

**CHAPTER 13**

**FREEWAY CONCEPTS**

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## I. INTRODUCTION

In this chapter capacity and quality-of-service concepts for freeways are introduced. This chapter can be used in conjunction with the methodologies of Chapter 22 (Freeway Facilities), Chapter 23 (Basic Freeway Segments), Chapter 24 (Freeway Weaving), and Chapter 25 (Ramps and Ramp Junctions).

A freeway is defined as a divided highway with full control of access and two or more lanes for the exclusive use of traffic in each direction. Freeways provide uninterrupted flow. There are no signalized or stop-controlled at-grade intersections, and direct access to and from adjacent property is not permitted. Access to and from the freeway is limited to ramp locations. Opposing directions of flow are continuously separated by a raised barrier, an at-grade median, or a continuous raised median.

Operating conditions on a freeway primarily result from interactions among vehicles and drivers in the traffic stream and among vehicles, drivers, and the geometric characteristics of the freeway. Operations can also be affected by environmental conditions, such as weather or lighting, by pavement conditions, and by the occurrence of traffic incidents.

A tollway or toll road is similar to a freeway, except that tolls are collected at designated points along the facility. Although the collection of tolls usually involves interruptions of traffic, these facilities may generally be treated as freeways. However, special attention should be given to the unique characteristics, constraints, and delays caused by toll collection facilities.

The freeway system is the sum total of all freeway facilities in a given area. The analyst must realize that freeway facilities may have interactions with other freeway facilities as well as local streets and take care to consider interactions with these other facilities. The performance of the freeway may be affected when demand exceeds capacity on nearby parts of the local street or freeway system or when the capacity of the street or ramp metering system limits demand approaching the freeway.

If the street system cannot accommodate the demand exiting the freeway, the oversaturation of the street system may result in queues backing onto the freeway, which adversely affects freeway performance. In effect, the limited capacity of the street system reduces the effective capacity of the exit ramp. Therefore, whether the downstream street system capacity can accommodate the exiting freeway demand is an important factor in whether the freeway facility analysis reflects freeway performance. Likewise, the presence of ramp metering affects freeway demand and must be taken into consideration in analyzing a freeway facility.

Freeway facilities are also assumed to have no interaction with adjacent freeways. In reality, freeway facilities may have interactions with other freeway facilities, as they do with surface streets. Free-flow conditions must therefore exist upstream and downstream of the facility being analyzed. In other words, the analysis of a freeway facility can only address local oversaturation within its time-space domain, not systemwide effects outside of its time-space domain.

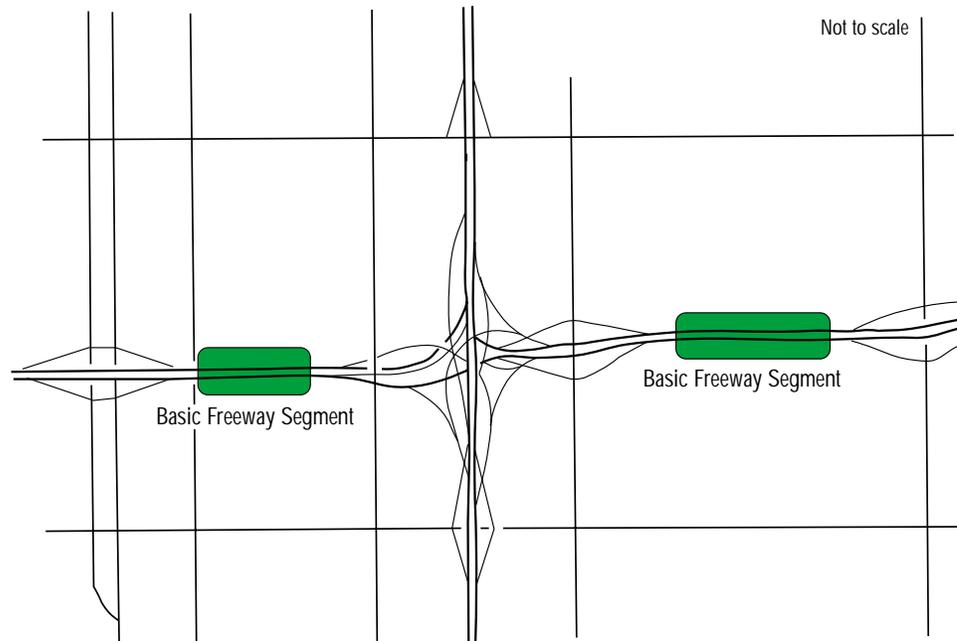
## II. BASIC FREEWAY SEGMENTS

Basic freeway segments are outside of the influence area of ramps or weaving areas of the freeway. Exhibit 13-1 illustrates a basic freeway segment.

Freeway defined

Toll road as freeway

EXHIBIT 13-1. EXAMPLE OF BASIC FREEWAY SEGMENT



### **FREEWAY CAPACITY TERMS**

- Freeway capacity: the maximum sustained 15-min flow rate, expressed in passenger cars per hour per lane, that can be accommodated by a uniform freeway segment under prevailing traffic and roadway conditions in one direction of flow.
- Traffic characteristics: any characteristic of the traffic stream that may affect capacity, free-flow speed, or operations, including the percentage composition of the traffic stream by vehicle type and the familiarity of drivers with the freeway.
- Roadway characteristics: the geometric characteristics of the freeway segment under study, including the number and width of lanes, right-shoulder lateral clearance, interchange spacing, vertical alignment, and lane configurations.
- Free-flow speed (FFS): the mean speed of passenger cars that can be accommodated under low to moderate flow rates on a uniform freeway segment under prevailing roadway and traffic conditions.
- Base conditions: an assumed set of geometric and traffic conditions used as a starting point for computations of capacity and level of service (LOS).

Capacity analysis is based on freeway segments with uniform traffic and roadway conditions. If any of the prevailing conditions change significantly, the capacity of the segment and its operating conditions change as well. Therefore, each uniform segment should be analyzed separately.

### **FLOW CHARACTERISTICS**

Traffic flow within basic freeway segments can be highly varied depending on the conditions constricting flow at upstream and downstream bottleneck locations. Bottlenecks can be created by ramp merge and weaving segments, lane drops, maintenance and construction activities, accidents, and objects in the roadway. An incident does not have to block a travel lane to create a bottleneck. For example, disabled vehicles in the median or on the shoulder can influence traffic flow within the freeway lanes.

Freeway research has resulted in a better understanding of the characteristics of freeway flow relative to the influence of upstream and downstream bottlenecks. Traffic flow within a basic freeway segment can be categorized into three flow types: undersaturated, queue discharge, and oversaturated. Each flow type is defined within general speed-flow-density ranges, and each represents different conditions on the freeway.

- Undersaturated flow represents traffic flow that is unaffected by upstream or downstream conditions. This regime is generally defined within a speed range of 90 to 120 km/h at low to moderate flow rates and a range of 70 to 100 km/h at high flow rates.
- Queue discharge flow represents traffic flow that has just passed through a bottleneck and is accelerating back to the FFS of the freeway. Queue discharge flow is characterized by relatively stable flow as long as the effects of another bottleneck downstream are not present. This flow type is generally defined within a narrow range of 2,000 to 2,300 pc/h/ln, with speeds typically ranging from 55 km/h up to the FFS of the freeway segment. Lower speeds are typically observed just downstream of the bottleneck. Depending on horizontal and vertical alignments, queue discharge flow usually accelerates back to the FFS of the facility within 1 to 2 km downstream from the bottleneck. Studies suggest that the queue discharge flow rate from the bottleneck is lower than the maximum flows observed before breakdown. A typical value for this drop in flow rate is approximately 5 percent.
- Oversaturated flow represents traffic flow that is influenced by the effects of a downstream bottleneck. Traffic flow in the congested regime can vary over a broad range of flows and speeds depending on the severity of the bottleneck. Queues may extend several kilometers upstream from the bottleneck. Freeway queues differ from queues at intersections in that they are not static or “standing.” On freeways, vehicles move slowly through a queue, with periods of stopping and movement. Oversaturated flow is discussed further in the freeway facilities section of this chapter and in Chapter 22.

The three flow types on basic freeway segments are undersaturated, queue discharge, and oversaturated

Undersaturated flow defined

Queue discharge flow defined

Oversaturated flow defined

### Speed-Flow and Density-Flow Relationships

Speed-flow and density-flow relationships for a typical basic freeway segment under either base conditions or non-base conditions in which FFS is known are shown in Exhibits 13-2 and 13-3 (1).

