 2.0 km/h was observed (*9*). At a flow rate of 2,400 veh/h, the effect of light rain was to reduce speeds to about 82 km/h, compared with speeds of 89 to 95 km/h under clear and dry conditions. Under light rain conditions, little if any effect was observed on flow or capacity.

For heavy rain, the drop in free-flow speeds was 5 and 7 km/h. The result of heavy rain is to reduce speeds at 2,400 veh/h to 76 and 79 km/h from, respectively, 89 and 95 km/h. These are reductions of 13 and 16 km/h. Maximum flow rates can also be affected and might be 14 to 15 percent lower than those observed under clear and dry conditions.

Snow

For snow, major differences were found depending on the quantity or rate of snowfall, with light snow having minimal effects and heavy snow having potentially very large effects (*9*). If snow-clearing operations cannot keep the road relatively clear during a heavy snowfall, the snow accumulation on the highway obscures the lane markings. Observation suggests that under these circumstances, drivers often seek not only longer headways, but also greater lateral clearance. As a result, a three-lane freeway segment is used as if it had only two widely separated lanes. This alone has a considerable effect on capacity.

Light snow was associated with a statistically significant drop of 1 km/h in free-flow speeds. The effect on maximum observed flows was midway between the effects of light and heavy rain or between a 5 and 10 percent reduction.

Heavy snow significantly influences the speed-flow curve. Free-flow speeds were reduced by 37 and 42 km/h at the two stations from what they were under clear and dry conditions (102 and 106 km/h, respectively). Maximum observed flows dropped from 2,160 to 1,200 veh/h/ln at the station upstream of the queue. At the station that might be a bottleneck itself for part of the peak period, the maximum observed flows dropped from 2,400 to 1,680 veh/h/ln. This suggests a 30 percent drop in capacity due to heavy snow in an urban area where traffic will generally keep moving to some extent.

Fog

Although no studies have quantified the effects of fog on capacity, work has been done in Europe on fog warning systems, which use variable speed limit signs to reduce speeds during foggy conditions. Those studies tend to report on the effectiveness of the speed warning signs in reducing mean speeds, not on what speeds (or capacities) are due to the fog alone. For example, they report effectiveness of fog warning devices of 8 to 10 km/h in reducing speed but provide no information on capacity effects (*10, 11*).

Environmental Capacity Reduction

Research in Germany used speed-flow-density relationships to fit speed-flow curves to the field observations (*12*). The capacity of each study site under a variety of conditions was estimated from these curves. The results are useful not only in extending the research results cited on rain and wet pavement but also in identifying some other causes of temporary capacity reduction that have not generally been discussed (e.g., the difference between daylight and darkness and between weekdays and weekends).

A set of relationships for 10 to 15 percent heavy vehicles has been used for comparison in Exhibit 22-5. Although this exhibit shows the per lane capacities found in Germany, the numbers clearly do not translate directly to North American conditions. The most obvious difference is that capacity per lane is lower than would be found in North America. In addition, the capacity per lane for a six-lane freeway is lower than that of a four-lane freeway for all but one of the conditions. These results are no doubt a consequence of the function the researchers were fitting, together with the fact that there were few data near capacity because hourly data were analyzed. What is important in the exhibit is the percentage reduction in capacity under each of the sets of conditions, which is shown on the second line for each type of freeway and type of day (weekday or weekend).

EXHIBIT 22-5. CAPACITIES ON GERMAN AUTOBAHNS UNDER VARYING CONDITIONS (veh/h/ln)

Freeway Type	Weekday or Weekend	Daylight and Dry	Dark and Dry	Daylight and Wet	Dark and Wet
Six-lane freeway	Weekday	1489	1299	1310	923
	Change[a] (%)		13	12	38
Six-lane freeway	Weekend	1380	1084	1014	-
	Change[a] (%)		21	27	-
Four-lane freeway	Weekday	1739	1415	1421	913
	Change[a] (%)		19	18	47
Four-lane freeway	Weekend	1551	1158	1104	-
	Change[a] (%)		25	29	-

Note:
a. The percentage reduction from daylight and dry conditions for the same day of the week.
Source: Brilon and Ponzlet (*12*).

The estimates for weekdays and daylight for the reduction due to wet pavement, 12 and 18 percent, are consistent with the estimates discussed above for the effects of rain. The reductions in capacity due to darkness are of the same order as those due to rain: 13 and 19 percent for six- and four-lane facilities, respectively. Since winter peak-period commuter traffic occurs in darkness in many locations, these capacity reductions are important to recognize.

The capacity of a freeway on weekends or holidays can be substantially less than when it carries commuter traffic. Although the percentage change is not shown in the exhibit, it amounts to a 7 to 10 percent reduction during dry daylight conditions.

Capacity Reductions due to Traffic Accidents or Vehicular Breakdowns

Mean duration of an incident was 37 min but was highly variable

Capacity reductions due to traffic accidents or vehicular breakdowns are generally short-lived, ranging from less than 1 h before they can be cleared (for a minor fender-bender involving only passenger vehicles) to as long as 12 h (for a major accident involving fully loaded tractor-trailer rigs). For example, on the basis of research, the mean duration of a traffic incident was 37 min, with just over half of the incidents lasting 30 min or less and 82 percent of the incidents lasting 1 h or less (*13*). When trucks were involved, however, the duration was longer; accidents involving trucks lasted 63 min on the average.

The effect of an incident on capacity depends on the proportion of the traveled roadway that is blocked by the stopped vehicles, as well as on the number of lanes on the roadway at that point. Exhibit 22-6 gives information on these effects (*14, 15*).

EXHIBIT 22-6. PROPORTION OF FREEWAY SEGMENT CAPACITY AVAILABLE UNDER INCIDENT CONDITIONS

Number of Freeway Lanes by Direction	Shoulder Disablement	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
2	0.95	0.81	0.35	0.00	N/A
3	0.99	0.83	0.49	0.17	0.00
4	0.99	0.85	0.58	0.25	0.13
5	0.99	0.87	0.65	0.40	0.20
6	0.99	0.89	0.71	0.50	0.26
7	0.99	0.91	0.75	0.57	0.36
8	0.99	0.93	0.78	0.63	0.41

N/A - not applicable.
Source: Reiss and Dunn (*14*) and Gordon et al. (*15*).

Note that in the case of a blocked lane, the loss of capacity is likely to be greater than simply the proportion of original capacity that is physically blocked. For example, a four-lane (in one direction) freeway with two lanes blocked retains only 25 percent of its original capacity (Exhibit 22-6). The added loss of capacity arises because drivers slow to look at the incident while they are abreast of it and are slow to react to the possibility of speeding up to move through the incident area.

The rubbernecking factor is also responsible for a reduction in capacity in the direction of travel opposite to that in which the accident occurred. No quantitative studies of this effect have been published, but experience suggests that it depends on the magnitude of the incident (including the number of emergency vehicles present). The reduction may range from 5 percent for a single-car accident and one emergency vehicle to 25 percent for a multivehicle accident with several emergency vehicles.

Applying Capacity Reductions

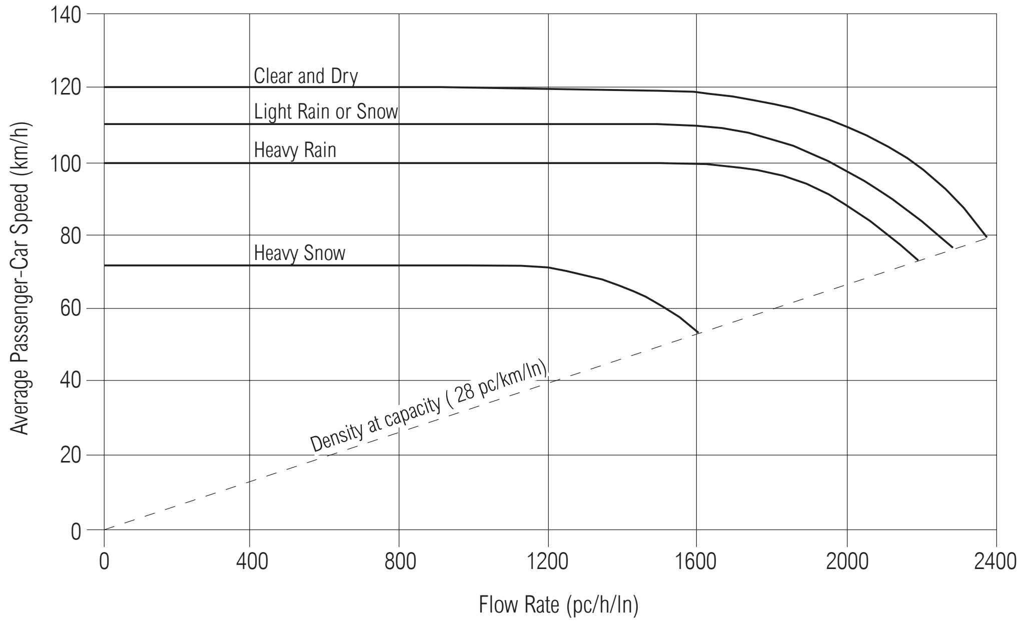
There are several ways to use the information on reduced capacities contained in the preceding material, ranging from quick approximations, through the application of the methodology described in this chapter, to other quantitative approaches involving queuing analysis or shock wave analysis. The quick approximations simply require reviewing the expected traffic demands and comparing them with the applicable capacity reduction. If the demands do not exceed the reduced capacity, there will not be any major difficulties in handling the traffic. A more detailed analysis may be desired to estimate the expected traffic performance, in which case use of the methodology would be appropriate.

The methodology works with the full speed-flow curve (for the undersaturated part of the relationship), but in many of the cases described above, only the effect on capacity has been identified by research reported to date. The literature does not describe the effect of the factor (incident, construction) on speeds and hence on the speed-flow curve. Without a full speed-flow curve, the analyst is forced to use other methods or to work around this limitation of the model. Consider each of the capacity reductions in turn, from the simplest to the most difficult to deal with.

Assumptions regarding effects on speeds

Adverse weather is the easiest to deal with, because the results cited above indicate effects on both speeds and capacity. Consequently, the analyst can simply use a speed-flow curve for a lower free-flow speed (FFS) to model the effects of inclement weather. Neither of the research studies reported a method that would equate to a reduction in FFS, but their results can be reasonably well approximated that way (*9, 12*). For light rain or snow, for example, speeds at capacity drop by 7 to 13 km/h, which can be approximated by a reduction in FFS of 10 km/h. For heavy rain, the approximation would be a reduction in FFS of 20 km/h. For heavy snow, the reduction would be 50 km/h. Exhibit 22-7 shows the approximate curves for these conditions, using a constant density to determine the capacity for each curve.

EXHIBIT 22-7. SPEED-FLOW CURVES FOR DIFFERENT WEATHER CONDITIONS



Note:
FFS = 120 km/h (base conditions).

For the other temporary capacity reductions, the research findings deal only with the change in capacity. In most of these cases, reasonable estimates of speed conditions can be made. For example, in construction zones, a reduced speed is usually posted, and lower speeds usually do occur, particularly where actual construction operations are taking place. Likewise, for incidents, traffic naturally slows as drivers pass the incidents and try to get a look at what happened. Thus, one can attempt to model these situations on the basis of a downward-shifted speed-flow curve, like those shown in Exhibit 22-8.

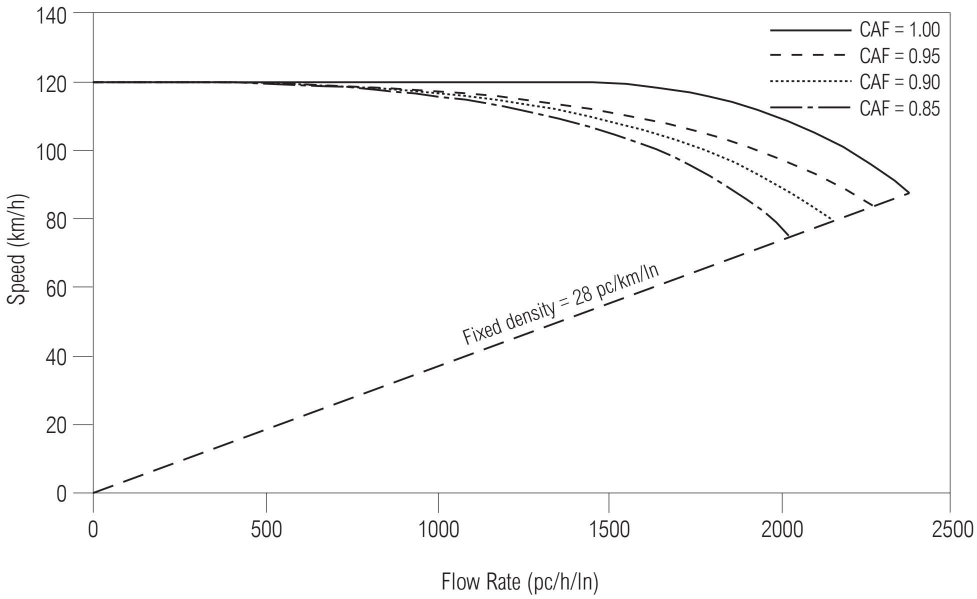
If the analyst were not interested in the speeds, the capacity reduction could be modeled by using a fractional number of lanes that would reflect the new capacity of the roadway, rather than the real number of lanes. For example, in the case of a four-lane (in one direction) freeway facility with two lanes blocked, Exhibit 22-6 shows that only 25 percent of the original capacity is available. To reflect this, the analyst could show only a single lane through the area of the incident, even though it is in fact a four-lane segment. However, since most of the performance measures rely on or are based on speed, this simplified approach will not permit a complete analysis. Consequently, use of a speed-flow curve from the family shown in Exhibit 22-7 or 22-8 is recommended.

The methodology and other methods have a role in analyzing the effect of incidents, even when they are of short duration, by assisting in the planning of responses to various types and locations of incidents before they occur. The advantage of planning is that it can minimize the need for improvising decisions about diversion plans and other methods of responding to incidents.

DEMAND-CAPACITY RATIO

Each cell in the time-space domain now contains an estimate of demand and capacity as well as an algorithm for calculating traffic performance measures. A demand-to-capacity ratio can be calculated for each cell, and the cell values should be reviewed to see whether in fact the time-space domain is free of congestion on its boundary and whether oversaturated flow conditions will occur anywhere in the time-space domain.

EXHIBIT 22-8. ADJUSTED SPEED-FLOW CURVES FOR INDICATED CAPACITY ADJUSTMENTS (SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumptions: FFS = 120 km/h, capacity adjustment factor (CAF) of 1.0, 0.95, 0.90, and 0.85.

Demand/capacity should be less than 1.0 for all cells along the four boundaries of the time-space domain

The demand-to-capacity ratio values should be less than 1.0 for all cells along the four boundaries of the time-space domain. If they are not, further analysis may be flawed and in some cases should not be undertaken. For example, if any cell in the first time interval has a demand-to-capacity ratio value greater than 1.0, there may have been oversaturated conditions in earlier time intervals without transfer of unsatisfied demand into the time-space domain. If any cell in the last time interval has a demand-to-capacity ratio greater than 1.0, the analysis will not be complete, since the unsatisfied demand within the time-space domain cannot be transferred to later time intervals. If any cell in the last downstream segment has a demand-to-capacity ratio greater than 1.0, there may be downstream bottlenecks that should be checked before proceeding with the analysis. Finally, if any cell in the first upstream segment has a demand-to-capacity ratio greater than 1.0, then oversaturation will be extended upstream of the freeway facility, but its effect will not be analyzed within the time-space domain.

These checks do not assure the analyst that the boundaries may not be violated later as the result of the more detailed analysis. If the initial checks indicate that demands exceed capacities at the boundary segments, the problem analysis domain should be adjusted. As the analysis is undertaken, the problem of demand exceeding capacity may occur again at the time-space domain boundaries, requiring that the problem be reformulated or that other techniques as described in Part V of this manual be considered. For example, oversaturated conditions at a downstream bottleneck may be so severe as to extend upstream into or beyond the first freeway segment or beyond the last time interval.

Another important check is to observe whether any cell in the entire time-space domain has a demand-to-capacity ratio value greater than 1.0. If all cells have demand-to-capacity ratio values less than 1.0, then the entire time-space domain contains undersaturated flow conditions, and the analysis is greatly simplified.

If any cell in the time-space domain has a demand-to-capacity ratio value greater than 1.0, then the time-space domain will contain both undersaturated and oversaturated flow conditions. Analysis of oversaturated flow conditions is much more complex because of the interactions between freeway segments.

The analysis begins in the first cell in the upper left-hand corner of the time-space domain (first segment in first time interval) and continues downstream along the freeway

Four-step process to analyze bottlenecks:

1. Bottleneck cell analysis
2. Downstream demand modifications
3. Upstream flow modifications
4. Demand transfer to next time interval

Shock wave analysis is used to analyze queue backup

facility for each segment in the first time interval. The analysis then returns to the first upstream segment in the second time interval and continues downstream along the freeway for each segment in the second time interval. This process is continued until all cells in the time-space domain have been analyzed.

As each cell is analyzed, the question is asked, Is the cell demand-capacity ratio less than or equal to 1.0? If the answer is yes, then the cell is not a bottleneck and is assumed to be able to handle all traffic that wants to enter. This process is continued in the sequence order described in the preceding paragraph until an answer of no is encountered, indicating that the demand-capacity ratio in this cell is greater than 1.0. This cell is identified as a bottleneck, and the traffic that wishes to enter cannot do so. The following four-step process is required to analyze each cell identified as a bottleneck: bottleneck cell analysis, downstream demand modifications, upstream flow modifications, and demand transfer to next time interval.

Since the demand in the bottleneck cell exceeds the bottleneck cell capacity, the flow in the cell will be equal to capacity, not demand. Each bottleneck cell will have a volume-capacity ratio of exactly 1.0. On the basis of this volume-capacity ratio, traffic performance measures can be estimated.

Since the bottleneck cell can only pass a flow equal to capacity (not demand) to the downstream segments in this time interval, the demands for all downstream cells must be modified in accordance with the destinations of the unsatisfied demand at the bottleneck.

The unsatisfied demand at the bottleneck cell must be stored in the upstream segment(s), and flow conditions and traffic performance measures in the upstream segments must be modified. This is accomplished through shock wave analysis.

The unsatisfied demand stored upstream of the bottleneck cell must be transferred to the next time interval. This is accomplished by adding the unsatisfied demand by desired destination to the origin-destination table of the next time interval.

This four-step process is implemented for each bottleneck encountered following the specified sequence of analyzing cells. If no bottleneck cells are encountered, then the entire time-space domain will have undersaturated conditions, and the sequence of analysis for oversaturation is not used. If major bottlenecks are encountered, the storage of unsatisfied demand may extend beyond the upstream boundary of the freeway facility or beyond the last time interval of the time-space domain. In such cases, the analysis will be flawed, and the time-space domain should be reformulated.

Once traffic performance measures have been estimated for each cell in the time-space domain, they can be aggregated for the entire freeway facility for each time interval and for the entire study time duration. The methodology for calculating these freeway traffic performance measures is described in the following section.

UNDERSATURATED CONDITIONS

The analysis begins by examining the demand-to-capacity ratios for all segments during the first time interval. If all segments have a demand-to-capacity ratio less than 1.0, then this time interval is completely undersaturated. The flow (or volume) is equal to demand for each cell, and undersaturated flow conditions occur. Performance measures for the first segment during the first time interval are calculated by using the methodology for the corresponding segment type in Chapters 23, 24, and 25.

The analysis continues to the next downstream freeway segment in the same time interval, and the performance measures are calculated for all subsequent downstream segments. The analysis then resumes in the second time interval from the furthest upstream segment and moves downstream until all freeway segments in that time interval have been analyzed. This pattern continues until the methodology encounters a time interval having one or more segments with a demand-to-capacity ratio greater than 1.0. If this occurs, the oversaturated analysis module is executed.

When the analysis moves from isolated segments to a system, an additional constraint may be necessary. A maximum achievable speed constraint is imposed to limit the predicted speed downstream of a segment experiencing low speeds. This process prevents large speed fluctuations that can be predicted when applying the segment-based methods in Chapters 23, 24, and 25.