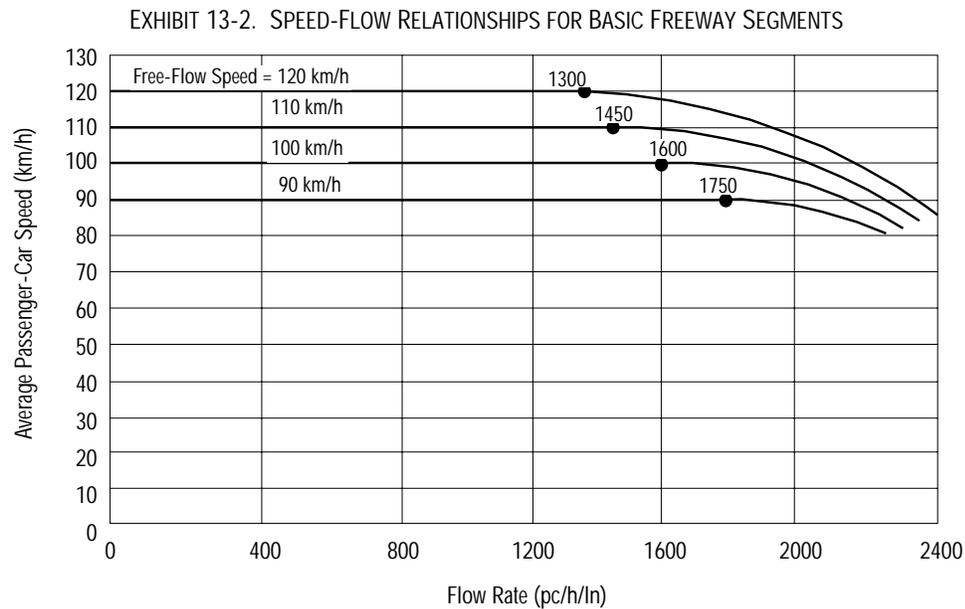
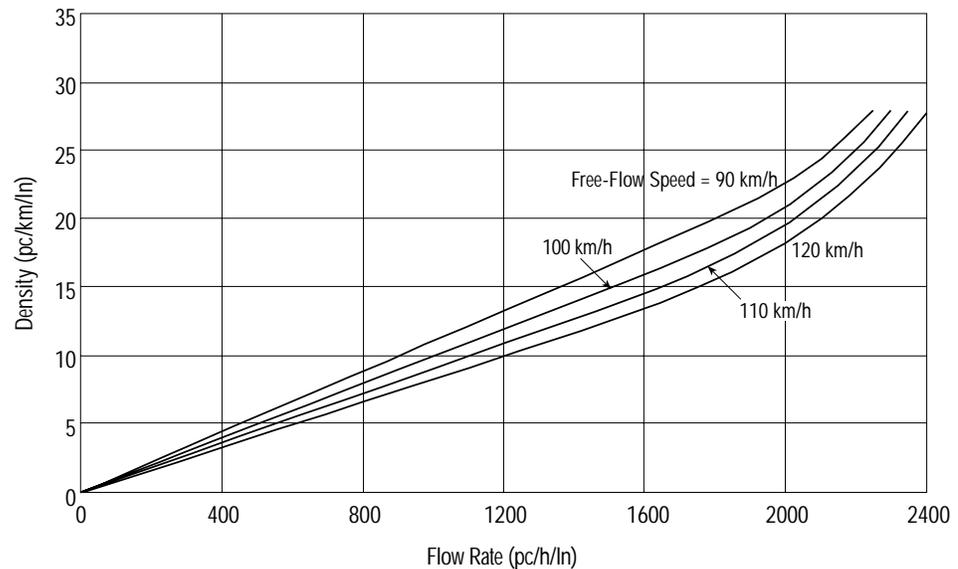


EXHIBIT 13-3. DENSITY-FLOW RELATIONSHIPS FOR BASIC FREEWAY SEGMENTS



All recent freeway studies indicate that speed on freeways is insensitive to flow in the low to moderate range. This is reflected in Exhibit 13-2, which shows speed to be constant for flows up to 1,300 pc/h/ln for a 120-km/h FFS. For lower free-flow speeds, the region over which speed is insensitive to flow extends to even higher flow rates. FFS is measured in the field as the average speed of passenger cars when flow rates are less than 1,300 pc/h/ln. Field determination of FFS is accomplished by performing travel time or spot speed studies during periods of low flows and low densities.

Although Exhibit 13-2 shows curves only for free-flow speeds of 120, 110, 100, and 90 km/h, a curve representing any FFS between 120 and 90 km/h can be defined by interpolation.

The research leading to these curves found that a number of factors affect free-flow speed ( $I$ ). The factors include number of lanes, lane width, lateral clearance, and interchange density or spacing. Other factors believed to influence FFS, but for which little is known quantitatively, include horizontal and vertical alignment, speed limit, level of enforcement, lighting conditions, and weather.

Under base traffic and geometric conditions, freeways will operate with capacities as high as 2,400 pc/h/ln. This capacity is typically achieved on freeways with FFS of 120 km/h or greater. As the FFS decreases, there is a slight decrease in capacity. For example, the capacity of a basic freeway segment with a FFS of 90 km/h is expected to be approximately 2,250 pc/h/ln.

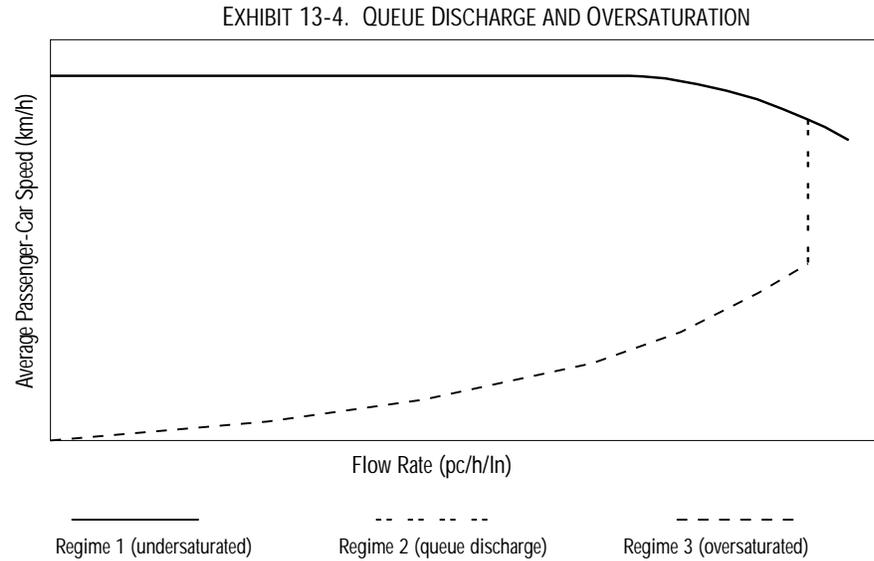
The average speed of passenger cars at flow rates that represent capacity is expected to range from 86 km/h (FFS of 120 km/h or greater) to 80 km/h for a segment with a 90-km/h FFS. Note that the higher the free-flow speed, the greater the drop in speed as flow rates move toward capacity. Thus, for a 120-km/h FFS, there is a 34-km/h drop from low-volume conditions to capacity conditions. The drop is only 10 km/h for a freeway with a 90-km/h FFS.

As indicated in Exhibit 13-2, the point at which an increase in flow rate begins to affect the average passenger car speed varies from 1,300 to 1,750 pc/h/ln. Speed will be reduced beginning at 1,300 pc/h/ln for freeway segments with free-flow speeds of 120 km/h. For lower-FFS facilities, the average speed begins to diminish at higher flow rates.

Interpolation is used to derive values between the curves

**Queue Discharge and Oversaturation**

Unlike free flow, queue discharge and congested flow have not been extensively studied, and these traffic flow types can be highly variable. However, freeway research performed since 1990 has provided valuable insight into possible speed-flow relationships that describe these two flow regimes. Exhibit 13-4 presents one suggested relationship and is displayed here for informational purposes only (2).



Source: Adapted from Hall et al. (2).

Analysts are cautioned that although the relationship in Exhibit 13-4 may provide a general predictive model for speed under queue discharge and oversaturated flows, it should be considered conceptual at best. Further research is needed to better define flow in these two regimes.

**FACTORS AFFECTING FFS**

The FFS of a freeway depends on traffic and roadway conditions. These conditions are described below.

**Lane Width and Lateral Clearance**

When lane widths are less than 3.6 m, drivers are forced to travel closer to one another laterally than they would normally desire. Drivers tend to compensate for this by reducing their travel speed.

The effect of restricted lateral clearance is similar. When objects are located too close to the edge of the median and roadside lanes, drivers in these lanes will shy away from them, positioning themselves further from the lane edge. This has the same effect as narrow lanes, which force drivers closer together laterally. Drivers compensate by reducing their speed. The closeness of objects has been found to have a greater effect on drivers in the rightmost travel lane than on those in the median lane.

Drivers in the median lane appear to be unaffected by lateral clearance when minimum clearance is 0.6 m, whereas drivers in the right (shoulder) lane are affected when lateral clearance is less than 1.8 m. Illustration 13-1 shows the influence of lane width and lateral clearance on lateral placement of vehicles. Illustration 13-2 shows a freeway segment considered to meet or exceed base conditions with respect to lane width and lateral clearance.

Lateral clearance is measured from edge of travel lane to curb, guardrail, or other physical obstruction

Note how vehicles shy away from both roadside and median barriers, driving as close to the lane marking as possible. The existence of narrow lanes compounds the problem, making it difficult for two vehicles to travel alongside each other.

This cross section illustrates base conditions of lane width and lateral clearance. The concrete median barrier does not cause vehicles to shift their lane position and therefore would not be considered an obstruction.

The horizontal and vertical geometry may influence free-flow speed



ILLUSTRATION 13-1.



ILLUSTRATION 13-2.

### Number of Lanes

The number of lanes on a freeway segment influences FFS. As the number of lanes increases, so does the opportunity for drivers to position themselves to avoid slower-moving traffic. In typical freeway driving, traffic tends to be distributed across lanes according to speed. Traffic in the median lane typically moves faster than in the lane adjacent to the right shoulder. Thus, a four-lane freeway (two lanes in each direction) provides less opportunity for drivers to move around slower traffic than does a freeway with 6, 8, or 10 lanes. Decreased maneuverability tends to reduce the average speed of vehicles.

### Interchange Density

Freeway segments with closely spaced interchanges, such as those in heavily developed urban areas, operate at lower FFS than suburban or rural freeways where interchanges are less frequent. The merging and weaving associated with interchanges affect the speed of traffic. Speeds generally decrease with increasing frequency of interchanges. The ideal average interchange spacing over a reasonably long section of freeway (8 to 10 km) is 3 km or greater. The minimum average interchange spacing considered possible over a substantial length of freeway is 1 km.

### Other Factors

The design speed of the primary physical elements of a freeway can affect travel speed. In particular, the horizontal and vertical alignments may contribute to the FFS of a given freeway segment. If a freeway has significant horizontal or vertical conditions, the analyst is encouraged to determine FFS from field observation and field study.

## PASSENGER-CAR EQUIVALENTS

The concept of vehicle equivalents is based on observations of freeway conditions in which the presence of heavy vehicles, including trucks, buses, and recreational vehicles (RVs), creates less than base conditions. The lesser conditions include longer and more frequent gaps of excessive length both in front of and behind heavy vehicles. Also, the speed of vehicles in adjacent lanes and their spacing may be affected by these generally slower-moving large vehicles. Finally, physical space taken up by a large vehicle is typically two to three times greater in terms of length than that taken up by a typical passenger car. To allow the analysis method for freeway capacity to be based on a consistent measure of flow, each heavy vehicle is converted into an equivalent number of passenger cars. The conversion results in a single value for flow rate in terms of passenger cars per hour per lane. The conversion factor used depends on the proportion of heavy vehicles in the traffic stream as well as the length and severity of the upgrade or downgrade.

Illustrations 13-3 and 13-4 show the effect of trucks and other heavy vehicles on freeway traffic.

## DRIVER POPULATION

Studies have noted that noncommuter driver populations do not display the same characteristics as regular commuters. For recreational traffic, capacities have been observed to be as much as 10 to 15 percent lower than for commuter traffic traveling on the same segment, but FFS does not appear to be similarly affected. If the analyst elects to account for this possible effect, locally derived data should be obtained and used in the analysis.



ILLUSTRATION 13-3.



ILLUSTRATION 13-4.

Note the formation of large gaps in front of slow-moving trucks climbing the grade

Even on relatively level terrain, the development of large gaps in front of trucks or other heavy vehicles is common

## LOS

Although speed is a major concern of drivers as related to service quality, freedom to maneuver within the traffic stream and proximity to other vehicles are equally noticeable concerns. These qualities are related to the density of the traffic stream. Unlike speed, density increases as flow increases up to capacity, resulting in a measure of effectiveness that is sensitive to a broad range of flows.

Operating characteristics for the six LOS are shown in Illustrations 13-5 through 13-10. The LOS are defined to represent reasonable ranges in the three critical flow variables: speed, density, and flow rate.

LOS A describes free-flow operations. Free-flow speeds prevail. Vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream. The effects of incidents or point breakdowns are easily absorbed at this level.

LOS B represents reasonably free flow, and free-flow speeds are maintained. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents and point breakdowns are still easily absorbed.

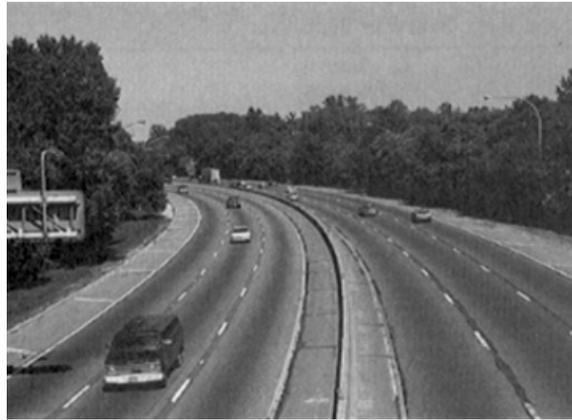


ILLUSTRATION 13-5. LOS A.



ILLUSTRATION 13-6. LOS B.



ILLUSTRATION 13-7. LOS C.



ILLUSTRATION 13-8. LOS D.



ILLUSTRATION 13-9. LOS E.



ILLUSTRATION 13-10. LOS F.

LOS C provides for flow with speeds at or near the FFS of the freeway. Freedom to maneuver within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service will be substantial. Queues may be expected to form behind any significant blockage.

LOS D is the level at which speeds begin to decline slightly with increasing flows and density begins to increase somewhat more quickly. Freedom to maneuver within the traffic stream is more noticeably limited, and the driver experiences reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.

At its highest density value, LOS E describes operation at capacity. Operations at this level are volatile, because there are virtually no usable gaps in the traffic stream. Vehicles are closely spaced leaving little room to maneuver within the traffic stream at speeds that still exceed 80 km/h. Any disruption of the traffic stream, such as vehicles entering from a ramp or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic flow. At capacity, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown with extensive queuing. Maneuverability within the traffic stream is extremely limited, and the level of physical and psychological comfort afforded the driver is poor.

LOS F describes breakdowns in vehicular flow. Such conditions generally exist within queues forming behind breakdown points. Breakdowns occur for a number of reasons:

- Traffic incidents can cause a temporary reduction in the capacity of a short segment, so that the number of vehicles arriving at the point is greater than the number of vehicles that can move through it.
- Points of recurring congestion, such as merge or weaving segments and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles discharged.
- In forecasting situations, the projected peak-hour (or other) flow rate can exceed the estimated capacity of the location.

Note that in all cases, breakdown occurs when the ratio of existing demand to actual capacity or of forecast demand to estimated capacity exceeds 1.00. Operations immediately downstream of such a point, however, are generally at or near capacity, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharged vehicles move away from the bottleneck.

LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point. LOS F is also used to describe conditions at the point of the breakdown or bottleneck and the queue discharge flow that occurs at speeds lower than

the lowest speed for LOS E, as well as the operations within the queue that forms upstream. Whenever LOS F conditions exist, they have the potential to extend upstream for significant distances.

### REQUIRED INPUT DATA AND ESTIMATED VALUES

Exhibit 13-5 gives default values for input parameters in the absence of local data. The analyst should note that taking field measurements for use as inputs to an analysis is the most reliable means of generating parameter values. Only when this is not feasible should default values be considered.

EXHIBIT 13-5. REQUIRED INPUT DATA AND DEFAULT VALUES FOR BASIC FREEWAY SEGMENTS

Required Data	Defaults
Geometric Data	
Number of lanes	-
Lane width	3.6 m
Lateral clearance	3.0 m
Interchange density	-
Specific grade or general terrain	Level
Base free-flow speed	120 km/h rural, 110 km/h urban
Demand Data	
Length of analysis period	15 min
Peak-hour factor	0.88 rural, 0.92 urban
Percentage of heavy vehicles	10% rural, 5% urban
Driver population factor	1.00

### Lane Width and Lateral Clearance

The standard lane width for new freeway construction in the United States is 3.6 m. The standard shoulder width is 3.0 m, but this can be increased to 3.6 m for high-speed highways carrying large numbers of trucks (3, p. 338). These standards may be reduced to accommodate special historical or environmental constraints.

Lane width data are needed only if it is known that the lanes are significantly narrower than 3.6 m. Shoulder widths are significant only if narrower than 1.8 m. Default values of 3.6 m for lane widths and 1.8 m for shoulder widths may be used in the absence of local data unless the analyst is aware of any overriding circumstances (such as mountainous topography, historic structures, or a physical obstruction) that might restrict the facility width.

In the case of minor variations in lane widths or shoulder widths within a segment, the analyst should compute the average of the lane widths and use this average to compute the effects on free-flow speed. Where variations in lane or shoulder widths extend 760 m or more, the segment should be divided to provide segments with consistent physical features.

### Interchange Density

The mean number of interchanges per kilometer is computed for at least a 10-km length of freeway in which the segment is located. The interchange density becomes significant for speed estimation purposes only when the density exceeds 0.5 interchanges per kilometer (an average spacing of 2 km or less).

### Specific Grade or General Terrain

The general terrain type of analysis can be used instead of specific grades wherever no single grade on the segment extends for more than 0.8 km or exceeds 3 percent for

more than 0.4 km. The rate of grade along significant grades can be obtained from geological survey maps.

The maximum extended grade for freeways is usually 6 percent (3, p. 585). If field measurement is not possible and construction plans are not available, extended grades can be approximated on the basis of the analyst's general knowledge of the local terrain. Default values of 2 percent grade for an extended grade on Interstate freeways, 4 percent for an extended grade in rolling terrain, and 6 percent for an extended grade in mountainous terrain may be used in the absence of local data.

### **Base FFS and FFS**

If field measurements are unavailable, FFS can be estimated by applying adjustments to a base free-flow speed (BFFS). The BFFS is 120 km/h for rural freeways and is 100 km/h for urban/suburban freeways. The BFFS is reduced for the effects of lane width, right-side lateral clearance, number of lanes, and density of interchanges.

Analysts should be careful not to assume that the FFS for a freeway is equal to its posted speed limit or the field-measured 85th percentile speed. The FFS is the mean speed measured in the field when volumes are less than 1,300 pc/h/ln.

### **Length of Analysis Period**

The planning, design, and analysis policies and the available resources of an agency will determine the selection of the analysis period(s). The analyst may desire to evaluate the peak hours occurring during the morning commute, at midday, and during the evening commute on a typical weekday or during a peak hour on a Saturday or Sunday if the roadway segment carries a high volume of weekend recreational traffic. Within each hour analyzed, the highest 15-min volume is of primary interest. A peak-hour factor (PHF) is applied to the hourly volume to convert it to a peak 15-min flow rate. A procedure to compute peak-direction, peak-hour demand from an average daily traffic volume is described in Chapter 8.