Additional system considerations:

- Speed on a segment may be constrained by a slower speed on an upstream segment
- Speed estimate is made for the full roadway width in a ramp influence area

OVERSATURATED CONDITIONS

Node defined

Once oversaturation is encountered, the methodology changes its temporal and spatial units of analysis. The spatial units become nodes and segments, and the temporal unit moves from a time interval of 15 min to smaller time steps as recommended in Appendix A. A node is defined as the junction of two segments. There is always one more node than segment, with a node analyzed at the beginning and end of the freeway facility as shown in Exhibit 22-9. The numbering of nodes and segments begins at the upstream end and moves to the downstream end, with the segment upstream of Node (i) numbered (i – 1) and the downstream segment numbered (i), as shown in Exhibit 22-10. The oversaturated analysis moves from the first node to each downstream node for a time step. After the completion of a time step, the same nodal analysis is performed for the following time steps. Many flow variables are computed in this analysis, with the most prominent described in the next section.

EXHIBIT 22-9. NODE-SEGMENT REPRESENTATION OF A FREEWAY FACILITY

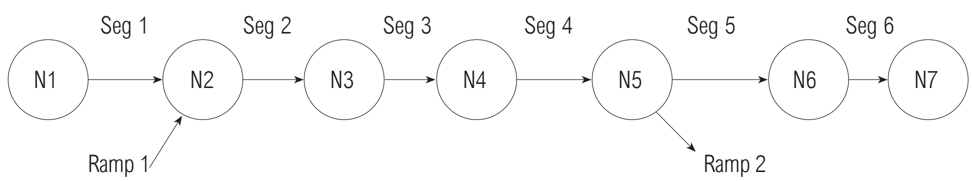
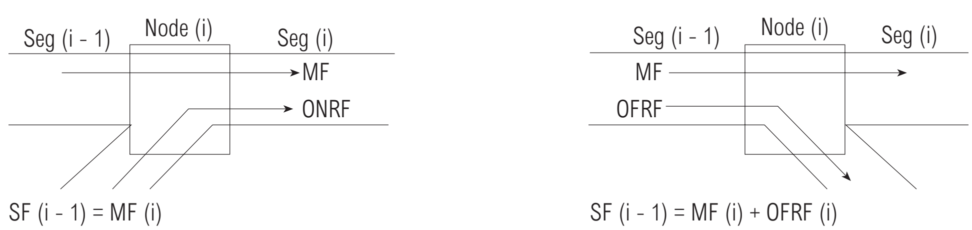


EXHIBIT 22-10. MAINLINE AND SEGMENT FLOW AT ON- AND OFF-RAMPS



Flow Fundamentals

Time steps of less than 15 min are used

Segment flow rates are calculated in each time step and are used to calculate the number of vehicles on each segment at the end of every time step. The number of vehicles on each segment is used to track queue accumulation and discharge and to calculate the average segment density.

The conversion from time intervals to time steps occurs during the first oversaturated time interval, and time steps continue in use until the end of the analysis. The transition to time steps is essential because at certain points in the methodology future performance is estimated from past performance of an individual variable. Use of time steps also allows more accurate tracking of queues.

Freeway analysis depends on the relationships between speed, flow, and density. Chapters 23, 24, and 25 define a relationship between these variables and the calculation of performance measures in the undersaturated regime. The methodology uses this relationship in the calculations for undersaturated segments. Calculations for segments performing in the oversaturated regime use a simplified linear flow-density relationship in the congested region, as detailed in Appendix A.

Segment Initialization

For the number of vehicles on each segment at the various time steps to be calculated, the segments must contain the proper number of vehicles before the queuing analysis places unserved vehicles on segments. The initialization of each segment is described below.

A simplified queuing analysis is initially performed to account for the effects of upstream bottlenecks. These bottlenecks meter traffic downstream. To obtain the proper number of vehicles on each segment, the segment's expected demand is calculated. The expected demand is based on demands for and capacities of the segment including the effects of all upstream segments. Expected demand represents the flow of traffic that would be expected to arrive at each segment if all queues were stacked vertically (i.e., if queues had no upstream effects). Thus, all segments upstream of a bottleneck have expected demands equal to actual demands. For the bottleneck and all further downstream segments, a capacity constraint at the bottleneck (which meters traffic to downstream segments) is applied in the computation of expected demand. From the expected segment demand, the background density can be obtained for each segment using the appropriate segment density estimation procedures from Chapters 23, 24, and 25.

Mainline Flow Calculation

Flows analyzed in oversaturated conditions are calculated for every time step and are expressed in vehicles per time step. They are analyzed separately on the basis of the origin and destination of the flow across the node. The flow from the mainline upstream Segment (i – 1) to mainline downstream Segment (i) is the mainline flow (MF). The flow from the mainline to an off-ramp is the off-ramp flow (OFRF). The flow from an on-ramp to the mainline is the on-ramp flow (ONRF). Each of these flows is shown in Exhibit 22-10 with its origin and destination and relationship to Segment (i) and Node (i).

The segment flow is the total output of a segment, as shown in Exhibit 22-10. The mainline flow is calculated as the minimum of six values. These constraints are the mainline input, Mainline Output 1, Mainline Output 2, Mainline Output 3, the upstream Segment (i – 1) capacity, and the downstream Segment (i) capacity.

Mainline Input

The mainline input is the number of vehicles that wish to travel through a node during the time step. The calculation includes the effects of bottlenecks upstream of the subject node. The effects include the metering of traffic during queue accumulation and the presence of additional traffic during queue discharge.

The mainline input is calculated by taking the number of vehicles entering the node upstream of the analysis node, adding on-ramp flows or subtracting off-ramp flows, if needed, and adding to it the number of unserved vehicles on the upstream segment. This is the maximum number of vehicles that desire to enter a node during a time step.

Mainline Output

The mainline output is the maximum number of vehicles that can exit a node, constrained by downstream bottlenecks or by merging traffic. Different constraints on the output of a node result in three separate types of mainline outputs (MO1, MO2, and MO3).

Mainline Output from Ramps

Mainline Output 1 (MO1) is the constraint caused by the flow of vehicles from an on-ramp. The capacity of an on-ramp segment is shared by two competing flows. This on-ramp flow limits the flow from the mainline at this node. The total flow that can pass the node is estimated as the minimum of the Segment (i) capacity and the mainline outputs (MO2 and MO3 below) calculated in the preceding time step.

Mainline Output from Segment Storage

The output of mainline flow through a node is also constrained by the growth of queues on the downstream segment. The presence of a queue limits the flow into the segment once the queue reaches its upstream end. The queue position is calculated from shock wave analysis.

The MO2 limitation is determined first by calculating the maximum number of vehicles allowed on a segment at a given queue density. The maximum flow that can enter a queued segment is the number of vehicles leaving the segment plus the difference between the maximum number of vehicles allowed on a segment and the number of vehicles already on the segment. The queue density is determined from the linear, congested portion of the density-flow relationship shown in Appendix A.

Mainline Output from Front-Clearing Queue

The final limitation on exiting mainline flows at a node is caused by front-clearing downstream queues or MO3. These queues typically occur when temporary incidents clear. Two conditions must be satisfied. First, the segment capacity (minus the on-ramp demand if present) for the current time interval must be greater than the segment capacity (minus on-ramp demand) in the preceding time interval. The second condition is that the segment capacity minus the ramp demand for the current time interval be greater than the segment demand in the same interval.

Front-clearing queues do not affect the segment throughput (which is limited by the queue throughput) until the recovery wave has reached the upstream end of the segment. The shock wave speed is estimated from the slope of the line connecting the bottleneck throughput and the segment capacity points.

Mainline Flow

The mainline flow across Node (i) is the minimum of the following variables: Node (i) mainline input, Node (i) Mainline Output 1, Node (i) Mainline Output 2, Node (i) Mainline Output 3, Segment (i – 1) capacity, and downstream Segment (i) capacity.

Determining On-Ramp Flow

The on-ramp flow is the minimum of the on-ramp input and output. Ramp input in a time interval is the ramp demand plus any unserviced ramp vehicles from a previous time interval.

On-ramp output is limited by the ramp roadway capacity and the ramp-metering rate. It is also affected by the volumes on the mainline segments. The latter is a very complex process that depends on the various flow combinations on the segment, the segment capacity, and the ramp roadway volumes. Details of the calculations are presented in Appendix A.

Determining Off-Ramp Flow

The off-ramp flow is determined by calculating a diverge percentage based on the segment and off-ramp demands. The diverge percentage varies only by time interval and remains constant for vehicles that are associated with a particular time interval. If there is an upstream queue, then there may be metering of traffic to this off-ramp. This will cause a decrease in the off-ramp flow. When the vehicles that were metered arrive in the next time interval, they use the diverge percentage associated with the preceding time interval. The methodology ensures that all off-ramp vehicles prevented from exiting during the presence of a bottleneck are appropriately discharged in later time intervals.

Ramp diverge percentages may vary by time interval, and this will be affected by queuing at bottlenecks

Determining Segment Flow

The segment flow is the number of vehicles that flow out of a segment during the current time step. These vehicles enter the current segment either to the mainline or to an off-ramp at the current node as shown in Exhibit 22-9. The number of vehicles on each segment is calculated on the basis of the following: the number of vehicles that were on the segment in the previous time step, the number of vehicles that have entered the segment in the current time step, and the number of vehicles that can leave the segment in the current time step. Because the number of vehicles that leave a segment must be known, the number of vehicles on the current segment cannot be determined until the upstream segment is analyzed.

The number of unserved vehicles stored on a segment is calculated as the difference between the number of vehicles on the segment and the number of vehicles that would be on the segment at the background density.

Determining Segment Service Measures

In the last time step of a time interval, the segment flows in each time step are averaged over the time interval, and the measures of effectiveness for each segment are calculated. If there were no queues on a particular segment during the entire time interval, then the performance measures are calculated from Chapters 23, 24, and 25 as appropriate.

Three-step procedure

If there was a queue on the current segment during the time interval, then the performance measures are calculated in three steps. First, the average number of vehicles over a time interval is calculated for each segment. Next, the average segment density is calculated by taking the average number of vehicles in all time steps in the time interval and dividing it by the segment length. Finally, the average speed on the current segment during the current time interval is calculated as the ratio of segment flow to density. The final segment performance measure is the length of the queue at the end of the time interval (if it exists), which is calculated from shock wave theory.

Queues on ramps

Queue length on on-ramps can also be calculated. A queue will form on the on-ramp roadway only if the flow is limited by a meter or by freeway traffic in the gore area. If the flow is limited by the ramp roadway capacity, unserved vehicles will be stored on a facility upstream of the ramp roadway, most likely a surface street. The methodology does not account for this delay. If the queue is on the ramp roadway, the queue length is calculated by using the difference in background and queue densities.

DETERMINING FREEWAY FACILITY PERFORMANCE MEASURES

The previously discussed traffic performance measures can be aggregated over the length of the freeway facility, over the time duration of the study interval, or over the entire time-space domain.

Facilitywide measures:
- Trip time
- Vehicle and person distance of travel
- Vehicle and person-hours of travel

Aggregating the estimated traffic performance measures over the entire length of the freeway facility provides facilitywide estimates for each time interval. Averages and cumulative distributions of speed and density for each time interval can be determined, and patterns of their variation over the connected time intervals can be assessed. Trip times, vehicle (and person) distance of travel, and vehicle (and person) hours of travel can be computed and patterns of their variation over the connected time intervals can be assessed.

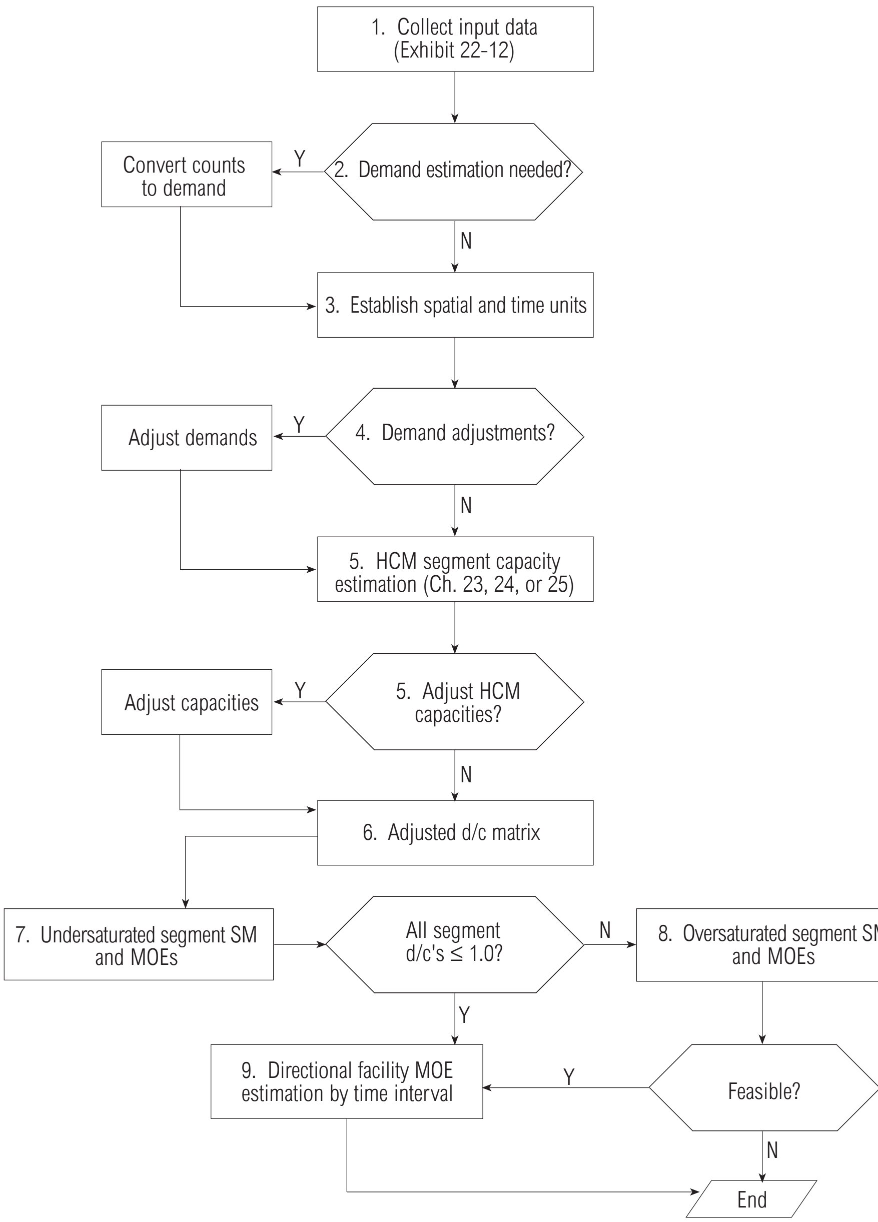
Aggregating the estimated traffic performance measures over the time duration of the study interval provides an assessment of the performance of each segment along the freeway facility. Average and cumulative distributions of speed and density for each segment can be determined, and patterns of the variation over connected freeway segments can be compared. Average trip times, vehicle (and person) distance of travel, and vehicle (and person) hours of travel are easily assessed for each segment and compared.

III. APPLICATIONS

The methodology is applied in the sequence indicated in Exhibit 22-11. The process consists of nine steps and is based on the methodology described in the preceding section. A detailed flowchart that processes the oversaturation portion of the analysis is provided in Appendix A.

Guidelines on required inputs and estimated values are given in Chapter 13

EXHIBIT 22-11. IMPLEMENTATION OF FREEWAY FACILITY METHODOLOGY



Notes:
d/c = demand-to-capacity ratio.
SM = service measure.
MOE = measure of effectiveness.

COMPUTATIONAL STEPS

1. Collect input data for the directional facility. Provide guidance on limits of congestion in time and space. Document any demand and capacity adjustments that should be considered in the analysis. The input data define the time-space domain of the

freeway facility. The data include the specification of the facility in terms of length, number of sections, and geometric attributes. Exhibit 22-12 summarizes the inputs required to perform an analysis.

EXHIBIT 22-12. REQUIRED INPUT DATA FOR FREEWAY FACILITY ANALYSIS

Geometric Data for Each Section
• Section length (m) • Mainline number of lanes • Mainline average lane width (m) • Mainline lateral clearance (m) • Terrain (level, rolling, or mountainous) • Ramp number of lanes • Ramp acceleration or deceleration lane length (m)
Traffic Characteristics Data
• Mainline free-flow speed (km/h) (optional) • Vehicle occupancy (passengers/veh) • Percent trucks and buses (%) • Percent recreational vehicles (%) • Driver population (commuter or recreational) • Ramp free-flow speeds (km/h)
Demand Data
• Mainline entry demand for each time interval • On-ramp demands for each time interval • Off-ramp demands for each time interval • Weaving demand on weaving segments

2. Check whether adjustments from counts to estimate demands are needed. If the demands represent actual counts from a freeway facility (for example, from a freeway management system) and the system is experiencing oversaturation, an adjustment from counts to demands may be carried out in this step.

3. Establish spatial and time analysis units. Convert sections to segments as described, calculate time step for oversaturation, and establish other time units such as time intervals and analysis duration.

Time steps can range from 15 to 60 s

Spatially, the HCM analysis unit is a segment. On the basis of the definitions of ramp influence areas (450 m upstream of off-ramps and downstream of on-ramps as indicated in Chapter 25), sections are subdivided into segments. Similarly, weaving sections are defined as having a maximum length of 750 m. The analysis duration can vary from one to twelve 15-min intervals. Demand and capacity rates are fixed during an interval. The time step for oversaturation analysis depends on the length of the shortest segment on the facility and can vary from 15 to 60 s.

4. The procedure permits manual adjustments of segment demands. This may encompass the application of overall growth factors to test the adequacy of the facility to meet projected demands or simulate the effect of demand diversion onto adjacent facilities. The factors can be applied to individual origin and destination points.

5. Calculate segment capacity using HCM methods and adjust capacity as needed. Using the segment analysis methodologies of Chapters 23 through 25, segmentwide capacities in vehicles per hour are computed. These values are assumed to reflect normal capacity conditions. If the user is interested in adjusting capacities to reflect field measurements or to simulate a capacity reduction occurrence such as an incident or a work zone, a capacity adjustment factor is introduced. This factor changes both the capacity value and the speed-flow relationship for the affected segment during the affected time intervals.

6. Generate an adjusted demand-to-capacity (d/c) matrix by segment and time interval. Identify whether this facility is completely undersaturated or has some oversaturated time intervals.

Each segment demand is divided by its corresponding capacity in each time interval. The resulting d/c matrix is then used to evaluate the feasibility of the analysis and to identify which intervals have oversaturated segments.

The segment procedures are applied to undersaturated time intervals. The analyst is referred to the speed, density, and LOS estimation methods for basic, weaving, and ramp segments in Chapters 23, 24, and 25, respectively.

7. For the first time interval with d/c > 1.0 for some segment, begin using the reduced time step to carry out all computations. Calculate the position of bottlenecks and queues in each time step. Use appropriate flow regimes (undersaturated for segments with no queues and oversaturated for segments with queues) to estimate speeds and densities on each segment. Aggregate measures of effectiveness (MOEs) for each segment by time interval. Proceed to the next time interval until all time intervals in the period are analyzed.

The purpose of the oversaturated analysis is to calculate the actual flows on and the number of vehicles occupying each segment. By comparing the current number of vehicles on a segment with the number of vehicles that would be expected on it at the background density, segment queues can be identified and tracked each minute. The smaller time step is necessary to ensure that fast-growing queues do not jump over a short segment if not updated frequently.

8. The bottleneck analysis begins by setting the flow-to-capacity ratio on that segment to 1.0. The unmet demand is transferred to the next time interval, and the reduced flow rate through the bottleneck is propagated upstream in the form of a queue whose density depends on the severity of the bottleneck. When an upstream on-ramp is encountered, its flow rate is calculated on the basis of the level of congestion on the segment immediately downstream of the on-ramp and the magnitude of mainline and on-ramp flows at that node. Downstream of the bottleneck, flows are metered at the bottleneck capacity rate, which may result in starving subsequent mainline segments and off-ramp flows. Only when the bottleneck effects clear (when demands drop or capacity increases) do the flows on downstream segments increase to serve the unmet demand from the preceding time interval. Given adequate time, the flows will catch up with demand, and undersaturated operations will resume.