### **Peak-Hour Factor**

In the absence of field measurements of PHF, approximations can be used. For congested conditions, 0.95 is a reasonable approximation. The PHF tends to be higher for oversaturated conditions and lower for undersaturated conditions. Default values of 0.92 for urban areas and 0.88 for rural areas may be used in the absence of local data.

### **Heavy Vehicles**

The percentage of heavy vehicles in rolling and mountainous terrain should be obtained from locally available data for similar facilities and demand conditions. If the proportion of RVs, trucks, and buses is not known, all the heavy vehicles can be considered to be trucks for the purposes of selecting passenger-car equivalents and computing the heavy-vehicle adjustment factor. Default values of 5 percent heavy vehicles for urban areas and 10 percent heavy vehicles for rural areas may be used in the absence of local data.

### **Driver Population**

The reciprocal of the driver population factor is used to increase the flow rate to account for a driver population not familiar with the freeway facility. The factor should normally be 1.00 but can be reduced to 0.85 for the analysis of weekend conditions in a recreational area.

### **SERVICE VOLUME TABLE**

Exhibit 13-6 may be used to estimate the number of through lanes required to obtain a desired level of service for basic freeway segments under default conditions. The table can be used to test the effect of different interchange densities and is sensitive to the

Do not assume that FFS is equal to the speed limit

different operating characteristics of urban and rural freeways. The example service volumes in the exhibit are highly dependent on the assumptions given in the footnote.

EXHIBIT 13-6. EXAMPLE SERVICE VOLUMES FOR BASIC FREEWAY SEGMENTS  
(SEE FOOTNOTE FOR ASSUMED VALUES)

	Number of Lanes	FFS (km/h)	Service Volumes (veh/h) for LOS				
			A	B	C	D	E
Urban	2	98	1230	1940	2820	3680	4110
	3	101	1900	2980	4340	5570	6200
	4	103	2590	4070	5920	7500	8310
	5	106	3320	5210	7550	9450	10,450
Rural	2	120	1440	2260	3150	3770	4120
	3	120	2160	3400	4720	5660	6180
	4	120	2880	4530	6300	7540	8240
	5	120	3600	5660	7870	9430	10,300

Notes:

Assumptions: Urban: 110-km/h base free-flow speed, 3.6-m-wide lanes, 1.8-m-wide shoulders, level terrain, 5 percent heavy vehicles, no driver population adjustment, 0.92 PHF, 0.63 interchanges per kilometer.

Rural: 120-km/h base free-flow speed, 3.6-m-wide lanes, 1.8-m-wide shoulders, level terrain, 5 percent heavy vehicles, no driver population adjustment, 0.88 PHF, 0.31 interchanges per kilometer.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values given in the footnote.

### III. FREEWAY WEAVING

Weaving is defined as the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices (with the exception of guide signs). Weaving segments are formed when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane. Note that if a one-lane on-ramp is closely followed by a one-lane off-ramp and the two are not connected by an auxiliary lane, the merge and diverge movements are considered separately using procedures for the analysis of ramp terminals.

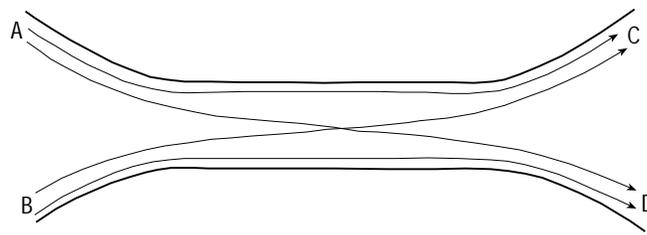
Weaving segments require intense lane-changing maneuvers as drivers must access lanes appropriate to their desired exit points. Thus, traffic in a weaving segment is subject to turbulence in excess of that normally present on basic freeway segments. The turbulence presents special operational problems and design requirements that are addressed by the procedures described in Chapter 24.

Exhibit 13-7 shows a weaving segment. If entry and exit roadways are referred to as legs, vehicles traveling from Leg A to Leg D must cross the path of vehicles traveling from Leg B to Leg C. Flows A-D and B-C are, therefore, referred to as weaving flows. Flows A-C and B-D may also exist, but they need not cross the path of other flows and are referred to as nonweaving flows.

Exhibit 13-7 shows a simple weaving segment formed by a single merge point followed by a single diverge point. Multiple weaving segments may be formed where one merge is followed by two diverge points or where two merge points are followed by one diverge point.

Lane changing is a key descriptor of weaving operation

EXHIBIT 13-7. FORMATION OF A WEAVING SEGMENT



Three key geometric features affect weaving operation:

- Configuration
- Length
- Width

Weaving segments may exist on any type of facility: freeways, multilane highways, two-lane highways, interchange areas, urban streets, or collector-distributor roadways. Whereas the methodology of Chapter 24 was developed for freeways, guidance is given on adapting the procedure to weaving segments on multilane highways. No guidance is given for analysis of weaving on urban streets, which is considerably more complex and involves signalization issues. At present, there are no generally accepted procedures for the analysis of weaving on urban streets.

Three geometric variables influence weaving segment operations: configuration, length, and width. These variables are discussed in the following sections.

### WEAVING CONFIGURATIONS

The most critical aspect of operations within a weaving segment is lane changing. Weaving vehicles, which must cross a roadway to enter on the right and leave on the left, or vice versa, accomplish these maneuvers by making the appropriate lane changes. The configuration of the weaving segment (i.e., the relative placement of entry and exit lanes) has a major effect on the number of lane changes required of weaving vehicles to successfully complete their maneuver. There is also a distinction between lane changes that must be made to weave successfully and additional lane changes that are discretionary (i.e., are not necessary to complete the weaving maneuver). The former must take place within the confined length of the weaving segment, whereas the latter are not restricted to the weaving segment itself.

The methodology of Chapter 24 identifies three major categories of weaving configurations: Type A, Type B, and Type C. Each has unique characteristics that are described below.

#### Type A Weaving Configuration

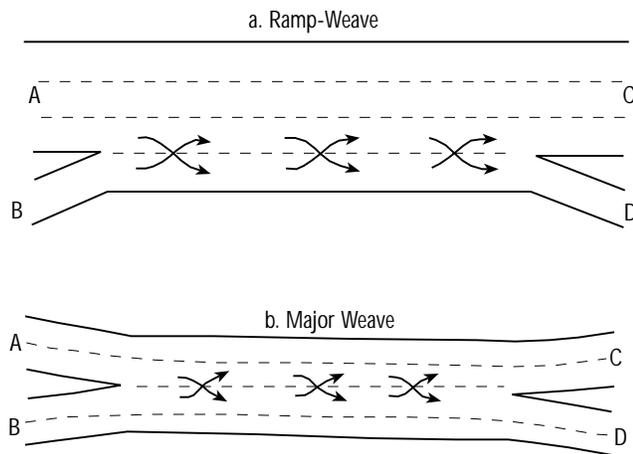
Exhibit 13-8 illustrates two subcategories of Type A weaving segments. The identifying characteristic of a Type A weaving segment is that all weaving vehicles must make one lane change to complete their maneuver successfully. All of these lane changes occur across a lane line that connects from the entrance gore area directly to the exit gore area. Such a line is referred to as a crown line. Type A weaving segments are the only such segments to have a crown line.

The most common form of Type A weaving segment is shown in Exhibit 13-8(a). The segment is formed by a one-lane on-ramp followed by a one-lane off-ramp, with the two connected by a continuous auxiliary lane. The lane line between the auxiliary lane and the right-hand freeway lane is the crown line for the weaving segment. All on-ramp vehicles entering the freeway must make a lane change from the auxiliary lane to the shoulder lane of the freeway. All freeway vehicles exiting at the off-ramp must make a lane change from the shoulder lane of the freeway to the auxiliary lane. This type of configuration is also referred to as a ramp-weave.

Exhibit 13-8(b) illustrates a major weaving segment that also has a crown line. A major weaving segment is formed when three or four of the entry and exit legs have multiple lanes. As in the case of a ramp-weave, all weaving vehicles, regardless of the direction of the weave, must execute one lane change across the crown line of the segment.

Type A: all weaving vehicles must make one lane change

EXHIBIT 13-8. TYPE A WEAVING SEGMENTS



The two segments differ primarily in the effect of ramp geometrics on speed. For most ramp-weave segments, the design speed of the ramps is significantly lower than the design speed of the freeway. Thus, on- and off-ramp vehicles must accelerate or decelerate as they traverse the weaving segment. For major weaving segments, entry and exit legs often have design speeds that are similar to that of the mainline freeway, and such acceleration and deceleration are not required. It should be noted that the methodology of Chapter 24 was calibrated for ramp-weave configurations.

Because all weaving vehicles in a Type A configuration must execute a lane change across the crown line, weaving vehicles are generally confined to occupying the two lanes adjacent to the crown line. Some nonweaving vehicles will share these lanes. This will essentially limit the number of lanes that weaving vehicles can occupy.

### Type B Weaving Configuration

Type B weaving segments are shown in Exhibit 13-9. All Type B weaving segments fall into the general category of major weaving segments in that such segments always have at least three entry and exit legs with multiple lanes (except for some collector-distributor configurations).

Once again, it is the lane changing required of weaving vehicles that characterizes the Type B configuration:

- One weaving movement can be made without making any lane changes, and
- The other weaving movement requires at most one lane change.

Exhibits 13-9(a) and 13-9(b) show two Type B weaving segments. In both cases, Movement B-C (entry on the right, departure on the left) may be made without executing any lane changes, whereas Movement A-D (entry on the left, departure on the right) requires only one lane change. Essentially, there is a continuous lane that allows for entry on the right and departure on the left. In Exhibit 13-9(a), this is accomplished by providing a diverging lane at the exit gore. From this lane, a vehicle may proceed down either exit leg without executing a lane change. This type of design is also referred to as lane balanced, that is, the number of lanes leaving the diverge is one more than the number of lanes approaching it.

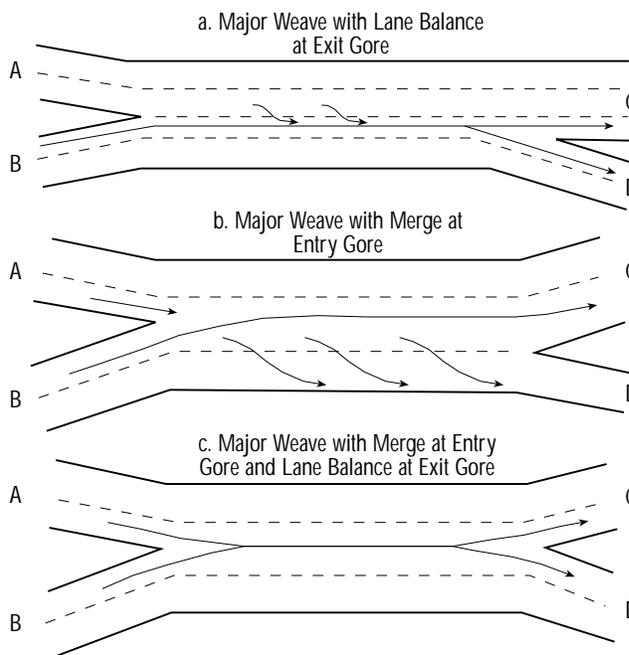
In Exhibit 13-9(b), the same lane-changing scenario is provided by having a lane from Leg A merge with a lane from Leg B at the entrance gore. This is slightly less efficient than providing lane balance at the exit gore but produces similar numbers of lane changes by weaving vehicles.

The analysis of a major weave in the HCM is an approximation

Lane-changing characteristics for Type B

Lane balance defined

EXHIBIT 13-9. TYPE B WEAVING SEGMENTS



The configuration shown in Exhibit 13-9(c) is unique, having both a merge of two lanes at the entrance gore and lane balance at the exit gore. In this case, both weaving movements can take place without making a lane change. Such configurations are most often found on collector-distributor roadways as part of an interchange.

Type B weaving segments are extremely efficient in carrying large weaving flows, primarily because of the provision of a through lane for at least one of the weaving movements. Weaving movements can also be made with a single lane change from either of the lanes adjacent to the through lane. Thus, weaving vehicles can occupy a substantial number of lanes in the weaving segment and are not as restricted as in Type A segments.

**Type C Weaving Configuration**

Type C weaving segments are similar to those of Type B in that one or more through lanes are provided for one of the weaving movements. The distinguishing characteristic of a Type C weaving segment is that the other weaving movement requires a minimum of two lane changes for successful completion of a weaving maneuver. Thus, a Type C weaving segment is characterized by the following:

- One weaving movement may be made without making a lane change, and
- The other weaving movement requires two or more lane changes.

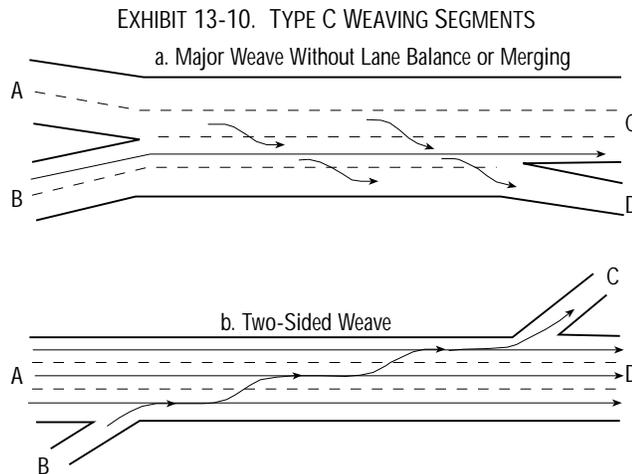
Exhibit 13-10 shows two types of Type C weaving segments. In Exhibit 13-10(a), Movement B-C does not require a lane change, whereas Movement A-D requires two lane changes. This type of segment is formed when there is neither merging of lanes at the entrance gore nor lane balance at the exit gore, and no crown line exists. Although such a segment is relatively efficient for weaving movements in the direction of the freeway flow, it cannot efficiently handle large weaving flows in the other direction.

Exhibit 13-10(b) shows a two-sided weaving segment. It is formed when a right-hand on-ramp is followed by a left-hand off-ramp, or vice versa. In such cases, the through freeway flow operates functionally as a weaving flow. Ramp-to-ramp vehicles must cross all lanes of the freeway to execute their desired maneuver. Freeway lanes are, in effect, through weaving lanes, and ramp-to-ramp vehicles must make multiple lane changes as they cross from one side of the freeway to the other. Although it is technically

Type C: one weaving maneuver requires at least two lane changes

Analysis of two-sided weaving in the HCM is only a rough approximation

a Type C configuration, there is little information concerning the operation of such segments. The methodology of Chapter 24 was calibrated for the type of segment in Exhibit 13-10(a) and provides only the roughest of approximations when applied to two-sided weaving segments.



### Effects of Weaving Configuration

The configuration of the weaving segment has a marked effect on operations because of its influence on lane-changing behavior. A weaving segment with 1,000 veh/h weaving across 1,000 veh/h in the other direction requires at least 2,000 lane changes per hour in a Type A segment, since each vehicle makes one lane change. In a Type B segment, only one movement must change lanes, reducing the number of required lane changes per hour to 1,000. In a Type C segment, one weaving flow would not have to change lanes, while the other would have to make at least two lane changes, for a total of 2,000 lane changes per hour.

Because of this, the models and algorithms of Chapter 24 are keyed to the type of configuration, with parameters that depend specifically on configuration. Thus, for a given number of lanes and length of segment, models will predict different operating characteristics for different configurations.

Configuration has a further effect on the proportional use of lanes by weaving and nonweaving vehicles. Since weaving vehicles must occupy specific lanes to efficiently complete their maneuvers, the configuration can limit the ability of weaving vehicles to use outer lanes of the segment. This effect is most pronounced for Type A segments, because weaving vehicles must primarily occupy the two lanes adjacent to the crown line. It is least severe for Type B segments, since these segments require the fewest lane changes for weaving vehicles, thus allowing more flexibility in lane use.

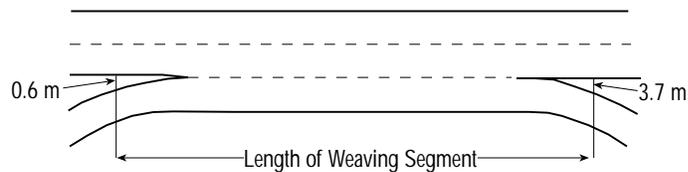
### WEAVING LENGTH

Because weaving vehicles must execute all the required lane changes for their maneuver within the weaving segment boundary from the entry gore to the exit gore, the parameter of weaving length is important. The length of the weaving segment constrains the time and space in which the driver must make all required lane changes. Thus, as the length of a weaving segment decreases (configuration and weaving flow being constant), the intensity of lane changing, and the resulting turbulence, increases.

The measurement of weaving length is shown in Exhibit 13-11. Length is measured from a point at the merge gore where the right edge of the freeway shoulder lane and the left edge of the merging lane(s) are 0.6 m apart to a point at the diverge gore where the two edges are 3.7 m apart.

Configuration affects use of lanes

EXHIBIT 13-11. MEASURING THE LENGTH OF A WEAVING SEGMENT



Segments may become so long that the procedure no longer applies

Procedures in Chapter 24 generally apply to weaving segments up to 750 m long. Weaving may exist in longer segments, but merging and diverging movements are often separated, with lane changing tending to concentrate near merge and diverge gore areas. Weaving turbulence may exist to some degree throughout longer segments, but operations are approximately the same as those for a basic freeway segment, except for the ramp influence areas near the entry and exit gore areas.

### WEAVING WIDTH

The third geometric variable influencing the operation of the weaving segment is its width, which is defined as the total number of lanes between the entry and exit gore areas, including the auxiliary lane, if present. As the number of lanes increases, the throughput capacity increases. At the same time, the opportunity for lane changing also increases for discretionary lane changes that may take place within the weaving segment.

### TYPE OF OPERATION

Whereas the total number of lanes in the weaving segment is important, the proportional use of those lanes by weaving and nonweaving vehicles is even more important. Under normal circumstances, weaving and nonweaving vehicles compete for space, and operations across all lanes tend to reach an equilibrium in which all drivers experience similar conditions. In a weaving segment, there is some segregation of weaving and nonweaving flows as nonweaving vehicles tend to stay in outside lanes and weaving vehicles tend to occupy the lanes involved in crossing the roadway. Nevertheless, there is substantial sharing of lanes by weaving and nonweaving vehicles.

Under normal circumstances, weaving and nonweaving vehicles will reach equilibrium operation in which weaving vehicles effectively occupy  $N_w$  lanes of the segment, with nonweaving vehicles occupying the remaining lanes.