9. Aggregate individual segment MOEs into a directional facility MOE for each time interval. Examples include average speed, density, vehicle kilometers of travel (VkmT), vehicle hours of travel (VHT), vehicle hours of delay (VHD), and travel time. Facilitywide performance measure calculations by time interval are detailed in Appendix A.

TRAFFIC MANAGEMENT STRATEGIES

The methodology for freeway facilities has incorporated procedures for the assessment of a variety of traffic management strategies. The methodology permits the modification of previously calculated cell demands or capacities (or both) within the time-space domain to assess a traffic management strategy or a combination of strategies, as described below.

1. A growth factor parameter has been incorporated to evaluate traffic performance when traffic demands are higher or lower than the demand calculated from the traffic counts. This parameter would be used to undertake a sensitivity analysis of the effect of demand on freeway performance and to evaluate future scenarios. In these cases, all cell demand estimates are multiplied by the growth factor parameter.

2. The effect of a predetermined ramp-metering plan can be evaluated by modifying the ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to the desired metering rate. This feature permits both evaluation of a

predetermined ramp-metering plan and experimentation to obtain an improved ramp-metering plan.

3. Freeway design improvements can be evaluated within this methodology by modifying the design features of any portion or portions of the freeway facility. For example, the effect of adding an auxiliary lane at a critical location can be assessed. The effect of adding merging or diverging lanes can also be assessed.

4. Reduced-capacity situations can be investigated. The capacity in any cell or cells of the time-space domain can be reduced to represent situations such as construction and maintenance activities, adverse weather, and traffic accidents and vehicle breakdowns.

5. An independent HOV facility can be evaluated with this methodology. The analysis is similar to that of a freeway facility without an HOV lane. The methodology does not permit the analysis of concurrent HOV lanes.

User demand responses such as spatial, temporal, modal, or total demand responses caused by a traffic management strategy are not automatically incorporated into the methodology. On viewing the new freeway traffic performance results, the user can modify the demand input manually to evaluate the effect of anticipated demand responses.

As stated earlier, these traffic management strategies can be evaluated individually or in combinations. For more complex traffic management strategies for which the chapter methodology is not appropriate (such as concurrent HOV-lane freeways or significant demand responses), refer to Part V of this manual.

IV. EXAMPLE PROBLEMS

Problem No.	Description
1	Fully undersaturated directional freeway facility with 6 sections and 11 segments
2	Application of demand growth factors resulting in recurring oversaturation
3	Treatment of oversaturation by means of geometric improvements of the facility
4	Effect of temporary capacity reduction due to incident
5	Effect of reduced incident response time on freeway facility operation
6	Effect of ramp metering on freeway facility operation

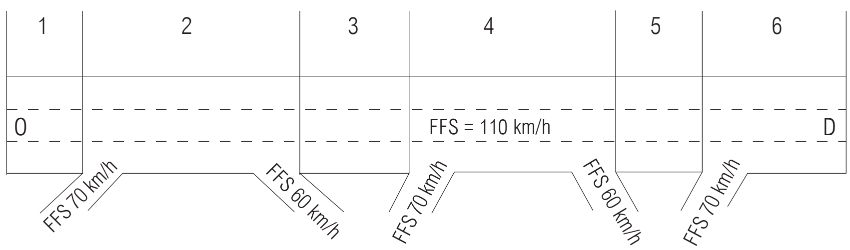
EXAMPLE PROBLEM 1

The Facility The freeway facility is operating under capacity, and traffic is expected to grow in the near future.

The Question What is the capacity and level-of-service profile for the given directional freeway facility under existing conditions?

The Facts
√ The facts known about this freeway facility are shown in the exhibits below.
√ Acceleration and deceleration lanes are 100 m long.
√ Each time interval is 15 min, and all demands are expressed as hourly flow rates during each time interval.

SYSTEMWIDE INPUT DATA REQUIREMENTS



Section Characteristics	Freeway Section 1	2	3	4	5	6
Length (m)	300	2200	800	700	350	1150
Number of lanes	3	3	3	3	3	3
Mainline FFS (km/h)	110	110	110	110	110	110
Vehicle occupancy (pass/veh)	1.20	1.20	1.20	1.20	1.20	1.20
Lane width (m)	N/A	N/A	N/A	N/A	N/A	N/A
Lateral clearance (m)	N/A	N/A	N/A	N/A	N/A	N/A
Trucks (%)	3	3	3	3	3	3
RVs (%)	0	0	0	0	0	0
Terrain	Level	Level	Level	Level	Level	Level
Driver population	Commuter	Commuter	Commuter	Commuter	Commuter	Commuter

Note:
Each ramp has one lane.
N/A - not available.

INPUT DEMANDS

Time Interval	Entry Mainline (O)	On-Ramp O1	On-Ramp O2	On-Ramp O3	Off-Ramp D1	Off-Ramp D2	Exit Mainline (D)
1	4796	756	1456	648	656	560	6440
2	4772	973	1164	636	588	477	6480
3	4700	1002	1712	596	636	802	6572
4	4164	555	1548	580	520	608	5719
5	3727	485	1180	484	632	448	4796

Outline of Solution The facility is analyzed assuming undersaturated conditions. First, the facility is divided into segments to enable the application of the segment analysis methods in Chapters 23 through 25. The performance results are summarized by each time interval and across time intervals using appropriate tables and charts. The steps below follow Exhibit 22-11.

Steps

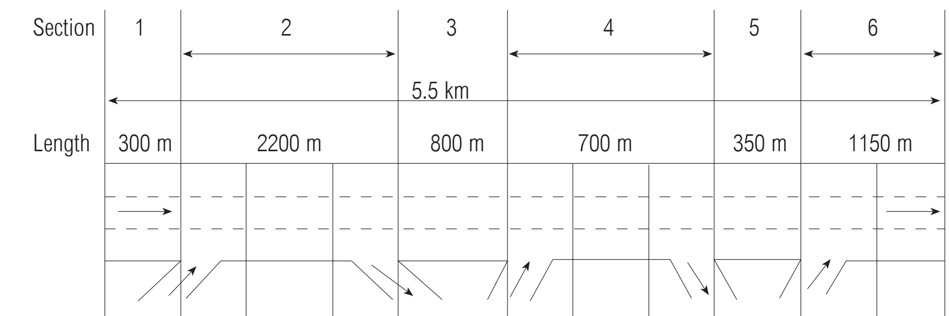
1. Collection of input data: The input data are summarized in the tables. Note the heavy ramp demand volumes for On-Ramp O2, which exceed 1,700 veh/h in Time Interval 3. These demands are still below the ramp roadway capacity, estimated at about 2,000 veh/h for the ramp FFS of 70 km/h. Thus, whereas there may be no capacity problem on the ramp roadway proper, these demands may cause a merge problem on the segment immediately downstream of that ramp.

2. Demand estimation: No adjustments are necessary at this stage since the facility has been observed to operate under capacity.

3. Establishment of spatial and time units: Using the definition of ramp influence area, the original 6 sections are further subdivided into 11 analysis segments. The conversion is shown graphically in the exhibit below. Section 4, with no auxiliary lanes and less than 900 m long, contains an overlap segment (7) that is labeled R. This segment's performance is calculated as the worse of Segments 6 and 8. The time intervals have been set at 15 min. Furthermore, since the shortest segment length is 250 m, a time step of 1 min is sufficient to carry out the oversaturated analysis.

4. Demand adjustments: The values in the Input Demands table are used directly to calculate segment demands by adding or subtracting ramp demands at each section.

CONVERSION OF FACILITY SECTIONS INTO SEGMENTS



Segment #	1	2	3	4	5	6	7	8	9	10	11
Type	B	ONR	B	OFR	B	ONR	R	OFR	B	ONR	B
Length	300 m	450 m	1300 m	450 m	800 m	250 m	200 m	250 m	350 m	450 m	700 m

5. HCM segment capacity estimation and adjustment: The facility has five basic freeway segments (numbered 1, 3, 5, 9, and 11), three on-ramp segments (2, 6, and 10), two off-ramp segments (4 and 8), and the overlap segment (7). For each segment type, the appropriate HCM chapter (23 or 25) is consulted and the segment capacity computed. The major difference in this chapter is that all segment capacities are expressed in units of vehicles per hour. No adjustments of the estimated capacities are needed.

6. Adjusted d/c matrix: After capacities are computed, the d/c matrix is generated for each segment and time interval. Both segment capacity and d/c ratios are shown in the exhibit below. As suspected, all segments have d/c ratios less than 1.0, and therefore a complete undersaturated analysis can be carried out. A review of the matrix indicates that Time Intervals 1 through 3 are critical and that Segments 6 through 8, 10, and 11 have d/c ratios above 0.90 during those three intervals. Traffic demands subside considerably in Time Intervals 4 and 5, with a maximum d/c ratio of 0.83 on Segment 8 in Time Interval 4.

ESTIMATED CAPACITY AND d/c RATIO MATRIX

Time	Segment Number and Type										
Time Interval	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	0.69	0.80	0.80	0.80	0.70	0.91	0.91	0.91	0.83	0.93	0.93
2	0.69	0.83	0.83	0.83	0.74	0.91	0.91	0.91	0.84	0.93	0.93
3	0.68	0.82	0.82	0.82	0.73	0.98	0.98	0.98	0.86	0.95	0.95
4	0.60	0.68	0.68	0.68	0.60	0.83	0.83	0.83	0.74	0.82	0.82
5	0.54	0.61	0.61	0.61	0.52	0.69	0.69	0.69	0.62	0.69	0.69
Capacity	6946	6946	6946	6946	6946	6946	6946	6946	6946	6946	6946

7. Undersaturated segment service measure and MOEs: The methods in Chapters 23 through 25 are applied to estimate individual segment speeds, densities, and travel times. The two exhibits below summarize the results for segment speed, density (the service measure), and LOS for the entire time-space domain.

ESTIMATED SEGMENT SPEEDS (km/h)

	Segment Number and Type										
Time Interval	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	109.6	94.5	106.2	96.5	109.4	86.9	86.9	93.3	104.1	91.1	94.9
2	109.7	93.4	104.5	96.6	108.6	89.2	89.2	94.9	103.5	90.8	94.2
3	109.8	93.4	104.9	96.5	108.9	77.8	77.8	91.3	102.0	90.0	92.4
4	110.0	96.9	109.8	97.1	109.5	90.3	90.3	92.1	106.3	94.4	104.8
5	110.0	98.0	109.9	96.7	109.5	95.8	93.8	93.8	106.7	96.8	109.4

ESTIMATED SEGMENT DENSITIES (veh/km/ln) AND LOS

	Segment Number and Type										
Time Interval	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	14.6 C	19.2 D	17.4 D	19.1 D	14.9 C	21.6 D	22.4 E	22.4 E	18.6 D	22.4 E	22.6 E
2	14.5 C	19.8 D	18.3 D	19.8 D	15.8 C	21.6 D	22.1 E	22.1 E	18.8 D	22.5 E	22.9 E
3	14.3 C	19.6 D	18.1 D	19.6 D	15.5 C	22.9 D	24.1 E	24.1 E	19.5 D	22.8 E	23.7 E
4	12.6 C	16.5 C	14.3 C	16.3 C	12.8 C	19.6 D	20.5 D	20.5 D	16.1 D	19.9 D	18.2 D
5	11.3 C	14.8 C	12.8 C	16.6 C	10.9 B	16.4 C	16.9 C	16.9 C	13.5 C	16.8 C	14.6 C

8. Oversaturated segment service measure and MOEs: Does not apply in this case since the facility is fully undersaturated.

9. Directional facility MOE estimation: The individual segment performance measures are aggregated for each time interval. The exhibit below summarizes these results. Note that the average speed is defined as the ratio of vehicle kilometers to vehicle hours of travel in each time interval and therefore does not consider the effect of any on-ramp delays. On the other hand, vehicle hours of delay is the sum of mainline delays and ramp delays. Mainline delays are computed as the difference between total mainline travel time and free-flow travel time.

SUMMARY OF FACILITYWIDE PERFORMANCE MEASURES

Time Interval	Performance Measures					
	Vehicle-km of Travel	Vehicle-h of Travel	Vehicle-h of Delay	Average Speed (km/h)	Average Density (veh/km/ln)	Facility Travel Time (min)
1	7862	79.5	8.0	98.9	18.9	3.31
2	8030	81.4	8.4	98.7	19.4	3.33
3	8100	83.9	10.3	96.5	19.7	3.39
4	6847	67.1	4.8	102.1	16.1	3.22
5	5901	56.9	3.2	103.8	13.8	3.17
Overall	36 740	368.8	34.7	99.6	-	3.30

Results It is evident from the results that the facility provides free-flow conditions. Time Intervals 1 through 3 are fairly similar, with average speeds varying within a 3-km/h range and densities slightly below 20 veh/km/ln. Time Intervals 4 and 5 have higher average speeds exceeding 102 km/h and average densities under 17 veh/km/ln.

EXAMPLE PROBLEM 2

The Facility In this example, the facility described in Example Problem 1 is evaluated under revised traffic demands.

The Question What is the capacity and level-of-service profile for a directional freeway facility using the revised demands?

The Facts

√ The facts shown in Example Problem 1 apply.

√ Demand is adjusted upward by 6 percent uniformly. The new demand rates are shown in the exhibit below.

REVISED INPUT DEMANDS

Time Interval	Entry Mainline (O)	On-Ramp O1	On-Ramp O2	On-Ramp O3	Off-Ramp D1	Off-Ramp D2	Exit Mainline (D)
1	5084	801	1543	689	695	594	6826
2	5058	1031	1234	674	623	506	6869
3	4982	1062	1815	632	674	850	6966
4	4414	588	1641	615	551	644	6062
5	3951	514	1251	513	670	475	5084

Outline of Solution Since the base demands and capacities have not changed, Steps 1 to 3 of Example Problem 1 are skipped. The analysis begins by adjusting demands and then proceeds to determine whether oversaturated conditions will prevail. If they do, a shorter time step will be used to track the position of the queues, the location of the bottlenecks, and their effect on both mainline and ramp flows, speeds, and densities.

Steps

1. Demand adjustments: The input demands shown in the exhibit above represent increases of 6 percent compared with Example Problem 1.

2. HCM capacity estimation and adjustment: Normally, no adjustments of the capacities computed in Example Problem 1 are needed since the facility geometrics are fixed. There is evidence, however, that when queuing occurs on a segment, the discharge flow rate in the queue may be less than the HCM-estimated capacity (by 3 to 5 percent). The HCM capacities assume undersaturated flow conditions. To implement a variable segment capacity under queuing, the analyst must first identify which, if any, segments have a queue and then make a second run with a reduced capacity for those segments using the capacity adjustment factor. For simplicity, in this example the HCM capacity is assumed to apply to queued segments as well.

3. Adjusted d/c matrix: Using the adjusted demands, a revised d/c matrix, shown in the exhibit below, is generated. Cells having d/c > 1.0 are underlined.

ESTIMATED CAPACITY AND d/c RATIO MATRIX

Time Interval	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 B	8 OFR	9 B	10 ONR	11 B
1	0.73	0.85	0.85	0.85	0.75	0.97	0.97	0.97	0.88	0.98	0.98
2	0.73	0.88	0.88	0.88	0.79	0.96	0.96	0.96	0.89	0.99	0.99
3	0.72	0.87	0.87	0.87	0.77	<u>1.03</u>	<u>1.03</u>	<u>1.03</u>	0.91	<u>1.003</u>	<u>1.003</u>
4	0.64	0.72	0.72	0.72	0.64	0.88	0.88	0.88	0.78	0.87	0.87
5	0.57	0.64	0.64	0.64	0.55	0.73	0.73	0.73	0.66	0.73	0.73
Capacity	6946	6946	6946	6946	6946	6946	6946	6946	6946	6946	6946

(Segment Number and Type)

The d/c matrix indicates that fully undersaturated conditions prevail in the first two and the last two time intervals. Two sets of bottlenecks occur in Time Interval 3. The first and more severe bottleneck is on Segments 6 through 8 with a d/c of 1.03. The second and less severe bottleneck is on Segments 10 and 11 with a d/c of 1.003. It is likely that the second bottleneck will be hidden as a result of the metering effect of the first one. Whether the two queues from the bottlenecks will overlap, thus violating an important constraint of the methodology, remains to be seen.

4. Undersaturated segment service measure and MOEs: The first two time intervals are undersaturated, and therefore all MOEs are derived directly from the procedures in Chapters 23, 24, and 25.

5. Oversaturated segment service measure and MOEs: Starting with Time Interval 3, the analysis is performed in 1-min increments, and a set of nodes is defined at each ramp terminal. First, flow rates are determined in each time step. Starting from the furthest upstream segment in Time Interval 3, flows across nodes are calculated until the bottleneck Segment 6 is reached. On Segment 6, flow is equated to capacity, and the residual demand is applied at that bottleneck in Time Interval 4. Upstream of Segment 6, the queue density is calculated, and queue length is tracked on Segments 5, 4, and so forth. Downstream of Segment 6, flows are metered at the segment capacity rate. The same process is applied to the bottlenecks on Segments 10 and 11. Since the demands in Time Intervals 4 and 5 drop significantly, queues will begin to clear and dissipate by the end of the analysis period. The exhibit below shows the actual volume-to-capacity ratios estimated on each segment and time interval. Note that volumes are averaged over the 15 time steps per interval and reflect the output flow for a segment. By definition no v/c ratio can exceed 1.00.

As anticipated, the bottleneck Segments 6 through 8 are operating at capacity in Time Interval 3. The metering effect of this bottleneck hides the second bottleneck on Segments 10 and 11, which have a v/c < 1.0 in Time Interval 3. A comparison of the d/c and v/c matrices indicates that flows exceed demands in Time Interval 4, which indicates that the unserved demand in Time Interval 3 is now being served in Time Interval 4. There are no differences in demand and flows in the last time interval, suggesting that the facility performance has fully recovered by the end of the analysis period.

ESTIMATED CAPACITY AND v/c RATIO MATRIX

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 B	8 OFR	9 B	10 ONR	11 B
1	0.73	0.85	0.85	0.85	0.75	0.97	0.97	0.97	0.88	0.98	0.98
2	0.73	0.88	0.88	0.88	0.79	0.96	0.96	0.96	0.89	0.99	0.99
3	0.72	0.87	0.87	0.87	0.77	1.00	1.00	1.00	0.88	0.97	0.97
4	0.64	0.72	0.72	0.72	0.64	0.91	0.91	0.91	0.81	0.90	0.90
5	0.57	0.64	0.64	0.64	0.55	0.73	0.73	0.73	0.66	0.73	0.73
Capacity	6946	6946	6946	6946	6946	6946	6946	6946	6946	6946	6946

The next performance measure investigated is the queue lengths observed on the mainline segments and on the ramps. These values represent instantaneous observations at the end of each time interval. The results are shown in the exhibit below, by segment and time interval. Blank entries indicate no queuing. The v/c matrix above indicates that a queue develops on the on-ramp roadway on Segment 6 in Time Interval 3. This queue is caused by the heavy on-ramp demand of 1,815 veh/h in that time interval. Since the mainline entering demand is under 1,800 veh/h/ln, no queuing occurs on the freeway mainline. The actual on-ramp flow is estimated at 1,576 veh/h, and the difference (1,815 – 1,576) causes a queue to develop and reach a length of 1189 m at the end of Interval 3. That queue is fully dissipated by the end of Time Interval 4. While there are no queues in Time Interval 5, the excess flows withheld in the previous intervals are served fully, and all vehicles are discharged at the end of the analysis period.

ESTIMATED QUEUE LENGTH (m) ON MAINLINE AND RAMPS (R) AT END OF EACH TIME INTERVAL

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1											
2											
3						1189 (R)					
4											
5											

To complete the oversaturated segment analysis, a summary of the resulting segment speeds, segment densities, and segment LOS for each time interval is given in the two exhibits below.