In a very real sense, however, the lane configuration limits the total number of lanes that can be used by weaving vehicles because of the lane changes that must be made. The following statements describe this effect.

- Weaving vehicles may occupy all of a lane in which weaving is accomplished without a lane change.
- Weaving vehicles may occupy most of a lane from which a weaving maneuver can be accomplished with a single lane change.
- Weaving vehicles may occupy a small portion of a lane from which a weaving maneuver can be completed by making two lane changes.
- Weaving vehicles cannot occupy a measurable portion of any lane from which a weaving maneuver would require three or more lane changes.

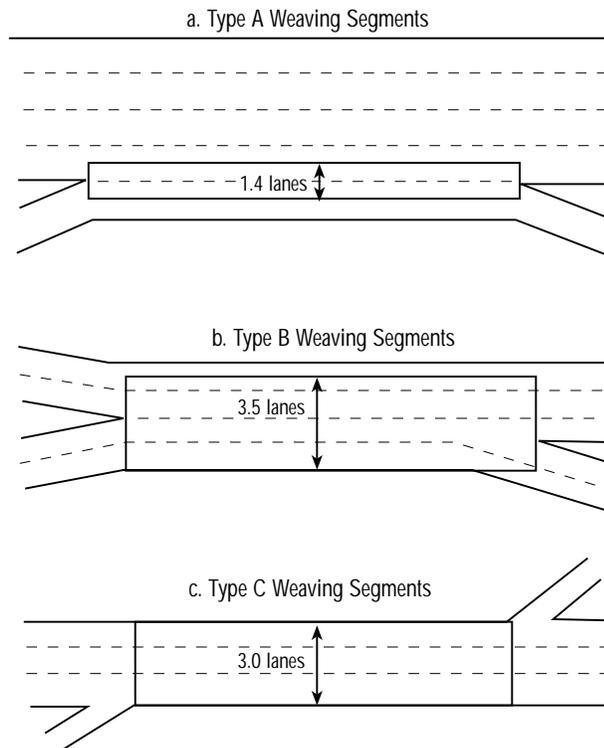
This translates into limitations on the maximum number of lanes that weaving vehicles can occupy based on the configuration of the segment, as shown in Exhibit 13-12.

In a typical Type A configuration, almost all ramp vehicles are weaving (i.e., there is little ramp-to-ramp flow). Thus, the auxiliary lane is almost fully occupied by weaving vehicles. However, the shoulder lane of the freeway is shared by weaving and nonweaving vehicles. Studies have shown that weaving vehicles rarely occupy more than 1.4 lanes of a Type A configuration.

The relative use of lanes by weaving and nonweaving vehicles is an important consideration

Guidelines on lane use characteristics

## EXHIBIT 13-12. MAXIMUM USE OF LANES BY WEAVING VEHICLES



Type B configurations are far more flexible. There is always one through lane for weaving vehicles that can be fully occupied by those vehicles. In addition, the two lanes adjacent to the through lane can also be substantially used by weaving vehicles. There can be some usage of the next adjacent lanes as well. Studies have shown that weaving vehicles can occupy up to 3.5 lanes in a Type B configuration.

Type C configurations are somewhat more restrictive than Type B configurations, particularly for the movement requiring two or more lane changes. Weaving vehicles can still occupy all of the through lane and substantial portions of the lanes adjacent to the through lane. Partial use of other lanes, however, is usually quite restricted. Studies indicate that the practical limit on lane usage by weaving vehicles in a Type C configuration is 3.0.

In this discussion, two important parameters have been defined:

$N_w$  = number of lanes weaving vehicles must occupy to achieve equilibrium operation with nonweaving vehicles, and

$N_w(max)$  = maximum number of lanes that can be occupied by weaving vehicles, based on geometric configuration.

The methodology of Chapter 24 includes models for determining values of  $N_w$ , whereas values of  $N_w(max)$  have been specified herein. The comparison of the two values determines the type of operation that is present in the weaving segment.

Where  $N_w \leq N_w(max)$ , equilibrium operation will be established. This is referred to as unconstrained operation, because there are no constraints preventing the equilibrium from occurring. Where  $N_w > N_w(max)$ , weaving vehicles can only occupy  $N_w(max)$  lanes. Thus, they will occupy less space than is needed to establish equilibrium, while nonweaving vehicles occupy more space than normal. Operations for weaving vehicles become worse, while those for nonweaving vehicles get better. This is referred to as constrained operation, because the configuration constrains weaving vehicles from establishing equilibrium with nonweaving vehicles.

Constrained and  
unconstrained operation

Under unconstrained operation, weaving and nonweaving vehicles usually experience similar operational characteristics. In constrained operation, weaving vehicles often experience operating conditions that are markedly worse than those of nonweaving vehicles in the same segment. Thus, determining the type of operation is a key step in the Chapter 24 analysis methodology.

**SERVICE VOLUME TABLE**

The Chapter 24 methodology does not readily produce service volumes. The procedure is set up to determine LOS, with flows and geometrics being fully specified. Nevertheless, service volumes can be produced by trial-and-error computations of volume levels that result in threshold densities for the various LOS.

Service volumes depend on the type of configuration, the length of the segment, the volume ratio (the proportion of total flow that is weaving), the number of lanes in the segment, and the FFS of the freeway. Exhibit 13-13 gives example service volumes for weaving segments.

EXHIBIT 13-13. EXAMPLE SERVICE VOLUMES FOR FREEWAY WEAVING SEGMENTS  
(SEE FOOTNOTE FOR ASSUMED VALUES)

Weaving Section Number of Lanes	Service Volumes (veh/h) for LOS				
	A	B	C	D	E
Type A					
3	1710	3010	3920	4740	5490
4	2280	4010	5230	6320	7330
5	2850	5010	6540	7930	9260
Type B					
3	1780	3290	4430	5490	6320
4	2380	4390	5900	7320	8430
5	2970	5490	7380	9140	10,540
Type C					
3	1790	3280	4380	5400	6320
4	2380	4370	5840	7200	8430

Note:  
Assumptions: FFS = 120 km/h, PHF = 0.90, 5 percent trucks, level terrain, volume ratio = 0.20, weaving segment length = 300 m.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values given in the footnote.

**IV. RAMPS AND RAMP JUNCTIONS**

A ramp is a length of roadway providing an exclusive connection between two highway facilities. On freeways, all entering and exiting maneuvers take place on ramps that are designed to facilitate smooth merging of on-ramp vehicles into the freeway traffic stream and smooth diverging of off-ramp vehicles from the freeway traffic stream onto the ramp. Computational procedures for the analysis of ramps are contained in Chapter 25 of this manual.

**RAMP COMPONENTS**

A ramp may consist of three geometric elements of interest: the ramp-freeway junction, the ramp roadway, and the ramp-street junction. A ramp-freeway junction is typically designed to permit high-speed merging or diverging with minimum disruption to the adjacent freeway traffic. The geometric characteristics of ramp-freeway junctions vary. The length and type (taper, parallel) of acceleration or deceleration lanes, FFS of

The focus of this section is on the ramp-freeway junction

the ramp in the immediate vicinity of the junction, sight distances, and other elements all influence ramp operations.

Geometric characteristics of ramp roadways vary from location to location. Ramps may vary in terms of number of lanes (usually one or two), design speed, grades, and horizontal curvature. The design of ramp roadways is seldom a source of operational difficulty unless a traffic incident causes disruption along their length. Ramp-street terminal problems can cause queuing along the length of a ramp, but this is generally not related to the design of the ramp roadway.

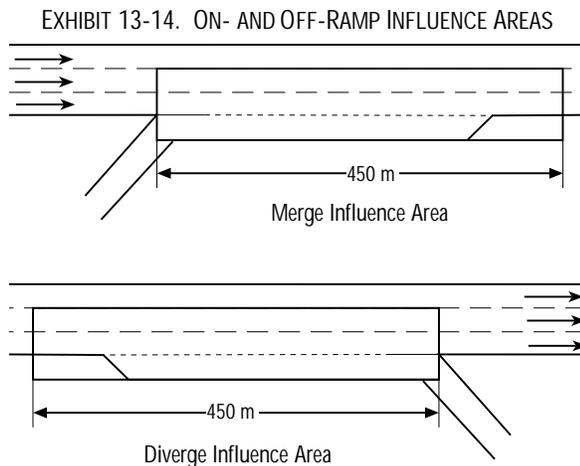
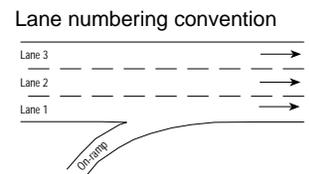
Freeway-to-freeway ramps have two ramp-freeway terminals and do not have a ramp-street terminal. However, many ramps connect limited-access facilities to local arterials and collectors. For such ramps, the ramp-street terminal is often a critical element in the overall design. Ramp-street junctions can permit uncontrolled merging and diverging movements, or they can take the form of an at-grade intersection. Queues forming at a ramp-street junction can, under extreme conditions, back up into the ramp-freeway junction and indeed onto the freeway mainline itself.

### OPERATIONAL CHARACTERISTICS

A ramp-freeway junction is an area of competing traffic demands for space. Upstream freeway traffic competes for space with entering on-ramp vehicles in merge areas. On-ramp demand is usually generated locally, although urban streets may bring some drivers to the ramp from more distant origins.

In a merge area, individual on-ramp vehicles attempt to find gaps in the adjacent freeway lane traffic stream. Because most ramps are on the right side of the freeway, the freeway lane in which on-ramp vehicles seek gaps is designated as Lane 1 in this manual. By convention, freeway lanes are numbered from 1 to N, from the right shoulder to the median.

The action of individual merging vehicles entering the Lane 1 traffic stream creates turbulence in the vicinity of the ramp. Approaching freeway vehicles move toward the left to avoid this turbulence. Studies (4) have shown that the operational effect of merging vehicles is heaviest in Lanes 1 and 2 and the acceleration lane for a distance extending from the physical merge point to 450 m downstream. Exhibit 13-14 shows this influence area for on-ramp and off-ramp junctions.



Interactions are dynamic in ramp influence areas. Approaching freeway vehicles will move left as long as there is capacity to do so. Whereas the intensity of ramp flow influences the behavior of freeway vehicles, general freeway congestion can also act to limit ramp flow, causing diversion to other interchanges or routes.

At off-ramps, the basic maneuver is a diverge, that is, a single traffic stream separating into two streams. Exiting vehicles must occupy the lane adjacent to the off-ramp (Lane 1 for a right-hand off-ramp). Thus, as the off-ramp is approached, diverge vehicles move right. This effects a redistribution of other freeway vehicles, as they move left to avoid the turbulence of the immediate diverge area. Studies (4) show that the area of greatest turbulence is the deceleration lane plus Lanes 1 and 2 for a distance extending 450 m upstream from the physical diverge point, as shown in Exhibit 13-14.

**IMPORTANT PARAMETERS**

A number of variables influence the operation of ramp-freeway junctions. They include all of the variables affecting basic freeway segment operation: lane widths, lateral clearances, terrain, driver population, and the presence of heavy vehicles. There are additional parameters of particular importance to the operation of ramp-freeway junctions, including length of acceleration/deceleration lane, ramp free-flow speed, and lane distribution of upstream traffic.

Length of acceleration and deceleration lanes

The length of the acceleration or deceleration lane has a significant effect on merging and diverging operations. Short lanes provide on-ramp vehicles with restricted opportunity to accelerate before merging and off-ramp vehicles with little opportunity to decelerate off-line. The result is that most acceleration and deceleration must take place on the mainline, which disrupts through vehicles. Short acceleration lanes also force many vehicles to slow significantly and even stop while seeking an appropriate gap in the Lane 1 traffic stream.

Ramp FFS

Many characteristics influence the free-flow speed of the ramp, including degree of curvature, number of lanes, grades, and sight distances, among others. FFS is an influential factor, since it determines the speed at which merging vehicles enter the acceleration lane and the speed at which diverging vehicles must enter the ramp. This, in turn, determines the amount of acceleration or deceleration that must take place. Ramp FFS generally vary between 30 and 80 km/h. Although FFS is best determined in the field, a default value of 55 km/h may be used where specific measurements or predictions are unavailable.

Upstream mainline lane distribution

Several factors influence the lane distribution of traffic immediately upstream of an on- or off-ramp: number of lanes on the facility, proximity of adjacent upstream and downstream ramps, and the activity on those ramps. As conditions force more approaching freeway flow into Lanes 1 and 2, merging and diverging maneuvers become more difficult. Therefore, estimation of the upstream freeway flow approaching in Lanes 1 and 2 of the freeway (which are the freeway lanes included in the merge and diverge influence areas) is important.

**CAPACITY OF MERGE AND DIVERGE AREAS**

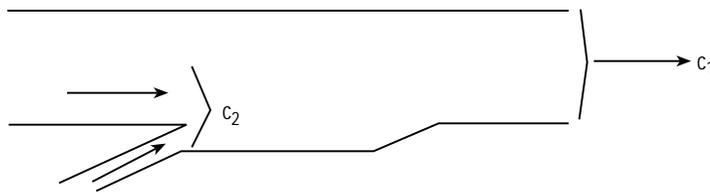
Downstream capacity is not influenced by turbulence at an upstream merge

There is no evidence that merging or diverging maneuvers restrict the total capacity of the upstream or downstream basic freeway segments. Their influence is primarily to add or subtract demand at the ramp-freeway junction. Thus, the capacity of a downstream basic freeway segment is not influenced by turbulence in a merge area. The capacity will be the same as if the segment were a basic freeway segment. As on-ramp vehicles enter the freeway at a merge area, the total number of ramp and approaching freeway vehicles that can be accommodated is the capacity of the downstream basic freeway segment, as shown in Exhibit 13-15.

Upstream capacity is not influenced by turbulence at a downstream diverge

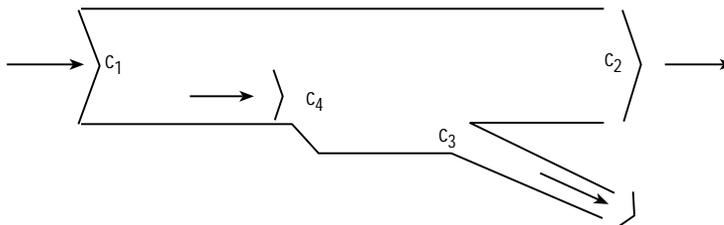
Similarly, the capacity of an upstream basic freeway segment is not influenced by the turbulence in a diverge area. The total capacity that may be handled by the diverge junction is limited either by the capacity of the approaching (upstream) basic freeway segment or by the capacity of the downstream basic freeway segment and the ramp itself, as shown in Exhibit 13-16. Most breakdowns at diverge areas occur because the capacity of the exiting ramp is insufficient to handle the ramp demand flow. This results in queuing that backs up into the freeway mainline.

EXHIBIT 13-15. CAPACITY OF MERGE AREAS



$c_1$  = capacity of merge area, controlled by the capacity of the downstream basic freeway segment.  
 $c_2$  = maximum flow into the merge influence area (4,600 pc/h).

EXHIBIT 13-16. CAPACITY OF DIVERGE AREAS



Total diverge capacity cannot be more than the upstream basic freeway capacity ( $c_1$ ) or the total downstream capacity of the basic freeway ( $c_2$ ) plus the ramp ( $c_3$ ).  
 $c_4$  = maximum freeway flow in Lanes 1 and 2 that may enter the diverge influence area (4,400 pc/h).

Another capacity value that affects ramp-freeway junction operation is an effective maximum number of freeway vehicles that can enter the ramp junction influence area without causing local congestion and local queuing. For on-ramps, the total entering flow in Lanes 1 and 2 of the freeway plus the on-ramp flow cannot exceed 4,600 pc/h. For off-ramps, the total entering flow in Lanes 1 and 2 of the freeway (which includes the off-ramp flow) cannot exceed 4,400 pc/h. Demands exceeding these values will cause local congestion and queuing. However, as long as demand does not exceed the capacity of the upstream or downstream freeway sections or the off-ramp, breakdown will normally not occur. Thus, this condition is not labeled as LOS F, but rather at an appropriate LOS based on density in the section.

If local congestion occurs because too many vehicles try to enter the merge ( $c_2$  in Exhibit 13-15) or diverge ( $c_4$  in Exhibit 13-16) influence area, the capacity of the merge or diverge area is unaffected. In such cases, more vehicles move to outer lanes (if they are available), and the lane distribution predicted by the methodology of Chapter 25 is approximate.

### LOS

Levels of service in merge and diverge influence areas are defined in terms of density for all cases of stable operation, LOS A through E. LOS F exists when the demand exceeds the capacity of upstream or downstream freeway sections or the capacity of an off-ramp.

LOS A represents unrestricted operations. Density is low enough to permit smooth merging and diverging, with virtually no turbulence in the traffic stream. At LOS B, merging and diverging maneuvers become noticeable to through drivers, and minimal turbulence occurs. Merging drivers must adjust speeds to accomplish smooth transitions from the acceleration lane to the freeway. At LOS C, speed within the influence area begins to decline as turbulence levels become noticeable. Both ramp and freeway vehicles begin to adjust their speeds to accomplish smooth transitions. At LOS D, turbulence levels in the influence area become intrusive, and virtually all vehicles slow to

The HCM allows analysis of the capacity of the ramp influence area

accommodate merging and diverging. Some ramp queues may form at heavily used on-ramps, but freeway operation remains stable. LOS E represents conditions approaching capacity. Speeds reduce significantly, and turbulence is felt by virtually all drivers. Flow levels approach capacity, and small changes in demand or disruptions within the traffic stream can cause both ramp and freeway queues to form.

**REQUIRED INPUT DATA AND ESTIMATED VALUES**

Exhibit 13-17 gives default values for input parameters in the absence of local data. The analyst should note that taking field measurements for use as inputs to an analysis is the most reliable means of generating parameter values. Only when this is not feasible should default values be considered.

EXHIBIT 13-17. REQUIRED INPUT DATA AND DEFAULT VALUES

Item	Default
Geometric Data	
Ramp lanes	-
Acceleration lane length	180 m
Deceleration lane length	42 m
Ramp free-flow speed	55 km/h
Demand Data	
Demand volume	-
PHF	0.88 rural, 0.92 urban
Percentage of heavy vehicles	10% rural, 5% urban
Driver population factor	1.0

**Ramp Lanes**

The analyst should assume single-lane ramps unless there is an indication of particularly heavy ramp demand. Ramp demands in excess of 1,500 veh/h generally warrant a second lane (3, p. 87). A metered on-ramp may have two approach lanes to accommodate demand levels that could otherwise be accommodated by a single lane. One lane may be a high-occupancy vehicle (HOV) bypass lane.