ESTIMATED SEGMENT SPEEDS (km/h)

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	108.9	93.2	103.1	96.2	108.4	82.5	82.5	93.0	99.8	87.5	86.9
2	109.0	91.7	100.5	96.3	106.8	85.8	85.8	94.6	99.0	87.0	85.8
3	109.2	91.8	101.8	96.2	107.5	75.7	75.7	92.0	100.0	88.3	88.5
4	110.0	96.3	109.1	97.0	109.5	83.8	83.8	90.6	105.3	92.2	97.8
5	110.0	97.5	109.9	96.6	109.5	94.8	93.6	93.6	106.7	96.7	108.9

ESTIMATED SEGMENT DENSITIES (veh/km/ln) AND LOS

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	15.6 C	20.3 D	19.0 D	20.3 D	16.0 D	22.9 E	23.8 E	23.8 E	20.5 D	23.6 E	26.2 E
2	15.5 C	21.0 D	20.2 D	21.0 D	17.1 D	22.9 E	23.4 E	23.4 E	20.9 D	23.8 E	26.7 E
3	15.2 C	20.8 D	19.9 D	20.8 D	16.7 D	23.5 E	24.5 E	24.5 E	20.4 D	23.4 E	25.4 E
4	13.4 C	17.4 D	15.3 C	17.2 D	13.5 C	21.4 D	22.8 E	22.8 E	17.9 D	21.9 D	21.4 D
5	12.0 C	15.6 C	13.5 C	15.4 C	11.6 C	17.3 D	17.9 D	17.9 D	14.3 C	17.7 D	15.6 C

6. Directional facility MOE estimation: The individual segment performance measures are aggregated for each time interval. An important addition in this example is the inclusion of two VkmT measures, the first based on demands, VkmT(D), and the second based on actual flow rates, VkmT(F). These values are used to detect whether vehicle storage [when VkmT(D) > VkmT(F)] or queue release [VkmT(D) < VkmT(F)] is occurring in each interval. Appendix A provides details of the computations needed to obtain the facilitywide measures. The exhibit below summarizes the facilitywide results by time interval.

SUMMARY OF FACILITYWIDE PERFORMANCE MEASURES

Time Interval	Performance Measures						
	Vehicle-km of Travel (Demand)	Vehicle-km of Travel (Flow Rate)	Vehicle-h of Travel	Vehicle-h of Delay	Average Speed (km/h)	Average Density (veh/km/ln)	Facility Travel Time (min)
1	8334	8334	87.2	11.5	95.5	20.5	3.43
2	8511	8511	89.6	12.2	95.0	21.5	3.45
3	8586	8466	89.7	20.2	94.4	20.9	3.47
4	7258	7379	74.1	10.9	99.5	17.6	3.28
5	6255	6255	60.4	3.6	103.5	14.7	3.18
Overall	38 944	38 945	401.0	58.4	97.1	-	3.4

Result It is instructive to compare the above results with those obtained in Example Problem 1. Whereas the total VkmT between the two problems increased only by 6 percent, the total vehicle hours of travel on the mainline increased by 8.9 percent. The total vehicle delay on the system, which includes estimated delay on the on-ramps, went up by 68 percent. If one compares the performance in the third time interval, the difference is even greater, with delays increasing by more than 96 percent. On average, the system density appears to have increased by 10 percent. Interestingly, the average speeds do not vary substantially. This may be due to boundary segments, which contribute significantly to the overall speed value by virtue of their length but typically experience little congestion.

EXAMPLE PROBLEM 3

The Facility In this example, the facility analyzed in Example Problem 2 is evaluated with revised geometry.

The Question What is the capacity and level-of-service profile for the directional freeway facility using the revised geometry?

The Facts
√ The facts from Example Problem 2 apply.
√ Recurring congestion is observed during the first hour downstream of on-ramp Segment 6 to the end of the study section. An auxiliary lane on the freeway mainline between Segments 6 and 8 over a distance of 700 m is proposed.

Outline of Solution The capacity of the upgraded segment (which is now a Type A weave) is first estimated. Its effects on the segment and facility performance measures are shown. For ease of reference, this segment is labeled "678." The total number of segments on the facility is reduced from 11 to 9.

Steps

1. The analysis of Weaving Segment 678 requires knowledge of weaving and nonweaving demand volume. In this example, weaving demands of 1,868, 1,340, 2,065, 1,925, and 1,326 veh/h are assumed to occur in Time Intervals 1 through 5, respectively.
2. HCM capacity estimation and adjustments: The number of lanes on Segments 6 through 11 is now adjusted to four. No changes in segment types or other geometric data are made.
3. Adjusted d/c matrix: The application of the methodology yields the revised d/c matrix and segment capacities indicated in the exhibit below. The auxiliary lane addition to Segment 678 was sufficient to restore undersaturated conditions on that segment. However, the second bottleneck on Segments 10 and 11 still exists. This implies that the improved facility can, for the most part, absorb the additional growth rate in traffic demands and still operate at an acceptable level.

ESTIMATED CAPACITY (veh/h) AND d/c RATIO MATRIX

Time Interval	Segment Number and Type								
	1 B	2 ONR	3 B	4 OFR	5 B	678 W	9 B	10 ONR	11 B
1	0.73	0.85	0.85	0.85	0.75	0.80 (8410)	0.88	0.98	0.98
2	0.73	0.88	0.88	0.88	0.79	0.75 (8946)	0.89	0.99	0.99
3	0.72	0.87	0.87	0.87	0.77	0.87 (8272)	0.91	1.003	1.003
4	0.64	0.72	0.72	0.72	0.64	0.76 (8055)	0.78	0.87	0.87
5	0.57	0.64	0.64	0.64	0.55	0.60 (8469)	0.66	0.73	0.73
Capacity	6946	6946	6946	6946	6946	(capacity)	6946	6946	6946

4. Undersaturated segment service measure and MOEs: The segment speeds, densities, and level of service for the upgraded facility are summarized in the exhibits below. In Time Interval 3, the oversaturated analysis is initiated.
5. Oversaturated segment service measures and MOEs: The now-active bottleneck on Segments 10 and 11 yields a queue 122 m long on Segment 9 in Time Interval 3. The

queue is dissipated at the start of Time Interval 4. More important, however, is that the geometric improvement on Segment 678 has eliminated the 1189-m queue that was observed on the ramp roadway in the preceding example.

6. Directional facility MOE estimation: The individual segment performance measures are aggregated for each time interval. The results are shown in the three exhibits below.

Results The upgraded facility performance is compared with that given in Example Problem 2. The mainline travel time has increased slightly (by 0.6 percent) because of the now-active bottleneck on Segments 10 and 11 in Time Interval 3. However, the overall system delay, including on-ramp delays, dropped by 15 percent. There were minor changes in overall facility speeds, densities, and travel times.

ESTIMATED SEGMENT SPEEDS (km/h)

Time Interval	Segment Number and Type								
	1 B	2 ONR	3 B	4 OFR	5 B	678 W	9 B	10 ONR	11 B
1	108.9	93.2	103.1	96.2	108.4	84.0	99.8	87.3	86.9
2	109.0	91.7	100.5	96.3	106.8	91.2	99.0	86.8	85.8
3	109.2	91.8	101.1	96.2	107.5	80.9	86.2	85.9	83.9
4	110.0	96.3	109.1	97.0	109.5	82.0	107.4	92.7	100.6
5	110.0	97.5	109.9	96.6	109.5	90.6	108.8	96.0	108.9

ESTIMATED SEGMENT DENSITIES (veh/km/ln) AND LOS

Time Interval	Segment Number and Type								
	1 B	2 ONR	3 B	4 OFR	5 B	678 W	9 B	10 ONR	11 B
1	15.6 C	20.3 D	19.0 D	20.3 D	16.0 C	20.0 D	20.5 D	23.6 E	26.2 E
2	15.5 C	21.0 D	20.2 D	21.0 D	17.1 D	18.4 D	20.9 D	23.8 E	26.7 E
3	15.2 C	20.8 D	19.9 D	20.8 D	16.7 D	22.2 E	24.4 E	24.0 E	27.6 E
4	13.4 C	17.4 D	15.3 C	17.2 D	13.5 C	18.6 D	17.0 D	21.1 D	20.2 D
5	12.0 C	15.6 C	13.5 C	15.4 C	11.6 C	13.9 C	14.0 C	17.7 D	15.6 C

SUMMARY OF FACILITYWIDE PERFORMANCE MEASURES

Time Interval	Performance Measures						
	Vehicle-km of Travel (Demand)	Vehicle-km of Travel (Flow Rate)	Vehicle-h of Travel	Vehicle-h of Delay	Average Speed (km/h)	Average Density (veh/km/ln)	Facility Travel Time (min)
1	8334	8334	87.6	11.8	95.2	20.1	3.44
2	8511	8511	89.2	11.9	95.4	20.4	3.44
3	8586	8579	92.4	14.9	92.4	21.3	3.54
4	7258	7266	73.1	7.1	99.4	17.0	3.29
5	6255	6255	60.7	3.9	103.0	14.2	3.19
Overall	38 944	38 945	403.0	49.6	96.5	-	3.38

EXAMPLE PROBLEM 4

The Facility In this example, the facility analyzed in Example Problem 1 is evaluated with reduction of capacity on Segment 9 in the first four time intervals due to an accident on the shoulder (nonrecurring congestion).

The Question What is the capacity and level-of-service profile for a directional freeway facility with nonrecurring congestion on Segment 9?

The Facts
√ The facts shown in Example Problem 1 apply.

Outline of Solution The segment capacity adjustment factor is used. This factor reduces the subject segment capacity for a limited number of time intervals. A revised speed-flow curve is also used in this case. Because of space limitations, the results that follow are confined to the effect of the incident on segment and facility performance. The results are compared with those obtained in Example Problem 1.

Steps

1. Adjustment of HCM capacities: in this example, the previously computed capacity for Segment 9 (6,946 veh/h) is multiplied by the capacity adjustment factor for shoulder accidents. This value is taken from Exhibit 22-6 and is estimated at 0.83. It yields a revised segment capacity of 5,765 veh/h. The revised capacity is applied in Time Intervals 1 through 4 only.

2. Adjusted d/c matrix: The matrix is shown in the exhibit below. As suspected, a single bottleneck on Segment 9 appears and is active during the first three time intervals. As stated in Example Problem 2, the incident causes oversaturation in the first time interval, and therefore it may be desirable for the user to begin the analysis one time interval earlier. However, because the level of oversaturation is very light (Segment 9 d/c is 1.005), the methodology will still produce correct estimates of performance in this case. Note that demand drops significantly in Time Interval 4 and demand flows can still pass through the reduced segment capacity.

ESTIMATED CAPACITY (veh/h) AND d/c RATIO MATRIX

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	0.69	0.80	0.80	0.80	0.70	0.91	0.91	0.91	<u>1.005</u>	0.93	0.93
2	0.69	0.83	0.83	0.83	0.74	0.91	0.91	0.91	<u>1.01</u>	0.93	0.93
3	0.68	0.82	0.82	0.82	0.73	0.98	0.98	0.98	<u>1.04</u>	0.95	0.95
4	0.60	0.68	0.68	0.68	0.60	0.83	0.83	0.83	0.89	0.82	0.82
5	0.54	0.61	0.61	0.61	0.52	0.69	0.69	0.69	0.62	0.69	0.69
Capacity	6946	6946	6946	6946	6946	6946	6946	6946	5765[a] 6946	6946	6946

Note:
a. Applies to Time Intervals 1 through 4 only.

3. Oversaturated segment service measures and LOS: The results of the oversaturated analysis are shown in the exhibit below for queue length position at the end of each interval.

ESTIMATED QUEUE LENGTH (m) AT END OF EACH TIME INTERVAL

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1								181			
2						55 100 (R)	200	250			
3						204 1430 (R)	200	250			
4						430 (R)					
5											

The exhibits below illustrate segment speeds, densities, and levels of service.

ESTIMATED SEGMENT SPEEDS (km/h)

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	109.6	94.5	106.2	96.5	109.4	86.9	75.2	75.2	69.7	91.3	95.4
2	109.7	93.4	104.5	96.6	108.6	88.9	65.5	56.9	69.7	91.4	95.6
3	109.8	93.4	104.9	96.5	107.8	79.2	74.6	65.8	70.4	91.8	96.8
4	110.0	96.9	109.8	97.1	109.5	102.4	104.7	101.5	78.3	93.5	101.7
5	110.0	98.0	109.9	96.7	109.5	95.5	93.5	93.5	106.6	96.7	109.4

ESTIMATED SEGMENT DENSITIES (veh/km/ln) AND LOS

Time Interval	Segment Number and Type										
	1 B	2 ONR	3 B	4 OFR	5 B	6 ONR	7 R	8 OFR	9 B	10 ONR	11 B
1	14.6 C	19.2 D	17.4 D	19.1 D	14.9 C	21.6 D	28.0 F	28.0 F	27.6 E	22.3 E	22.4 E
2	14.5 C	19.8 D	18.3 D	19.8 D	15.8 C	23.6 F	31.8 F	36.6 F	27.6 E	22.3 E	22.3 E
3	14.3 C	19.6 D	18.1 D	19.6 D	15.7 C	27.2 F	29.0 F	32.9 F	27.2 E	22.1 E	21.8 D
4	12.6 C	16.5 C	14.3 C	16.3 C	12.8 C	19.6 D	19.2 D	19.9 D	23.1 E	20.9 E	19.7 D
5	11.3 C	14.8 C	12.8 C	14.6 C	10.9 B	16.6 C	17.2 D	17.2 D	13.7 C	17.0 D	14.8 C

The above results indicate that the incident causes queues to develop on both the freeway mainline and the on-ramp at Segment 6 in Time Intervals 1 through 3. Since traffic demand drops sharply in Interval 4, the mainline queues dissipate by the end of that interval. A residual queue 430 m long remains on the on-ramp roadway proper at the end of Time Interval 4. Poor level of service is observed in Segments 6 through 8 upstream of the bottleneck during the first three time intervals. However, all queues are cleared and undersaturated operations are restored during Time Interval 5.

4. Directional facility MOE estimation: The individual segment performance measures are aggregated for each time interval. Both VkmT measures based on demands [VkmT(Demand)] and the actual flow rates [VkmT(Flow Rate)] are computed. The following exhibit summarizes the facilitywide results by time interval.

SUMMARY OF FACILITYWIDE PERFORMANCE MEASURES

Time Interval	Performance Measures						
	Vehicle-km of Travel (Demand)	Vehicle-km of Travel (Flow Rate)	Vehicle-h of Travel	Vehicle-h of Delay	Average Speed (km/h)	Average Density (veh/km/ln)	Facility Travel Time (min)
1	7862	7851	83.3	11.9	94.3	19.9	3.47
2	8030	7989	87.1	15.4	91.7	21.0	3.57
3	8100	7960	85.8	24.5	92.8	20.7	3.53
4	6847	7005	69.3	18.1	101.0	16.8	3.25
5	5901	5937	57.3	3.45	103.7	13.9	3.17
Overall	36 740	36 742	382.8	73.35	96.0	-	3.40

Results It is instructive to compare the results of this example problem with those obtained in Example Problem 1. While serving the same VkmT, the total vehicle hours of travel on the mainline due to the incident increases by 3.8 percent. The total vehicle delay on the system, which includes estimated delay on the on-ramps, increases by 111 percent. In the fourth time interval, the differences are even greater, with delays increasing by 320 percent. On average, the system density under incident conditions appears to have increased by 10 percent, and the average speed has dropped by about 3.6 percent.

EXAMPLE PROBLEM 5

The Facility In this example problem, the facility analyzed in Example Problem 4 is evaluated with incident management to mitigate the effect of the incident.

The Question For normal conditions, a 60-min incident, and a 30-min incident, compare quality-of-service and performance measures.

The Facts

√ The facts of Example Problem 1 apply, except that the incident effect on the capacity of Segment 9 is limited to the first two time intervals.

Steps

1. A summary of the results is given in the exhibit below.

EFFECT OF REDUCED INCIDENT DURATION ON SELECTED FACILITY PERFORMANCE MEASURES

Facility Performance Measure	Normal Conditions Example 1	60-min Incident Example 4	30-min Incident Example 5
Vehicle-km mainline travel	36 741	36 741	36 741
Vehicle-h mainline travel	368.7	382.7	378.2
Vehicle-h mainline delay (h)	34.7	48.7	44.2
Vehicle-h on-ramp delay (h)	0.0	24.5	8.7
Vehicle-h total delay (h)	34.7	73.2	52.9
Overall maximum d/c ratio (segment)	0.98 (6, 7, 8)	1.04 (9)	1.01 (9)
Average mainline facility speed (km/h)	99.6	96.0	97.1
Average mainline travel time (min)	3.30	3.40	3.40
Maximum mainline queue[a] (m)	0	654	505
Time interval with max. mainline queue	N/A	3	2
Maximum ramp queue (m)	0	1430	842
Time interval with max. ramp queue	N/A	3	3

Note:
a. Measured from the downstream end of Segment 8, just upstream of Segment 9.
N/A - not applicable.

Results As expected, the reduced incident duration improves both mainline and ramp traffic performance. Overall, mainline delays drop by a modest 9.2 percent, while ramp delays are reduced by 65 percent. Similarly, the maximum queue length on the mainline drops by 23 percent compared with a 41 percent drop in ramp queues. The maximum on-ramp demand occurs at the start of Interval 3, by which time the incident has been cleared under the 30-min incident scenario.

EXAMPLE PROBLEM 6

The Facility In this example problem the facility analyzed in Example Problem 5 is evaluated with ramp metering for the on-ramp flow on Segment 6.

The Question What are the performance measures of normal, 60-min incident, and ramp-metering conditions?

The Facts
√ The facts shown in Example Problem 4 apply.
√ Metering rates of 900 veh/h and 1,200 veh/h are selected.

Outline of Solution This strategy is intended to minimize the queues on the freeway at the expense of ramp queues and delays. The effect on the adjacent surface operation is not considered in this analysis, and therefore the results should be viewed with caution. The metering rate selected was 900 veh/h, which is the maximum rate recommended for single-lane on-ramps (15). Ramp metering is applied only to the first three time intervals, since the on-ramp demand drops significantly in Time Interval 4. A second metering strategy is also evaluated. This strategy uses a two-lane ramp-metering rate of 1,200 veh/h, in which vehicle departures alternate at the higher rate. As in Example Problem 5, only facilitywide measures are reported.

Steps

1. A summary of the facility performance measures is given in the exhibit below. As expected, the single-lane ramp metering causes severe congestion and queuing on the on-ramp at Segment 6 while eliminating the mainline queue. In fact, the on-ramp queues on Segment 6 are not cleared by the end of the analysis period, resulting in fewer vehicle kilometers of travel production. However, it is virtually impossible that the observed maximum ramp queue of 9788 m could ever materialize in the field without spilling back onto the surface street system or causing ramp drivers at Segment 6 to divert elsewhere.

EFFECT OF RAMP-METERING STRATEGIES ON SELECTED FACILITY PERFORMANCE MEASURES

Facility Performance Measure	Normal Conditions Example 1	60-min Incident Example 4 No Metering	Metering Rate 900 veh/h Single Lane	Metering Rate 1200 veh/h Two Lanes
Vehicle-km mainline travel	36 741	36 741	36 658	36 741
Vehicle-h mainline travel	368.7	382.7	370.9	380.1
Vehicle-h mainline delay (h)	34.7	48.7	37.7	46.0
Vehicle-h on-ramp delay (h)	0.0	24.5	260.5	94.9
Vehicle-h total delay (h)	34.7	73.2	298.2	140.9
Overall maximum d/c ratio (segment)	0.98 (6, 7, 8)	1.04 (9)	1.04 (9)	1.04 (9)
Average mainline facility speed (km/h)	99.6	96.0	98.8	96.7
Average mainline travel time (min)	3.30	3.40	3.30	3.40
Maximum mainline queue[a] (m)	0	654	0	338
Time interval with max. mainline queue	N/A	3	N/A	2
Maximum ramp queue (m)	0	1430	9788	3011
Time interval with max. ramp queue	N/A	3	4	2

Note:
a. Measured from the downstream end of Segment 8.
N/A - not applicable.

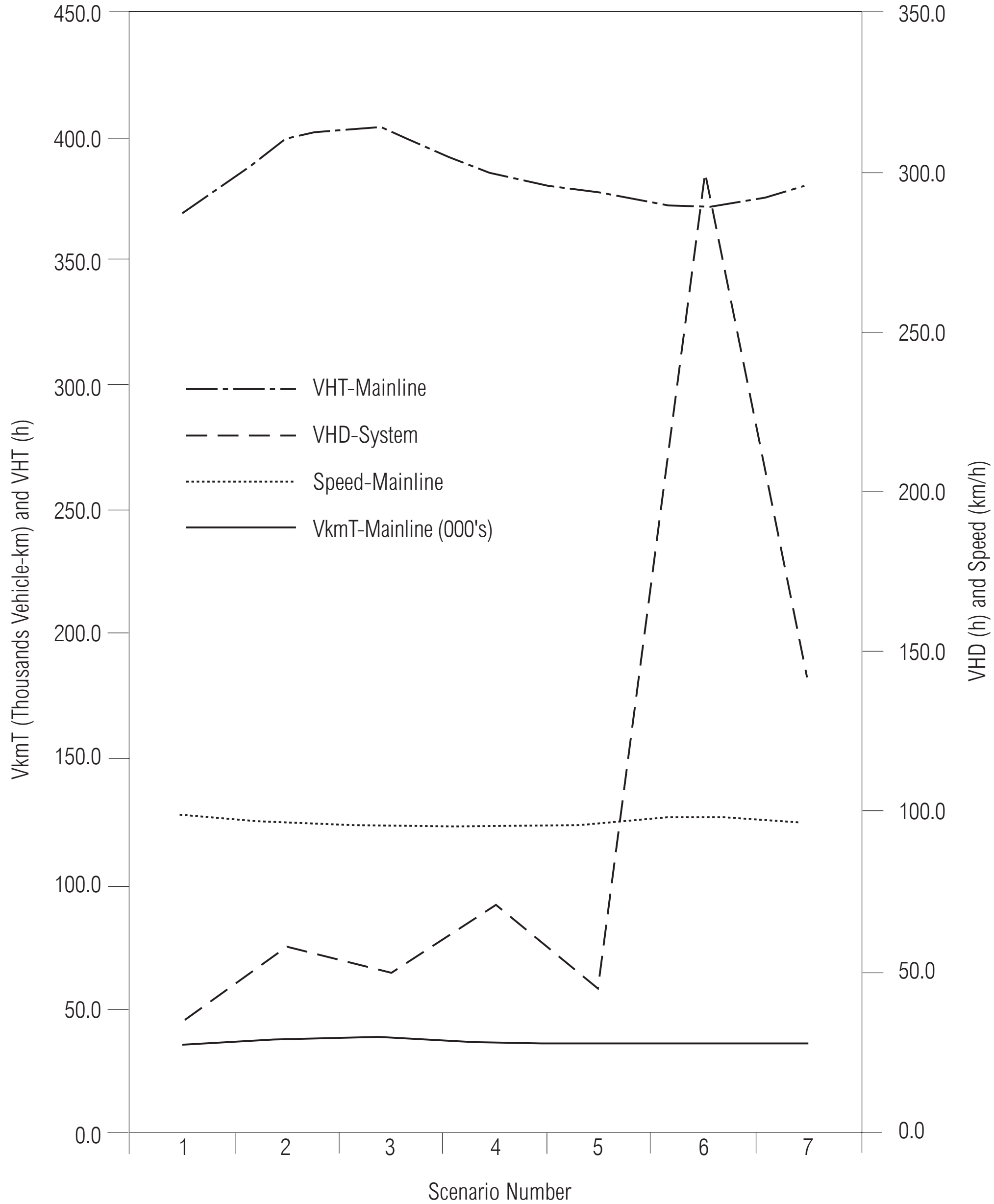
With two-lane metering, the on-ramp queue is reduced to 3011 m, which is still quite high. Furthermore, there is now queuing on the mainline. Thus, it appears that the two ramp-metering strategies presented in this example are not as effective in improving system performance as the reduction in incident response time performed in Example Problem 5.

The user may test additional strategies that can be handled by the methodology. They include, for example, a combination of reduced incident response and ramp metering and systemwide ramp metering in which metering rates on Segments 3 and 6 are jointly determined. Diversion strategies in which the excess demand on Segment 6 is rerouted to the surface street system and back onto the freeway downstream of the incident location can also be evaluated.

Results An overall comparison of the freeway facility performance measures is shown in the exhibit below. Scenarios 1 through 5 represent the conditions described in Examples Problems 1 through 5, respectively. Scenarios 6 and 7 represent the effects of the two ramp-metering strategies described in Example Problem 6.

The purpose of this chart is to demonstrate the sensitivity of various facility performance measures to key geometric and traffic management improvement strategies. The results suggest that mainline speed, total vehicle kilometers of travel, and total vehicle hours of travel are not very sensitive to the various strategies. On the other hand, total system delay (VHD) appears to vary considerably across scenarios. VHD is defined as the difference between the actual mainline travel time and travel time at the free-flow speed + the sum of all ramp delays. Since this is the only performance measure that incorporates ramp delays in its calculations, it should be considered a key measure in evaluating the operational performance of freeway facilities.

SUMMARY COMPARISON OF FACILITY PERFORMANCE MEASURES



Notes:
Scenario 1: base scenario - Example 1
Scenario 2: base + 6% demand growth
Scenario 3: Scenario 2 + auxiliary lane on Section 4
Scenario 4: base + 60-min incident on Section 5
Scenario 5: base + 30-min incident on Section 5
Scenario 6: base + 60-min incident + 900 veh/h single-lane meter - second on-ramp
Scenario 7: base + 60-min incident + 1,200 veh/h two-lane meter - second on-ramp

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APPENDIX A. DETAILED COMPUTATIONAL MODULES FOR FREEWAY FACILITIES

A.1 SCOPE OF APPENDIX MATERIAL

The freeway facility analytical methodology is described in the main body of this chapter. The computations contained within the methodology are detailed in this appendix. In Section II of the main body, the characteristics of freeway flow that are computed in the methodology are discussed. The computational steps are given in Section III. In Section A.1 of this appendix, the limitations of the methodology are outlined, and a glossary of all relevant variables is presented. The overall procedure presented in Section III of the main body is described in more detail in Section A.2. The computations for the undersaturated portion of the methodology are detailed in Section A.3. The oversaturated computations of the methodology are detailed in Section A.4. Section A.5 contains the directional facility computations.

A.1.1 Limitations

The procedure described herein becomes extremely complex when the queue from a downstream bottleneck extends into an upstream bottleneck causing a queue collision. When such cases arise, the reliability of the methodology is questionable, and the user is cautioned about the validity of the results. However, noninteracting bottlenecks are accommodated by the methodology.

The completeness of the analysis will be limited if freeway segment cells in the first time interval, the last time interval, and the first freeway segment do not all have demand-to-capacity ratios less than 1.00. The methodology can handle congestion in the first interval properly, although it will not quantify any congestion that could have occurred before the analysis. To ensure complete quantification of the effects of congestion, it is recommended that the analysis contain an initial undersaturated time interval. If all freeway segments in the last time interval do not exhibit demand-to-capacity ratios less than 1.00, congestion continues beyond the last time interval, and additional time intervals should be added. This fact will be noted as a difference between the vehicle kilometers of travel demand desired at the end of the analysis and the corresponding vehicle kilometers of travel flow generated. If queues extend upstream of the first segment, the analysis will not account for the congestion outside the freeway facility but will store the vehicles vertically until the congestion clears the first segment. The same process is followed for queues on on-ramp roadways.

The analyst could, given enough time, analyze a completely undersaturated time-space domain manually, although this is highly unlikely. It is not expected that an analyst will ever manually analyze a time-space domain that includes oversaturation. For heavily congested directional freeway facilities with interacting bottleneck queues, a simulation model might be more applicable.

A.1.2 Glossary

In this glossary internal variables used exclusively in the freeway facilities methodology are defined. The glossary of variables covers six parts: global variables, segment variables, node variables, on-ramp variables, off-ramp variables, and facilitywide variables. Segment variables represent conditions on segments. Node variables denote flows across a node connecting two segments. Facilitywide variables pertain to aggregate traffic performance over the entire facility. On-ramp and off-ramp variables are variables that correspond to flow on ramps. In addition to these spatial categories, there are temporal divisions that represent characteristics over either a time step or a time interval. The first dimension associated with each variable specifies whether the variable refers to segment or node characteristics. The labeling scheme for nodes and segments is such that Segment (i) is immediately downstream of Node (i).

Thus, there is always one more node than the number of segments on a facility. The second and third dimensions denote a time step (t) and a time interval (p). Facility variables are estimates of the average performance over the entire length of the facility. The units of flow are in vehicles per time step. The selection of the time step size is discussed later in this appendix.

Global Variables

- KC—Density at capacity: the ideal density at capacity (veh/km/ln). The density at capacity is 28 pc/km/ln, which must be converted to veh/km/ln using the heavy-vehicle factor (f_{HV}) described in Chapter 23.
- KJ—Jam density: the facilitywide jam density (veh/km/ln).
- NS—Number of segments: the number of segments on the facility.
- i—Index to segment or node number: i = 1, 2, …, NS (for segments) and i = 1, 2, ..., NS + 1 (for nodes).
- P—Number of time intervals: number of time intervals in the analysis period.
- p—Time interval number: p = 1, 2, …, P.
- S—Time steps per interval: number of time steps in a time interval (integer).
- t—Number of time steps in a single interval: t = 1, 2, …, S.
- T—Time steps per hour: number of time steps in 1 h (integer).

Segment Variables

- ED(i, p)—Expected demand: the demand that would arrive at Segment (i) on the basis of upstream conditions over Time Interval (p). The upstream queuing effects include the metering of traffic from an upstream queue, but not the spillback of vehicles from a downstream queue.
- K(i, p)—Average segment density: the average traffic density of Segment (i) over Time Interval (p), as estimated by the oversaturated procedure.
- KB(i, p)—Background density: Segment (i) density (veh/km/ln) over Time Interval (p) assuming there is no queuing on the segment. This density is calculated using the expected demand on the segment in the corresponding undersaturated procedure in Chapters 23 through 25.
- KQ(i, t, p)—Queue density: vehicle density in the queue on Segment (i) during Time Step (t) in Time Interval (p). The queue density is calculated on the basis of a linear density-flow relationship in the congested regime (see Exhibit A22-5).
- L(i)—Length: the length of Segment (i) (km).
- N(i, p)—Number of lanes: the number of lanes on Segment (i) in Time Interval (p). Could vary by time interval if a temporary lane closure is in effect.
- NV(i, t, p)—Number of vehicles: the number of vehicles present on Segment (i) at the end of Time Step (t) during Time Interval (p). The number of vehicles is initially based on the calculations of Chapters 23 through 25, but as queues grow and dissipate, input-output analysis updates these values in each time step.
- Q(i, t, p)—Queue length: total queue length on Segment (i) at the end of Time Step (t) in Time Interval (p) (m).
- SC(i, p)—Segment capacity: maximum number of vehicles that can pass through Segment (i) in Time Interval (p) based strictly on traffic and geometric properties. These capacities are calculated using Chapters 23 through 25.
- SD(i, p)—Segment demand: the desired flow rate through Segment (i) including on- and off-ramp demands in Time Interval (p) (veh). This segment demand is calculated without any capacity constraints.
- SF(i, t, p)—Segment flow: the segment flow out of Segment (i) during Time Step (t) in Time Interval (p) (veh).
- WS(i, p)—Wave speed: the speed at which a front-clearing queue shock wave travels through Segment (i) during Time Interval (p) (m/s).

- WTT(i, p)—Wave travel time: the time taken by the shock wave traveling at the wave speed (WS) to travel from the downstream end of Segment (i) to the upstream end of the segment during Time Interval (p), in time steps.
- U(i, p)—Average segment speed: the average space-mean speed over the length of Segment (i) during Time Interval (p) (km/h).
- UV(i, t, p)—Unserved vehicles: the additional number of vehicles stored on Segment (i) at the end of Time Step (t) in Time Interval (p) due to a downstream bottleneck.

Node Variables

- MI(i, t, p)—Maximum mainline input: the maximum flow desiring to enter Node (i) during Time Step (t) in Time Interval (p), based on flows from all upstream segments, taking into account all geometric and traffic constraints upstream of the node including queues accumulated from previous time intervals.
- MF(i, t, p)—Mainline flow: the actual mainline flow rate that can cross Node (i) during Time Step (t) in Time Interval (p).
- MO1(i, t, p)—Maximum Mainline Output 1: the maximum allowable mainline flow rate across Node (i) during Time Step (t) in Time Interval (p), limited by the flow from an on-ramp at Node (i).
- MO2(i, t, p)—Maximum Mainline Output 2: the maximum allowable mainline flow rate across Node (i) during Time Step (t) in Time Interval (p), limited by the available storage on Segment (i) due to a downstream queue.
- MO3(i, t, p)—Maximum Mainline Output 3: the maximum allowable mainline flow rate across Node (i) during Time Step (t) in Time Interval (p), limited by the presence of queued vehicles at the upstream end of Segment (i) while the queue clears from the downstream end of Segment (i).

On-Ramp Variables

- ONRI(i, t, p)—On-ramp input: ramp flow rate desiring to enter the merge point at On-Ramp (i) during Time Step (t) in Time Interval (p), based on current ramp demand and ramp queues accumulated from previous time intervals.
- ONRD(i, p)—On-ramp demand: desired entry flow rate for on-ramp at Node (i) in Time Interval (p).
- ONRC(i, p)—On-ramp capacity: geometric carrying capacity of on-ramp at Node (i) roadway during Time Interval (p).
- ONRF(i, t, p)—On-ramp flow: actual ramp flow rate that can cross On-Ramp Node (i) during Time Step (t) in Time Interval (p); takes into account control constraints (e.g., ramp meters).
- ONRQL(i, t, p)—On-ramp queue length: queue length on On-Ramp (i) at the end of Time Step (t) in Time Interval (p).
- ONRO(i, t, p)—On-ramp output: maximum flow rate that can enter the merge point from On-Ramp (i) during Time Step (t) in Time Interval (p); constrained by Lane 1 (shoulder lane) flow on Segment (i) and the Segment (i) capacity or by a queue spillback filling the mainline segment from a bottleneck further downstream, whichever governs.
- ONRQ(i, t, p)—On-ramp queue: the unmet demand that is stored on the on-ramp roadway at Node (i) during Time Step (t) in Time Interval (p) (veh).
- RM(i, p)—Ramp-metering rate: the maximum allowable rate of an on-ramp meter at on-ramp at Node (i) during Time Interval (p) (veh/h).

Off-Ramp Variables

- DEF(i, t, p)—Deficit: the unmet demand from a previous Time Interval (p) that flows past Node (i) during Time Step (t); used in off-ramp flow calculations downstream of a bottleneck.

- OFRD(i, p)—Off-ramp demand: the desired flow exiting at Off-Ramp (i) during Time Interval (p).
- OFRF(i, t, p)—Off-ramp flow: the actual flow that can exit at Off-Ramp (i) during Time Step (t) in Time Interval (p).

Facilitywide Variables

- SMS(NS, p)—Average time interval facility speed: the average space-mean speed over the entire facility during Time Interval (p).
- K(NS, p)—Average time interval facility density: the average vehicle density over the entire facility during Time Interval (p).
- SMS(NS, P)—Average analysis period facility speed: the average space-mean speed over the entire facility during the entire analysis period (P).
- K(NS, P)—Average analysis period facility density: the average vehicle density over the entire facility during the entire analysis period (P).

A.2 OVERALL PROCEDURE DESCRIPTION

The procedure is described according to the nine-step process shown in Exhibit A22-1.

A.2.1 Input Module

The first step in the methodology is to gather all geometric and traffic data. The most basic data are required for sizing the analysis. These basic data are listed below.

- Number of time intervals: the number of time intervals is input to size the analysis with the correct time dimension. There is no practical limit on the number time intervals, although the current computer implementation is limited to 12 intervals.
- Time interval duration: the time interval duration can vary to allow for finer or broader analysis of freeway facilities. Caution should be used when using other than the recommended 15-min time interval. First, the capacities that are calculated are based on the maximum hourly flow rate that can travel through a segment during a 15-min analysis interval. As the interval duration decreases, the capacity may actually increase, and vice versa. The methodology assumes that there is instantaneous travel time between segments when demands are computed on segments. In other words, there is no demand shock wave at any point where the demand changes (i.e., when a new time interval begins). For this assumption to be reasonable, the uncongested travel time of the freeway facility being analyzed (which is directly related to its length) should not be longer than the duration of the time intervals being used.
- Time step duration: once oversaturation begins, the procedure moves to time steps. The duration of the time steps should be determined on the basis of the segment lengths as shown later in Exhibit A22-4. There must be an integer number of time steps in a time interval.
- Number of segments: the number of segments must be determined from the freeway facility chapter. Refer to Exhibit 22-3 for a suggested process to divide a facility into sections and segments.
- Jam density: the systemwide jam density is required for oversaturated analysis. The default value is 120 pc/km/ln.

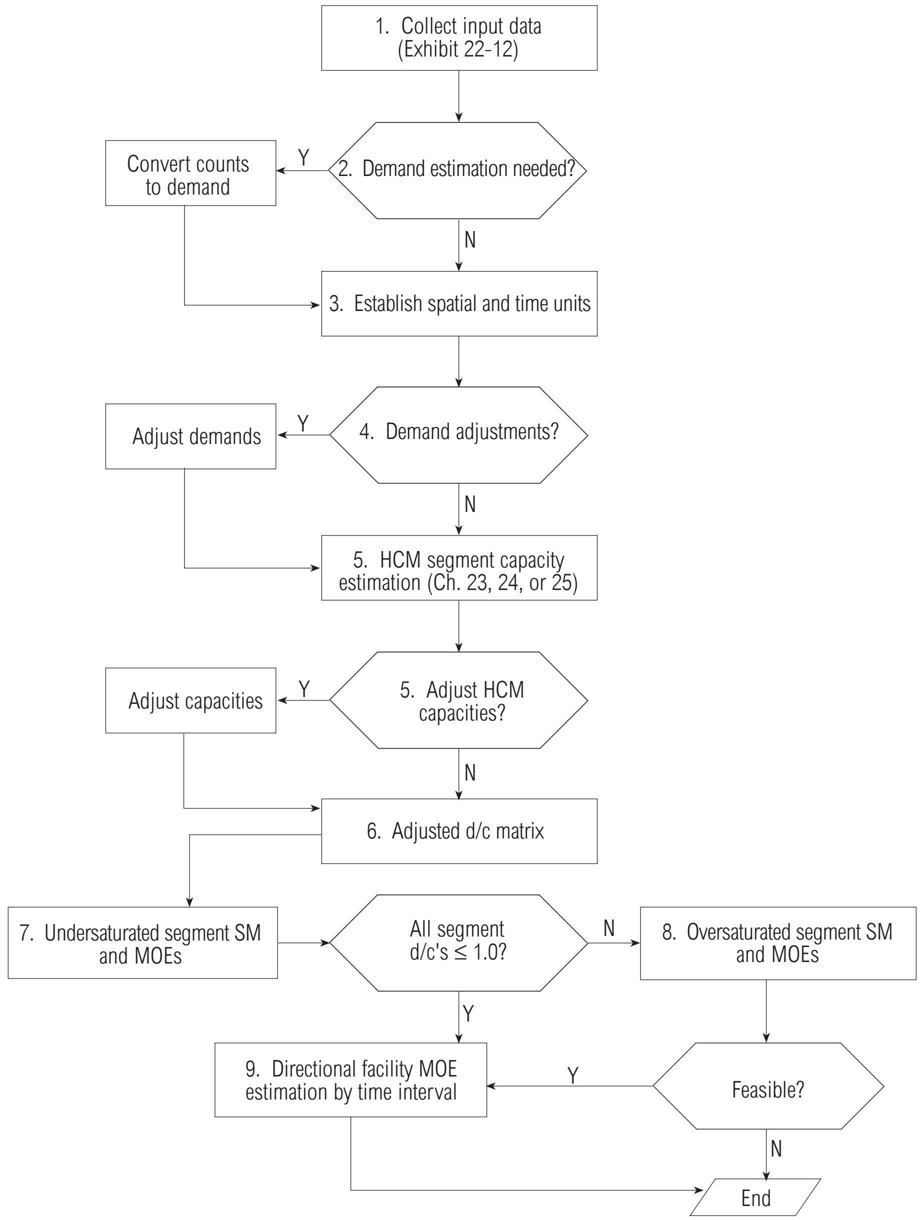
The geometric, traffic, and demand data required for a freeway facility analysis are shown in Exhibit 22-12.

A.2.2 Demand Estimation Module

The demand estimation module is invoked when the methodology uses actual freeway counts. If demand flows are known or can be projected, those values can be used directly. The demand estimation module is designed to convert the input set of freeway exit 15-min traffic counts into a set of freeway exit 15-min traffic demands. Freeway exit demand is defined as the number of vehicles that desire to exit the freeway in a given 15-

min time interval. This demand may not be represented by the 15-min exit count because of upstream freeway congestion within the freeway facility.

EXHIBIT A22-1. OVERALL PROCEDURE LAYOUT



Notes:

d/c = demand-to-capacity ratio.
SM = service measure.
MOE = measure of effectiveness.

The procedure followed is to sum the freeway entrance demands along the entire freeway facility (including the freeway mainline entrance) and to compare it with the sum of the freeway exit counts along the entire freeway facility (including the freeway mainline exit) for each time interval. The ratio of the freeway entrance demands to the freeway exit counts is calculated for each time interval and will be referred to as the time interval scale factor. Theoretically, the scale factor should approach 1.00 when the freeway exit counts are, in fact, freeway exit demands.

Scale factors greater than 1.00 indicate increasing levels of congestion within the freeway facility (and the storing of vehicles on the freeway). Here, the exit traffic counts underestimate the actual freeway exit demands. Scale factors less than 1.00 indicate decreasing levels of congestion within the freeway facility (and the release of stored vehicles on the freeway). Here, the exit traffic counts overestimate actual freeway exit demands. To provide an estimate of freeway exit demand, each freeway exit count in the time interval is multiplied by the time interval scale factor.

The accuracy of this procedure primarily depends on the quality of the set of freeway traffic counts and to a lesser extent on the length of the freeway facility. With the use of 15-min time intervals, freeway facility lengths up to 15 to 20 km should not introduce significant errors into the procedure. The calculated scale factor pattern over the study period duration offers a means of checking the quality of the traffic count data. For example, if there is no congestion over the entire time-space domain, then there should be no pattern in the calculated 15-min scale factors, and they all should be within the range of 0.95 to 1.05. If there is congestion within the time-space domain, then there should be a pattern in the calculated 15-min scale factors. During the early time intervals with no congestion, the scale factors are expected to approach 1.00 and be within the range of 0.95 to 1.05. As congestion begins to occur and increase over time, the scale factors are expected to increase over 1.00 and be within the range of 1.00 to 1.10. When the extent of congestion reaches its highest level, the scale factor is expected to approach 1.00 and be within the range of 0.95 to 1.05. As the level of congestion recedes, the scale factor is expected to be less than 1.00 and be within the range of 0.90 to 1.00. If the final time intervals exhibit no congestion over the complete time-space domain, then there should be no pattern in the calculated 15-min scale factors, and they all should be within the range of 0.95 to 1.05. Once the freeway entrance and exit demands are estimated using the scale factors, the traffic demands for each freeway section in each time interval can be determined.

A.2.3 Establish Spatial and Time Units

The procedure analyzes a freeway in spatial units called segments, which are defined in Chapters 23 through 25. The division of a freeway facility into segments is described in Section II of this chapter. Time units are described in Section A.2.1.

A.2.4 Demand Adjustment Module

Driver responses such as spatial, temporal, or modal shifts caused by traffic management strategies are not automatically incorporated in the methodology. On viewing the facility traffic performance results, the analyst can modify the demand input manually to simulate the effect of user demand responses or traffic growth effects. The accuracy of the results depends on the accuracy of the estimation of the user demand responses. Ramp-metering strategies are evaluated through adjusting the ramp roadway capacity, and this application is described in the segment capacity adjustment module.

A.2.5 Segment Capacity Estimation and Adjustment Module

Segment capacity estimates are determined from Chapters 23 through 25 for basic segments, weaving segments, and ramp segments, respectively. All capacities are expressed in vehicles per hour. All estimates of segment capacity should be carefully reviewed and compared with local knowledge and available traffic information for the study site. The capacity value used for bottleneck segments has the greatest effect on the predicted freeway traffic performance. Actual field-observed capacities at bottlenecks should be obtained whenever practical and substituted for estimated capacities.

On-ramp and off-ramp roadway capacities are also determined in this capacity module. On-ramp demands may exceed on-ramp capacities and limit the traffic demand entering the freeway. Off-ramp demands may exceed off-ramp capacities and cause congestion on the freeway, although this particular effect is not accounted for in the methodology. The relationships of demand and capacity for each on-ramp and off-ramp, as well as for each freeway segment, will be addressed later in the demand-to-capacity analysis module.

Again, unlike the analyses in the basic freeway, freeway weaving, and ramp chapters, all analyses in this chapter are on a vehicle-based capacity and not in passenger-car units.

The effect of a predetermined ramp-metering plan can be evaluated in this methodology by modifying the ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to the desired metering rate specified. This feature not only permits the evaluation of the prespecified ramp-metering plan but also permits the user to experiment to obtain an improved ramp-metering plan.

Freeway design improvements can be evaluated within this methodology by modifying the design features of any segment or segments of the freeway facility, as described in Section II of this chapter.

Reduced-capacity situations can also be investigated. The capacity in any cell of the time-space domain can be reduced to represent incident situations such as construction and maintenance activities, adverse weather, and traffic accidents/vehicular breakdowns. Similarly, capacity can be increased to match field measurements. When analyzing adjusted capacity situations, it is important to use an alternative speed-flow relationship. The following relationship assures a constant ideal density of 28 pc/km/ln at capacity as indicated in Chapter 23 of the manual. Exhibit A22-2 shows speed-flow plots for capacity adjustment factors (CAFs) of 100, 95, 90, and 85 percent of the original capacity. The predicted speed for the alternative speed-flow model can be computed by using Equation A22-1.

$$S = FFS + \left[1 - e^{\ln\left(FFS + 1 - \frac{C * CAF}{28}\right)\frac{v_p}{C * CAF}}\right] \tag{A22-1}$$

where

S	=	segment speed (km/h),
FFS	=	segment free-flow speed (km/h),
C	=	original segment capacity (pc/h/ln),
CAF	=	capacity adjustment factor (CAF = 1.0, use Chapters 23 through 25 speed estimation procedures), and
v_p	=	segment flow rate (pc/h/ln).

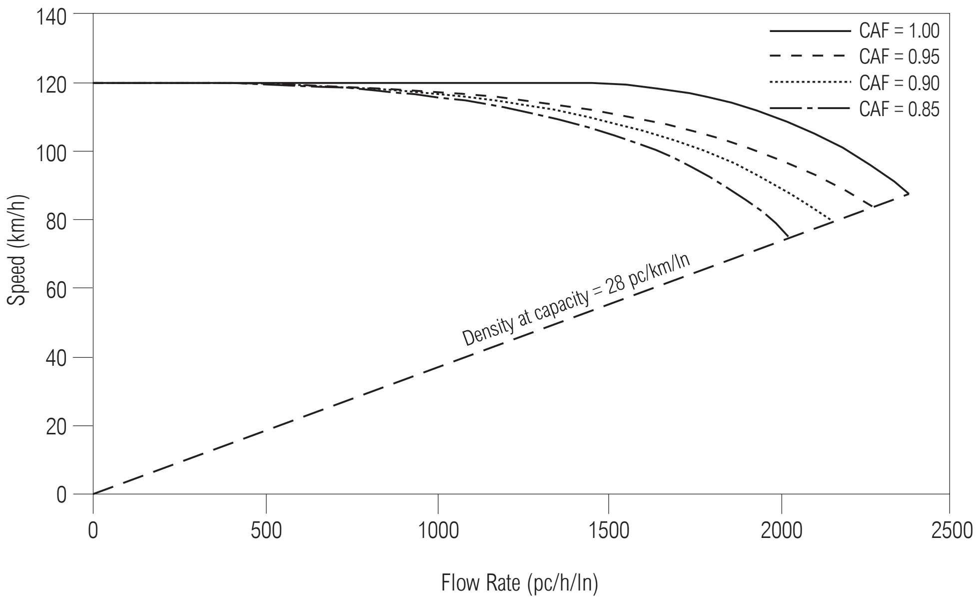
Note that when $v_p \approx 0$ in Equation A22-1, S approaches FFS. Similarly, when $v_p \approx C * CAF$, S approaches speed at capacity.

A.2.6 Demand-to-Capacity Ratio Module

Once all freeway segment cells have been analyzed, demand-to-capacity ratios are modified into volume-to-capacity ratios for later use in calculating freeway traffic performance measures. As stated earlier, if all freeway segment cells are undersaturated (demands less than capacities), the volume-to-capacity ratios are identical to the demand-to-capacity ratios, and the analysis is simple. However, if demand is greater than capacity in one or more of the freeway segment cells, oversaturated flow conditions will occur, and the time step analysis procedure is invoked.

Until oversaturated conditions are encountered, segments are analyzed using the undersaturated segment MOE module. All subsequent time intervals, however, are analyzed using the oversaturated segment MOE module.

EXHIBIT A22-2. ALTERNATIVE SPEED-FLOW CURVES FOR INDICATED CAPACITY ADJUSTMENT FACTORS (SEE FOOTNOTE FOR ASSUMED VALUES)



Notes:
Assumptions: FFS = 120 km/h, capacity adjustment factor (CAF) of 1.0, 0.95, 0.90, and 0.85.

A.3 UNDERSATURATED SEGMENT MOE MODULE

This module begins with the first segment in the first time interval. For each cell the flow (or volume) is equal to demand, the volume-to-capacity ratio is equal to the demand-to-capacity ratio, and undersaturated flow conditions prevail. Performance measures for the first segment during the first time interval are calculated using the procedures for the corresponding segment type in Chapters 23 through 25.

The analysis continues to the next downstream freeway segment in the same time interval, and the performance measures are calculated. The process is continued until the last downstream freeway segment cell in this time interval has been analyzed. For each cell, the volume-to-capacity ratio and performance measures are calculated for each freeway segment in the first time interval. The analysis continues in the second time interval beginning at the furthest upstream freeway segment and moving downstream until all freeway segments in that time interval have been analyzed. This pattern continues for the third time interval, fourth time interval, and so on until the methodology encounters a time interval that contains one or more segments with a demand-to-capacity ratio greater than 1.00 or when the last segment in the last time interval is analyzed. If none is encountered, the segment performance measures are taken directly from Chapters 23 through 25, and the facility performance measures are calculated as in Section A.4.

When the analysis moves from isolated segments to a facility, an additional constraint is necessary. To limit the speeds downstream of a segment experiencing a low average speed, a maximum achievable speed is imposed on each segment average speed. This maximum speed is based on acceleration characteristics reported by the American Association of State Highway and Transportation Officials and is shown in Equation A22-2 (*1*).

$$V_{max} = FFS - (FFS - V_{prev})e^{-0.0053L} \qquad \text{(A22-2)}$$

where

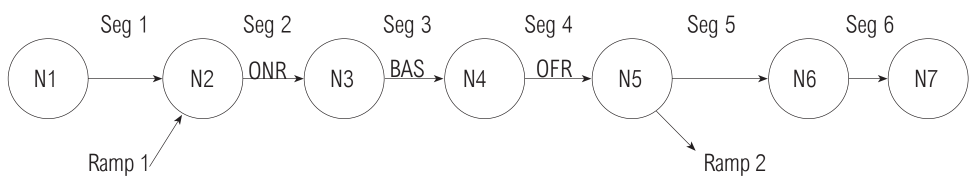
V_{max} = maximum achievable segment speed (km/h),
FFS = segment free-flow speed (km/h),
V_{prev} = average speed on immediate upstream segment (km/h), and

L = distance from midpoints of the upstream segment and the subject segment (m).

A.4 OVERSATURATED SEGMENT MOE MODULE

Oversaturated flow conditions occur when the demand on one or more freeway segment cells exceeds its capacity. Once oversaturation is encountered, the methodology changes its temporal and spatial units of analysis. The spatial units become nodes and segments, and the temporal unit moves from a time interval to smaller time steps. A node is defined as the junction of two segments. There is always one more node than segment, with nodes added at the beginning and end of each segment. The numbering of nodes and segments begins at the upstream end and moves to the downstream end, with the segment upstream of Node (i) numbered Segment (i – 1) and the downstream segment numbered (i), as shown in Exhibit A22-3. The intermediate segments and node numbers represent the division of the section between Ramps 1 and 2 into three segments numbered 2 (ONR), 3 (BASIC), and 4 (OFR). The oversaturated analysis moves from the first node to each downstream node in the same time step. After the completion of a time step, the same nodal analysis is performed for the subsequent time steps.

EXHIBIT A22-3. NODE-SEGMENT REPRESENTATION OF A DIRECTIONAL FREEWAY FACILITY



The oversaturated analysis focuses on the computation of segment average flows and densities in each time interval. These parameters are later aggregated to produce facilitywide estimates. Two key inputs into the flow estimation procedures are the time step duration for flow updates and a flow-density function. They are described in the next sections.

A.4.1 Procedure Parameters

Segment flows are calculated in each time step and are used to calculate the number of vehicles on each segment at the end of every time step. The number of vehicles on each segment is used to track queue accumulation and discharge and to calculate the average segment density.

To provide accurate estimates of flows in oversaturated conditions, the time intervals are divided into smaller time steps. The conversion from time intervals to time steps occurs during the first oversaturated time interval and remains until the end of the analysis. The transition to time steps is essential because at certain points in the methodology future performance estimates are made on the basis of the past value of a variable. The time steps correspond to the following lengths in Exhibit A22-4. These values are vital in two major situations.

EXHIBIT A22-4. RECOMMENDED TIME STEP DURATION FOR OVERSATURATED ANALYSIS

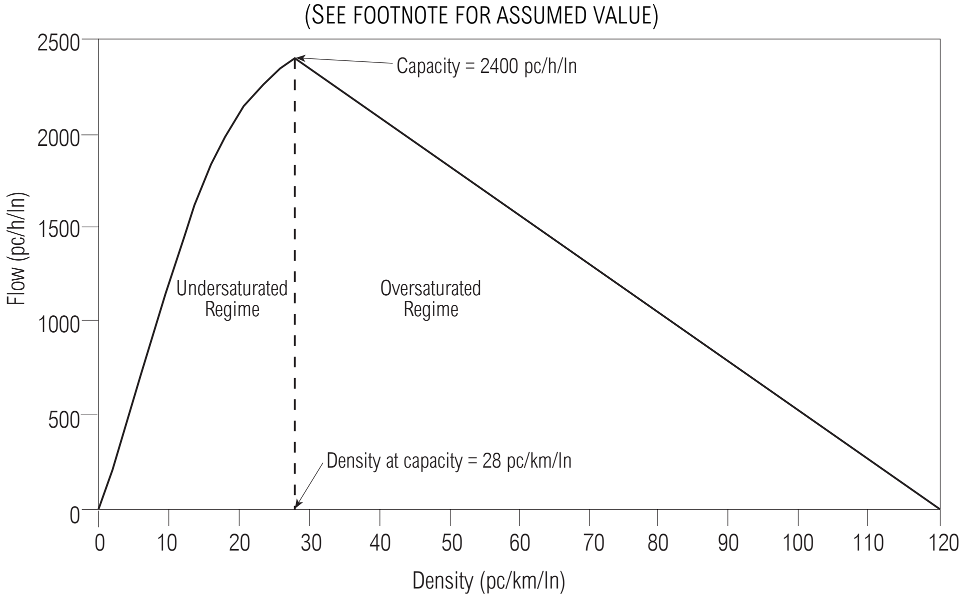
Shortest segment length (m)	≤ 100	200	300	400	≥ 450
Time step duration (s)	15	25	40	60	60

The first situation is when segments are short and the rate of queue growth is rapid. Under these conditions, a short segment may be completely undersaturated in one time step and completely queued in another. The methodology may store more vehicles in this segment during a time step than there is allowable space. Fortunately, this error is compensated for in the next time step, and the procedure continues to accurately track queues and store vehicles after this correction.

The second situation in which small time steps are important is when two queues interact. There is a temporary inaccuracy due to the maximum output of a segment changing, thus causing the estimation of available storage to be slightly in error. This results in the storage of too many vehicles on a particular segment. This supersaturation is temporary and is compensated for in the next time step. Inadequate time step size will result in erroneous estimation of queue lengths and may affect other performance measures as well. Regardless, if interacting queues occur, the results should be viewed with extreme caution.

Analysis of freeway segments depends on the relationships between segment speed, flow, and density. Chapter 23 of this manual defines a relationship between these variables and the calculation of performance measures in the undersaturated regime. The methodology presented here uses the same relationships for undersaturated segments. Calculations for oversaturated segments use a simplified linear flow-density diagram in the congested region. Exhibit A22-5 shows this flow-density diagram for a segment having a free-flow speed of 120 km/h. For other free-flow speeds, the corresponding capacities in Chapters 23 through 25 should be used.

EXHIBIT A22-5. SEGMENT FLOW-DENSITY FUNCTION
(SEE FOOTNOTE FOR ASSUMED VALUE)



Note:
Assumption: FFS = 120 km/h.

A.4.2 Flow Estimation

The oversaturated portion of the methodology is detailed in a flowchart as Exhibit A22-6. The flowchart is divided into nine parts, which are discussed in this section. Within each subsection, computations are detailed and labeled according to each step of the flowchart.

The procedure first calculates a number of flow variables starting at the first node during the first time step of oversaturation, followed by each downstream node and segment in that same time step. After all computations in the first time step are completed, calculations are performed at each node and segment during subsequent time steps for all remaining time intervals until the analysis is completed.

EXHIBIT A22-6. OVERSATURATED ANALYSIS PROCEDURE

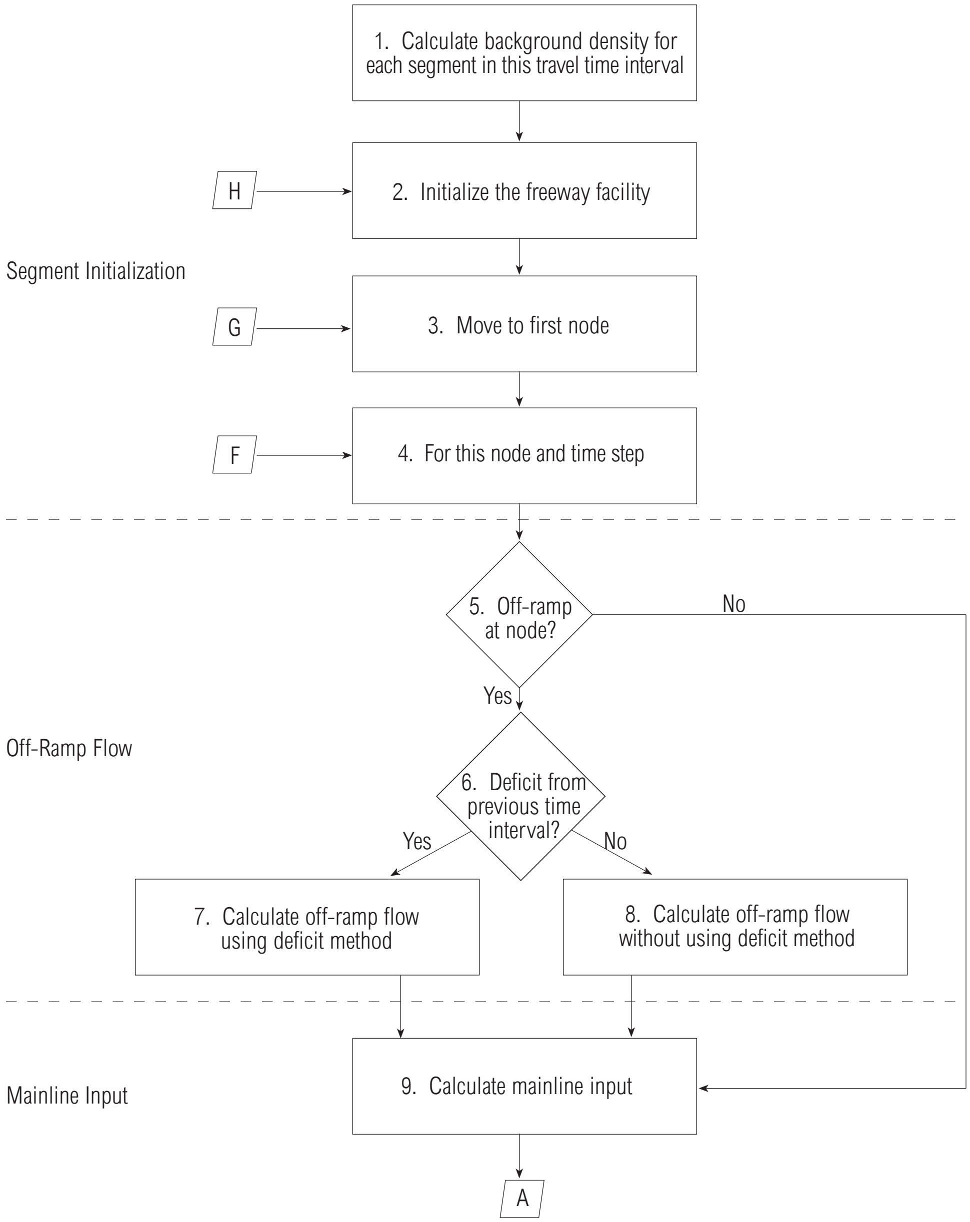


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EXHIBIT A22-6 (CONTINUED). OVERSATURATED ANALYSIS PROCEDURE

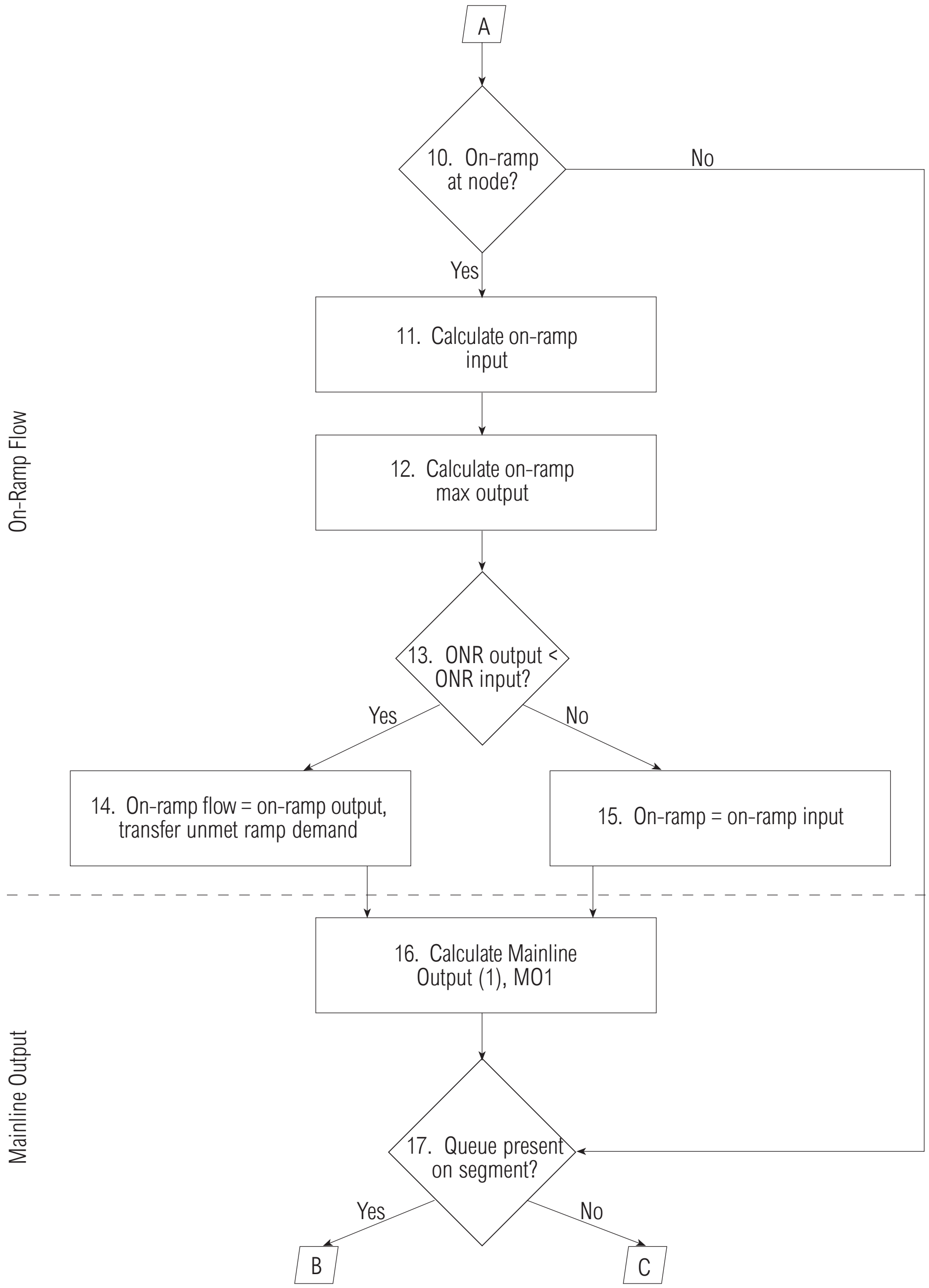

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EXHIBIT A22-6 (CONTINUED). OVERSATURATED ANALYSIS PROCEDURE

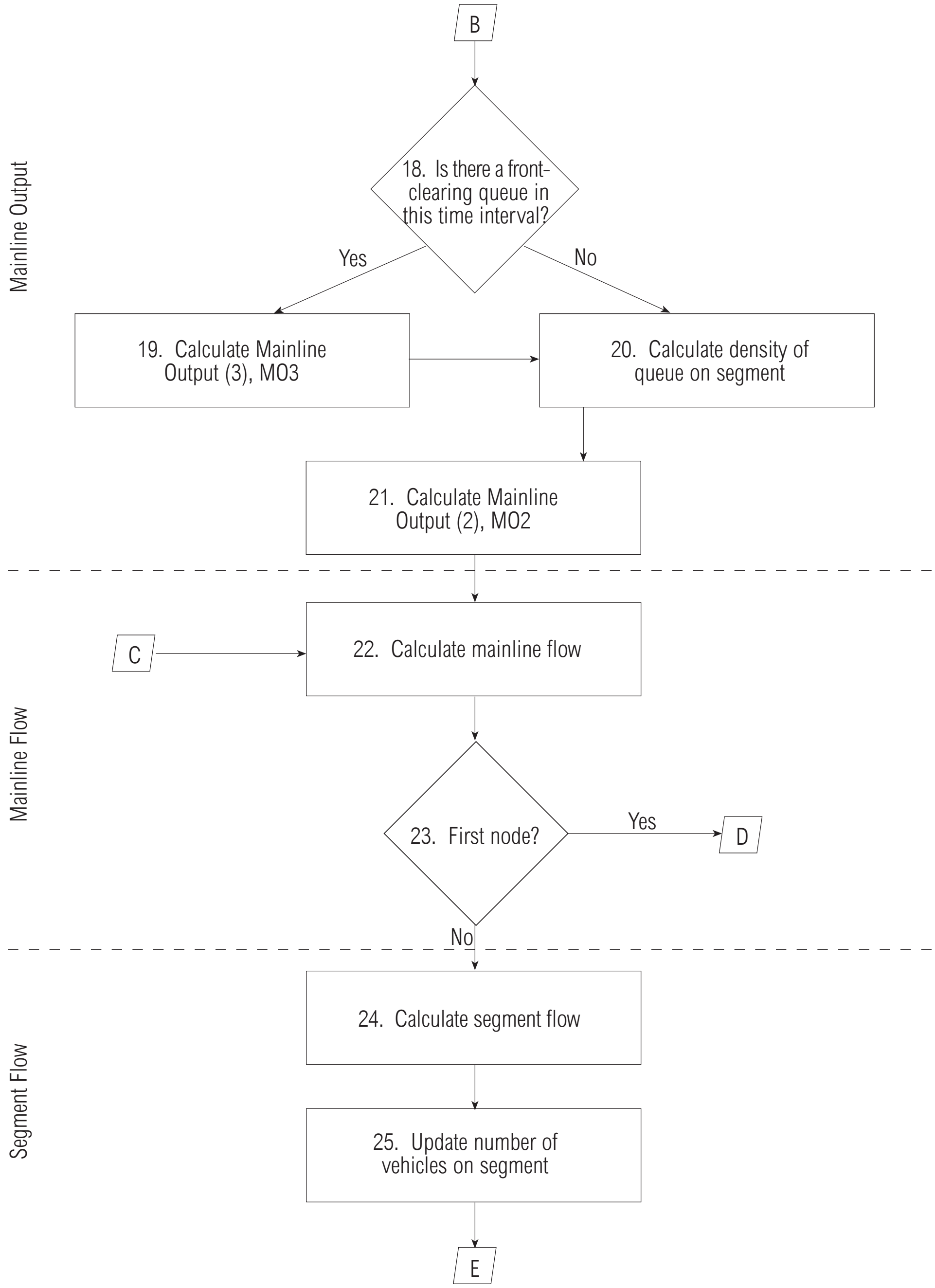

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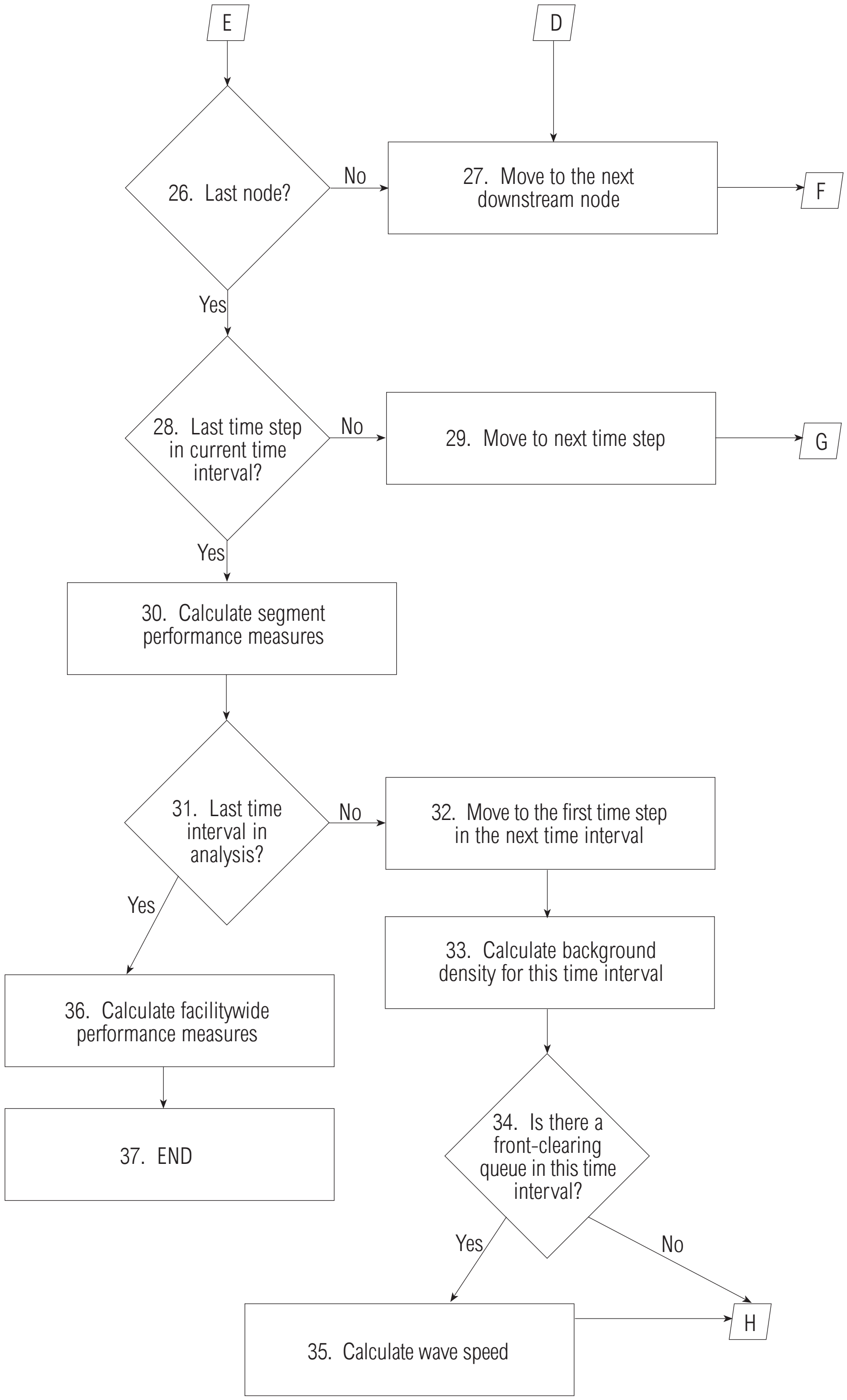
EXHIBIT A22-6 (CONTINUED). OVERSATURATED ANALYSIS PROCEDURE



Segment and Facility Performance Measures

A.4.2.1 Segment Initialization (Exhibit A22-6, Steps 1 Through 4)

Steps 1 through 4 of the oversaturated procedure prepare the flow calculations for the first time step and specify return points for subsequent time steps. To calculate the number of vehicles on each segment at the various time steps, the segments must contain the proper number of vehicles before the queuing analysis places unserved vehicles on segments. The initialization of each segment is described below.

A simplified queuing analysis is initially performed to account for the effects of upstream bottlenecks. These bottlenecks meter traffic downstream of their location. The storage of unserved vehicles (those unable to enter the bottleneck) on upstream segments is performed in a later module. To obtain the proper number of vehicles on each segment, the expected demand (ED) is calculated. ED is based on demands for and capacities of the segment and includes the effects of all upstream segments. The expected demand is the flow of traffic expected to arrive at each segment if all queues were stacked vertically (i.e., no upstream effects of queues). In other words, all segments upstream of a bottleneck have expected demands equal to their actual demand. The expected demand of the bottleneck segment and all further downstream segments are calculated assuming a capacity constraint at the bottleneck, which meters traffic to downstream segments. The expected demand is calculated for each segment using Equation A22-3.

$$ED(i, p) = min[SC(i, p), ED(i-1, p) + ONRD(i, p) - OFRD(i, p)] \qquad \text{(A22-3)}$$

Note that the segment capacity (SC) applies to the entire length of the segment. With the expected demand calculated, the background density (KB) can be obtained for each segment using the appropriate segment density estimation procedures in Chapters 23 through 25. The background density is used to calculate the number of vehicles on each segment (NV) using Equation A22-4. If there are unserved vehicles at the end of the preceding time interval, the unserved vehicles (UV) are transferred to the current time interval. Here, S refers to the last time step in the preceding time interval. The (0) term in NV represents the start of the first time step in Time Interval (p). The corresponding term at the end of the time step is NV(i, 1, p).

$$NV(i, 0, p) = KB(i, p) * L(i) + UV(i, S, p-1) \qquad \text{(A22-4)}$$

The number of vehicles calculated from the background density is the minimum number of vehicles that can be on the segment at any time. This is a powerful check on the methodology because the existence of queues downstream cannot reduce this minimum. Rather, the segment can only store additional vehicles. The storage of unserved vehicles will be determined in the segment flow calculation module later in this appendix.

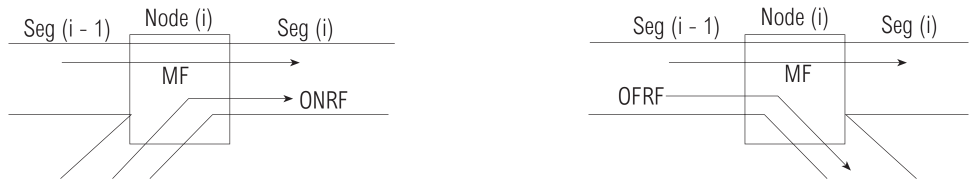
A.4.2.2 Mainline Flow Calculations (Exhibit A22-6, Steps 9 and 16 Through 23)

The description of ramp flows will follow the description of mainline flows. Thus, Steps 5 through 8 and 10 through 15 are skipped at this time to focus first on mainline flow computations. Because of the skipping of steps in the descriptions, some computations may include variables that have not been described but that have been previously calculated within the flowchart.

Flows analyzed in oversaturated conditions are calculated every time step and are expressed in terms of vehicles per time step. The procedure separately analyzes the flow across a node on the basis of the origin and destination of the flow across the node. The mainline flow is defined as the flow passing from upstream Segment (i – 1) to downstream Segment (i). It does not include the on-ramp flow. The flow to an off-ramp is the off-ramp flow. The flow from an on-ramp is the on-ramp flow. Each of these flows is shown in Exhibit A22-7 with their origin, destination, and relationship to Segment (i) and Node (i).

The segment flow is the total output of a segment, as shown in Exhibit A22-7. Segment flows are calculated by determining the mainline and ramp flows. The mainline flow is calculated as the minimum of six constraints: the mainline input, Mainline Output 1 (MO1), Mainline Output 2 (MO2), Mainline Output 3 (MO3), the upstream Segment (i – 1) capacity, and the downstream Segment (i) capacity, as explained next.

EXHIBIT A22-7. DEFINITIONS OF MAINLINE AND SEGMENT FLOWS



SF(i - 1) = MF(i)
SF(i) = MF(i) + ONRF(i)

SF(i - 1) = MF(i) + OFRF(i)
SF(i) = MF(i)

A.4.2.2.1 Mainline Input (Exhibit A22-6, Step 9)

The mainline input (MI) is the number of vehicles that wish to travel through a node during the time step. The calculation includes (a) the effects of bottlenecks upstream of the analysis node, (b) the metering of traffic during queue accumulation, and (c) the presence of additional traffic during upstream queue discharge.

The mainline input is calculated by taking the number of vehicles entering the node upstream of the analysis node, adding on-ramp flows or subtracting off-ramp flows, and adding the number of unserved vehicles on the upstream segment. This is the maximum number of vehicles that wish to enter a node during a time step. The mainline input is calculated using Equation A22-5, where all values have units of vehicles per time step.

$$MI(i, t, p) = MF(i-1, t, p) + ONRF(i-1, t, p) - OFRF(i, t, p) + UV(i-1, t-1, p) \quad \text{(A22-5)}$$

A.4.2.2.2 Mainline Output (Exhibit A22-6, Steps 16 Through 21)

The mainline output is the maximum number of vehicles that can exit a node, constrained by downstream bottlenecks or by merging on-ramp traffic. Different constraints on the output of a node result in three separate types of mainline outputs (MO1, MO2, and MO3).

A.4.2.2.2.1 Mainline Output 1—Ramp Flows (Exhibit A22-6, Step 16)

MO1 is the constraint caused by the flow of vehicles from an on-ramp. The capacity of an on-ramp segment is shared by two competing flows. This on-ramp flow limits the flow from the mainline through this node. The total flow that can pass the node is estimated as the minimum of the Segment (i) capacity and the mainline outputs from the preceding time step. The sharing of Lane 1 (shoulder lane) capacity is determined in the calculation of the on-ramp flow and is described in Section A.4.2.4. MO1 is calculated using Equation A22-6.

$$MO1(i, t, p) = min[SC(i, t, p) - ONRF(i, t, p), MO2(i, t-1, p), MO3(i, t-1, p)] \quad \text{(A22-6)}$$

A.4.2.2.2.2 Mainline Output 2—Segment Storage (Exhibit A22-6, Steps 20 and 21)

The second constraint on the output of mainline flow through a node is caused by the growth of queues on a downstream segment. As a queue grows on a segment, it may eventually limit the flow into the current segment once the boundary of the queue reaches the upstream end of the segment. The boundary of the queue is treated as a shock wave. MO2 is a limit on the flow exiting a node due to the presence of a queue on the downstream segment.

The MO2 limitation is determined first by calculating the maximum number of vehicles allowed on a segment at a given queue density. The maximum flow that can enter a queued segment is the number of vehicles that leave the segment plus the difference between the maximum number of vehicles allowed on the segment and the number of vehicles already on the segment. The density of the queue is calculated using Equation A22-7 for the linear density-flow relationship shown in Exhibit A22-5 earlier.

$$KQ(i, t, p) = KJ - \frac{(KJ - KC) * SF(i, t-1, p)}{SC(i, p)} \quad \text{(A22-7)}$$

Once the queue density is computed, MO2 can be computed using Equation A22-8.

$$MO2(i, t, p) = SF(i, t-1, p) - ONRF(i, t, p) + [KQ(i, t, p) * L(i)] - NV(i, t-1, p) \quad \text{(A22-8)}$$

The performance of the downstream node is estimated by taking the performance during the preceding time step. This estimation remains valid when there are no interacting queues. When queues do interact and the time steps are small enough, the error in the estimations are corrected in the next time step.

A.4.2.2.2.3 Mainline Output 3—Front-Clearing Queues (Exhibit A22-6, Steps 17 Through 19)

The final constraint on exiting mainline flows at a node is caused by downstream queues clearing from their downstream end. These front-clearing queues are typically caused by incidents where there is a temporary reduction in capacity. A queue will clear from the front if two conditions are satisfied. First, the segment capacity (minus the on-ramp demand if present) for this time interval must be greater than the segment capacity (minus the ramp demand if present) in the preceding time interval. The second condition is that the segment capacity minus the ramp demand for this time interval be greater than the segment demand for this time interval. A queue will clear from the front if both conditions in the following inequality (Equation A22-9) are met.

$$\text{If } [SC(i, p) - ONRD(i, p)] > [SC(i, p-1) - ONRD(i, p-1)]$$

$$\text{and } [SC(i, p) - ONRD(i, p)] > SD(i, p) \quad \text{(A22-9)}$$

A segment with a front-clearing queue will have the number of vehicles stored decrease during recovery, while the back of queue position is unaffected. Thus, the clearing does not affect the segment throughput until the recovery wave has reached the upstream end of the segment. In the flow-density graph shown in Exhibit A22-8, the wave speed is estimated by the slope of the dotted line connecting the bottleneck throughput and the segment capacity points.

The assumption of a linear flow-density function greatly simplifies the calculated wave speed. The bottleneck throughput value is not required to estimate the speed of the shock wave that travels along a known line. All that is required is the slope of the line, which is calculated using Equation A22-10.

$$WS(i, p) = \frac{SC(i, p)}{N(i, p) * (KJ - KC)} \quad \text{(A22-10)}$$

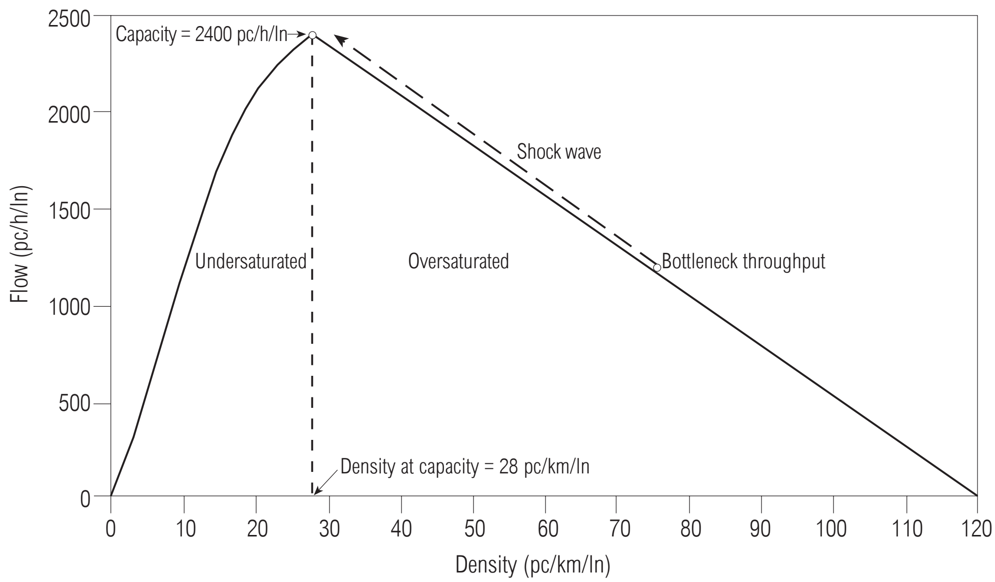
The wave speed is used to calculate the time it takes the front queue-clearing shock wave to traverse this segment, called the wave travel time (WTT). Dividing the wave speed (WS) by the segment length in kilometers gives the wave travel time.

The recovery wave travel time is the time required for the conditions at the downstream end of the current segment to reach the upstream end of the current segment. To place a limit on the current node, the conditions at the downstream node are observed at a time in the past. This time is the wave travel time. This constraint on the current node is called the Mainline Output 3, or MO3. The calculation of MO3 is performed by using Equations A22-11 and A22-12 . If the wave travel time is not an integer number of time steps, then the weighted average performance of each variable is taken for the time steps nearest to the wave travel time. This method is based on the process described in References *2–4*.

$$WTT = \frac{T * L(i)}{WS(i, p)} \quad \text{(A22-11)}$$

$$MO3(i, t, p) = min[MO1(i + 1, t - WTT, p),$$
$$MO2(i + 1, t - WTT, p) + OFRF(i + 1, t - WTT, p), MO3(i + 1, t - WTT, p)$$
$$+ OFRF(i + 1, t - WTT, p), SC(i, t - WTT, p), SC(i + 1, t - WTT, p)$$
$$+ OFRF(i + 1, t - WTT, p)] - ONRF(i, t, p) \quad \text{(A22-12)}$$

EXHIBIT A22-8. FLOW-DENSITY FUNCTION WITH A SHOCK WAVE
(SEE FOOTNOTE FOR ASSUMED VALUE)



Note:
Assumption: FFS = 120 km/h.

A.4.2.2.3 Mainline Flow (Exhibit A22-6, Steps 22 and 23)

The flow across a node is called the mainline flow and is the minimum of the following variables: mainline input, MO1, MO2, MO3, upstream Segment (i – 1) capacity, and downstream Segment (i) capacity.

$$MF(i, t, p) = min\ [MI(i, t, p), MO1(i, t, p), MO2(i, t, p), MO3(i, t, p),$$
$$SC(i, t, p), SC(i - 1, t, p)] \quad \text{(A22-13)}$$

In addition to mainline flows, ramp flows must be analyzed. The presence of mainline queues also affects ramp flows.

A.4.2.3 On-Ramp Calculations (Exhibit A22-6, Steps 10 Through 15)

A.4.2.3.1 On-Ramp Input (Exhibit A22-6, Steps 10 and 11)

The maximum on-ramp input is calculated by adding the on-ramp demand and the number of vehicles queued on the ramp. The queued vehicles are treated as unmet ramp demand that was not served in previous time steps. The on-ramp input is calculated using Equation A22-14.

$$ONRI(i, t, p) = ONRD(i, t, p) + ONRQ(i, t - 1, p) \quad \text{(A22-14)}$$

A.4.2.3.2 On-Ramp Output (Exhibit A22-6, Step 12)

The maximum on-ramp output is calculated on the basis of the mainline traffic through the node where the on-ramp is located. The on-ramp output is the minimum of two values. The first is Segment (i) capacity minus the mainline input, in the absence of downstream queues. Otherwise, the segment capacity is replaced by the throughput of the queue. This estimation implies that vehicles entering an on-ramp segment will fill lanes 2 to N (where N is the number of lanes on the current segment) to capacity before entering Lane 1. This assumption appears to be consistent with the estimation of V_{12} from Chapter 25 of this manual.

The second case is when the Lane 1 flow on Segment (i) is greater than one-half of the Lane 1 capacity. At this point the on-ramp maximum output is set to one-half of Lane 1 capacity. This implies that when the demands from the freeway and the on-ramp are very high, there will be forced merging in a one-to-one fashion on the freeway from the freeway mainline and the on-ramp in Lane 1. An important characteristic of traffic behavior is that in a forced merging situation, ramp and right-lane freeway vehicles will generally merge one on one, sharing the capacity of the rightmost freeway lane (*5*). In all cases, the on-ramp maximum output is also limited to the physical ramp road capacity and the ramp-metering rate, if present. The maximum on-ramp output is an important limitation on the ramp flow. Queuing occurs when the combined demand from the upstream segment and the demand on the on-ramp exceed the throughput of the ramp segment. The queue can be located on the upstream segment, on the ramp, or on both and is dependent on the on-ramp maximum output. Equation A22-15 determines the value of the maximum on-ramp output.

$$\begin{aligned} ONRO(i, t, p) = min\{&RM(i, p), ONRC(i, p), max[min[SC(i, p), \\ &MO2(i, t-1, p) + ONRF(i, t-1, p), MO3(i, t-1, p) + ONRF(i, t-1, p)] - MI(i, t, p), \\ &min[SC(i, p), MO2(i, t-p) + ONRF(i, t-1, p), MO3(i, t-1, p) \\ &+ ONRF(i, t-1, p)]/2N(i, p)]\} \end{aligned} \tag{A22-15}$$

Note that this model incorporates the maximum mainline output constraints from downstream queues, not just the segment capacity. This is significant because as a queue spills over an on-ramp segment, the flow through Lane 1 is constrained. This, in turn, limits the flow that can enter Lane 1 from the on-ramp. The values of MO2 and MO3 for this time step are not yet known, so they are estimated from the preceding time step. This estimation is one rationale for using small time steps. If there is forced merging during the time step where the queue spills back over the current node, the on-ramp will discharge more than its share of vehicles (i.e., more than 50 percent of the Lane 1 flow). This will cause the mainline flow past Node i to be underestimated. But during the next time step, the on-ramp flow will be at its correct flow rate, and a one-to-one sharing of Lane 1 will occur.

A.4.2.3.3 On-Ramp Flows, Queues, and Delays (Exhibit A22-6, Steps 13 Through 15)

Finally, the on-ramp flow is calculated on the basis of the on-ramp input and output values computed above. If the on-ramp input is less than the on-ramp output, then the on-ramp demand can be fully served in this time step and Equation A22-16 is used.

$$ONRF(i, t, p) = ONRI(i, t, p) \tag{A22-16}$$

Otherwise, the ramp flow is constrained by the maximum on-ramp output, and Equation A22-17 is used.

$$ONRF(i, t, p) = ONRO(i, t, p) \tag{A22-17}$$

In the latter case, the number of vehicles in the ramp queue is updated using Equation A22-18.

$$ONRQ(i, t, p) = ONRQ(i, t-1, p) + ONRI(i, t, p) - ONRO(i, t, p) \tag{A22-18}$$

The total delay for on-ramp vehicles can be estimated by integrating the value of on-ramp queues over time. The methodology uses the discrete queue lengths estimated at the end of each interval, ONRQ(i, S, p), to produce overall ramp delays by time interval.

A.4.2.4 Off-Ramp Flow Calculation (Exhibit A22-6, Steps 5 Through 8)

The off-ramp flow is determined by calculating a diverge percentage on the basis of the segment and off-ramp demands. The diverge percentage varies only by time interval and remains constant for vehicles that are associated with a particular time interval. If there is an upstream queue, traffic may be metered to this off-ramp. This will cause a decrease in the off-ramp flow. When the vehicles that were metered arrive in the next time interval, they use the diverge percentage associated with the preceding time interval.

A deficit in flow, caused by traffic from an upstream queue meter, creates delays for vehicles destined to this off-ramp and other downstream destinations. The upstream segment flow is used because the procedure assumes that a vehicle destined for an off-ramp is able to exit at the off-ramp once it enters the off-ramp segment. The calculation of this deficit is performed using Equation A22-19.

$$\mathrm{DEF}(i,t,p) = \mathrm{Max}\left\{0, \sum_{X=1}^{p-1} \mathrm{SD}(i-1,X) - \left[\sum_{X=1}^{p-1}\sum_{t=1}^{T}\left[\mathrm{MF}(i-1,t,X)+\mathrm{ONRF}(i-1,t,X)\right] + \sum_{t=1}^{t-1}\left[\mathrm{MF}(i-1,t,p)+\mathrm{ONRF}(i-1,t,p)\right]\right]\right\} \quad \text{(A22-19)}$$

If there is a deficit, then the off-ramp flow is calculated using the deficit method. The deficit method is used differently in two different situations. If the upstream mainline flow plus the flow from an on-ramp at the upstream node (if present) is less than the deficit for this time step, then the off-ramp flow is equal to the mainline and on-ramp flows times the off-ramp turning percentage in the preceding time interval, as indicated below.

$$OFRF(i,t,p) = [MF(i-1,t,p) + ONRF(i-1,t,p)] * [OFRD(i,p-1)/SD(i-1,p-1)] \quad \text{(A22-20)}$$

If the deficit is less than the upstream mainline flow plus the on-ramp flow from an on-ramp at the upstream node (if present), then Equation A22-21 is used. This equation separates the flow into the remaining deficit flow and the balance of the arriving flow.

$$OFRF(i,t,p) = DEF(i,t,p) * [OFRD(i,p-1)/SD(i-1,p-1)] + [MF(i-1,t,p) + ONRF(i-1,t,p) - DEF(i,t,p)] * [OFRD(i,p)/SD(i-1,p)] \quad \text{(A22-21)}$$

If there is no deficit, then the off-ramp flow is equal to the sum of upstream mainline flow plus the on-ramp flow from an on-ramp at the upstream node (if present), multiplied by the off-ramp turning percentage for this time interval according to Equation A22-22.

$$OFRF(i,t,p) = [MF(i-1,t,p) + ONRF(i-1,t,p)] * [OFRD(i,p)/SD(i-1,p)] \quad \text{(A22-22)}$$

Note that the procedure does not currently incorporate any delay or queue length computations for off-ramps.

A.4.2.5 Segment Flow Calculation (Exhibit A22-6, Steps 24 and 25)

The segment flow is the number of vehicles that flow out of a segment during the current time step. These vehicles enter the current segment either to the mainline or to an off-ramp at the current node. The vehicles that entered the upstream segment may or may not have become queued within the segment. The segment flow is calculated using Equation A22-23.

$$SF(i-1,t,p) = MF(i,t,p) + OFRF(i,t,p) \quad \text{(A22-23)}$$

The number of vehicles on each segment is calculated on the basis of the number of vehicles that were on the segment in the preceding time step, the number of vehicles that entered the segment in this time step, and the number of vehicles that leave the segment in this time step. Because the number of vehicles that leave a segment must be known, the number of vehicles on the current segment cannot be determined until the upstream segment is analyzed. The number of vehicles on each segment is calculated using Equation A22-24.

$$NV(i-1, t, p) = NV(i-1, t-1, p) + MF(i-1, t, p) + ONRF(i-1, t, p) - MF(i, t, p) - OFRF(i, t, p) \quad \text{(A22-24)}$$

The number of unserved vehicles stored on a segment is calculated as the difference between the number of vehicles on the segment and the number of vehicles that would be on the segment at the background density. The number of unserved vehicles stored on a segment is calculated using Equation A22-25.

$$UV(i-1, t, p) = NV(i-1, t, p) - [KB(i-1, p) * L(i-1)] \quad \text{(A22-25)}$$

A.4.3 Segment and Ramp Performance Measures (Exhibit A22-6, Steps 26 Through 30)

In the last time step of a time interval, the segment flows are averaged over the time interval and the measures of effectiveness for each segment are calculated. If there was no queue on a particular segment during the entire time interval, then the performance measures are calculated from the corresponding HCM 2000 method for that segment in Chapters 23 through 25. Since there are T time steps in an hour, the average segment flow rate in vehicles per hour in Time Interval (p) is calculated using Equation A22-26.

$$SF(i, p) = \frac{T}{S} \sum_{t=1}^{S} SF(i, t, p) \quad \text{(A22-26)}$$

Note that if T = 60 (1-min time step) and S = 15 (interval = 15 min), then T/S = 4. If there was a queue on the current segment in any time step during the time interval, then the segment performance measures are calculated in three steps. First, the average number of vehicles over a time interval is calculated for each segment using Equation A22-27.

$$NV(i, p) = \frac{I}{S} \sum_{t=1}^{S} NV(i, t, p) \quad \text{(A22-27)}$$

Next, the average segment density is calculated by taking the average number of vehicles (NV) for all time steps in the time interval and dividing it by the segment length using Equation A22-28.

$$K(i, p) = \frac{NV(i, p)}{L(i)} \quad \text{(A22-28)}$$

Next, the average speed on the current segment (i) during the current time interval (p) is calculated using Equation A22-29.

$$U(i, p) = \frac{SF(i, p)}{K(i, p)} \quad \text{(A22-29)}$$

Additional segment performance measures can be derived from the basic measures shown in Equations A22-26 through A22-28. Most prominent is segment delay, which can be computed as the difference in segment travel time at speed U(i, p) and at the segment free-flow speed.

The final segment performance measure is the length of the queue at the end of the time interval (i.e., Step S in Time Interval p). The length of a queue in meters on the segment is calculated using Equation A22-30.

$$Q(i,S,p)=\frac{UV(i,S,p)}{KQ(i,S,p)-KB(i,p)}*1000 \qquad \text{(A22-30)}$$

Queue length on on-ramps can also be calculated. A queue will form on the on-ramp roadway only if the flow is limited by a metering rate or by the merge area capacity. If the flow is limited by the ramp capacity, then unserved vehicles will be stored upstream of the ramp roadway, most likely a surface street. The methodology does not account for this delay, since vehicles cannot enter the ramp roadway. However, the unserved vehicles in this case are transferred as added demand in subsequent time intervals. If the queue is on the ramp roadway, the queue length is calculated by using the difference in background density and queue density. For an on-ramp, the background density is assumed to be the density at capacity and the queue density is calculated within Equation A22-31. For on-ramp queue length, Equation A22-31 is used.

$$ONRQL(i,S,p)=\frac{ONRQ(i,S,p)}{KJ-\dfrac{\min[RM(i,p),ONRO(i,S,p)*(KJ-KC)]}{ONRC(i,p)}} \qquad \text{(A22-31)}$$

A.5 DIRECTIONAL FACILITY MODULE (EXHIBIT A22-6, STEP 36)

The previously discussed traffic performance measures can be aggregated over the length of the directional freeway facility, over the time duration of the study interval, or over the entire time-space domain. Each will be discussed in the following paragraphs.

Aggregating the estimated traffic performance measures over the entire length of the freeway facility provides facilitywide estimates for each time interval. Facilitywide travel times, vehicle (and person) distance of travel, and vehicle (and person) hours of travel and delay can be computed, and patterns of their variation over the connected time intervals can be assessed. The current computer implementation of the methodology is limited to 15-min time intervals and 1-min time steps.

Aggregating the estimated traffic performance measures over the time duration of the study interval provides an assessment of the performance of each segment along the freeway facility. Average and cumulative distributions of speed and density for each segment can be determined, and patterns of the variation over connected freeway segments can be compared. Average trip times, vehicle (and person) distance of travel, and vehicle (and person) hours of travel are easily assessed for each segment and compared.

Aggregating the estimated traffic performance measures over the entire time-space domain provides an overall assessment over the study interval time duration. Overall average speeds, average trip times, total vehicle (and person) distance traveled, and total vehicle (and person) hours of travel and delay are the most obvious overall traffic performance measures. Equations A22-32 through A22-35 show how some of the facilitywide MOEs are calculated.

Facility space-mean speed in Time Interval (p):

$$SMS(NS,p)=\frac{\sum_{i=1}^{NS} SF(i,p)*L(i)}{\sum_{i=1}^{NS} SF(i,p)*\dfrac{L(i)}{U(i,p)}} \qquad \text{(A22-32)}$$

Average facility density in Time Interval (p):

$$K(NS,p)=\frac{\sum_{i=1}^{NS} K(i,p)*L(i)}{\sum_{i=1}^{NS} L(i)N(i,p)} \qquad \text{(A22-33)}$$

Overall space-mean speed across all intervals:

$$SMS(NS,P)=\frac{\sum_{p=1}^{P}\sum_{i=1}^{NS} SF(i,p)L(i)}{\sum_{p=1}^{P}\sum_{i=1}^{NS} SF(i,p)\dfrac{L(i)}{U(i,p)}} \qquad \text{(A22-34)}$$

Overall average density across all intervals:

$$K(NS, P) = \frac{\sum_{p=1}^{P} \sum_{i=1}^{NS} K(i, p) * L(i)}{\sum_{p=1}^{P} \sum_{i=1}^{NS} L(i) N(i, p)} \quad \text{(A22-35)}$$

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