**Length of Acceleration/Deceleration Lane**

The typical length of acceleration and deceleration lanes for ramps should be obtained from the design standards used by the highway operating agency. The length of the acceleration or deceleration lane is measured from the intersection of the edge of the travel way for the freeway and the ramp (Point A) and the downstream intersection of the freeway and ramp edges of the travel way (Point B). These features are shown in Exhibits 13-18 and 13-19. In the absence of design information or field measurements, a default value of 180 m may be used for the length of the acceleration lane, and a default value of 42 m may be used for the length of the deceleration lane.

**Ramp FFS**

Ramp free-flow speeds usually range between 30 and 80 km/h depending on the grade, alignment, and control. In the absence of field-observed or locally developed values, 55 km/h may be assumed.

**Length of Analysis Period**

Refer to basic freeway segment description of length of analysis period under required input data and estimated values.

EXHIBIT 13-18. ACCELERATION LANE LENGTH DIAGRAM

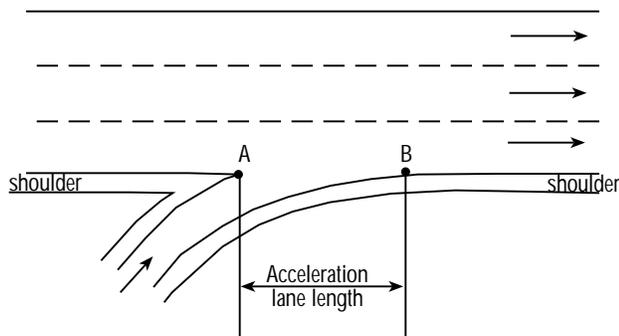
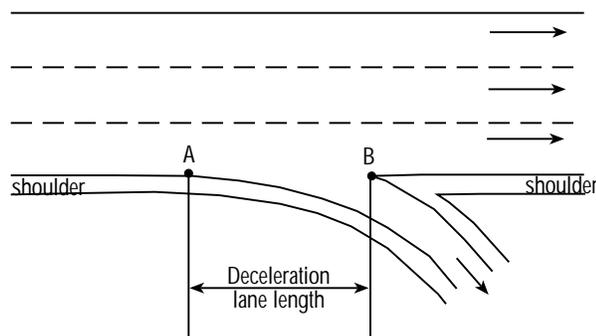


EXHIBIT 13-19. DECELERATION LANE LENGTH DIAGRAM

**PHF**

Refer to basic freeway segment description of peak-hour factor under required input data and estimated values.

**Percentage of Heavy Vehicles**

Refer to basic freeway segment description of percentage of heavy vehicles under required input data and estimated values.

**Driver Population Factor**

Refer to basic freeway segment description of driver population factor under required input data and estimated values.

**SERVICE VOLUME TABLE**

Service volumes for ramps are difficult to describe because of the number of variables that affect operations. Exhibit 13-20 gives example service volumes of a single lane on-ramp and off-ramp under a set of assumptions described in the footnote of the exhibit.

Service volumes for LOS A through D are based on conditions producing the limiting densities for these LOS. Service volumes for LOS E are based on the minimum of three limiting criteria: the capacity of the freeway, the maximum volume that can enter the ramp influence area, and the capacity of the ramp. In some cases, capacity constraints are more severe than density constraints. In such cases, some levels of service may not exist in practical terms for combinations of ramp and freeway volumes.

This table contains approximate values. It is meant for illustrative purposes only. The values are highly dependent on the assumptions used. It should not be used for operational analyses or final design. This table was derived using assumed values given in the footnote.

EXHIBIT 13-20. EXAMPLE SERVICE VOLUMES FOR SINGLE-LANE ON- AND OFF-RAMPS  
(SEE FOOTNOTE FOR ASSUMED VALUES)

Mainline Number of Lanes	Service Volumes (veh/h) for LOS				
	A	B	C	D	E
On-Ramp					
2	N/A	290	1250	1760	1760
3	5	1660	1760	1760	1760
4	650	1760	1760	1760	1760
Off-Ramp					
2	N/A	N/A	530	1360	1760
3	N/A	350	1340	1760	1760
4	N/A	830	1660	1760	1760

Notes:  
Assumptions: mainline FFS = 120 km/h, mainline volume = 2,000 veh/h/ln, ramp FFS = 55 km/h, acceleration lane length = 300 m, deceleration lane length = 50 m, level terrain for both ramp and freeway, 5 percent trucks, PHF = 0.90, lane width = 3.6 m, and standard shoulder widths.

Basic freeway segments are outside the influence of ramps or weaving  
Refer to Chapter 22 for a more detailed discussion of freeway facilities

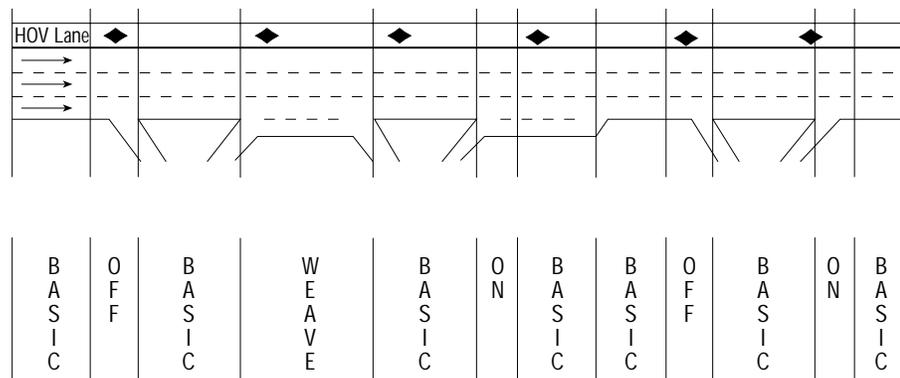
HOV facility with two or more lanes and limited access can be analyzed as a basic freeway segment

### V. FREEWAY FACILITIES

A freeway facility is composed of three types of segments. Weaving segments are segments of the freeway where two or more vehicle flows must cross each other's path. They are usually formed when merge areas are followed by diverge areas. They are also formed when an on-ramp is followed by an off-ramp and the two are connected by an auxiliary lane. Ramp junctions are points at which on- and off-ramps join the freeway. The junction formed at this point is an area of turbulence due to concentrations of merging or diverging vehicles. Basic freeway segments are outside the influence area of ramps or weaving segments of the freeway.

HOV lanes are adjacent to general freeway lanes and are designated for use by buses and vehicles with two or more persons. If an HOV facility has two or more lanes in each direction all or part of the day and if access to the HOV facility is limited from adjacent freeway lanes (e.g., 1.6 km or greater access point spacing), these procedures may be used. Otherwise, HOV lane(s) will have lower lane capacities. Exhibit 13-21 shows segments of an extended freeway facility (5).

EXHIBIT 13-21. FREEWAY FACILITY SEGMENTS



**TRAFFIC MANAGEMENT STRATEGIES**

Freeway traffic management is the implementation of strategies to improve freeway performance, especially when the number of vehicles desiring to use a portion of the freeway at a particular time exceeds its capacity. There are two approaches to improving system operation. Supply management strategies work on improving the efficiency and effectiveness of the existing freeway or adding additional freeway capacity. Demand management strategies work on controlling, reducing, eliminating, or changing the time of travel of vehicle trips on the freeway while providing a wider variety of mobility options to those who wish to travel. However, in actual application, some strategies may address both sides of the supply/demand equation. The important point is that there are two basic ways to improve system performance.

Demand management

Supply management strategies are intended to increase capacity. Capacity may be increased by building new pavement or by managing existing pavement. Supply management has been the traditional form of freeway system management for many years. Increasingly, the focus is turning to demand management as a tool to address freeway problems. Demand management programs include alternatives to reduce freeway vehicle demand by increasing the number of persons in a vehicle, diverting traffic to alternate routes, influencing the time of travel, or reducing the need to travel. Demand management programs must rely on incentives or disincentives to make these shifts in behavior attractive.

Supply management and demand management

Freeway traffic demand management strategies include the use of priority for high-occupancy vehicles, congestion pricing, and traveler information systems. Some alternative strategies such as ramp metering may restrict demand and possibly increase the existing capacity. In some cases, spot capacity improvements such as the addition of auxiliary lanes or minor geometric improvements may be implemented to better utilize overall freeway system capacity. In the remainder of this section the process of evaluating freeway management strategies and the most common freeway traffic management techniques will be presented. The freeway traffic management process is used to assess the effect on freeway performance that these strategies might produce.

**Freeway Traffic Management Process**

Freeway traffic management is the application of strategies that are intended to reduce the traffic using the facility or increase the capacity of the facility. Person demand can be shifted in time or space, vehicle demand can be reduced by a shift in mode, or total demand can be reduced by a variety of factors. Factors affecting total demand include changes in land use and elimination of trips due to telecommuting, reduced workweek, or a decision to forgo travel. By shifts of demand in time (e.g., leaving earlier), shifts of demand in space (e.g., taking an alternative route), shifts in mode, or changes in total demand, traffic on a freeway segment can be reduced. Likewise, if freeway capacity has been reduced (e.g., as the result of an incident that has closed a lane or adverse weather conditions), improved traffic management can return the freeway to normal capacity sooner, reducing the total delay to travelers.

Basic approach to evaluating management strategies

The basic approach used to evaluate traffic management is to compare alternative strategies. The base case would be operation of the facility without any freeway traffic management. The alternative case would be operation of the facility with the freeway traffic management strategy or strategies being evaluated. The alternative case could have different demands and capacities based on the conditions being evaluated. The evaluations could also be made for existing or future traffic demands. Combinations of strategies are also possible, but some combinations may be difficult to evaluate because of limited quantifiable data.

**Freeway Management Strategies**

Freeway traffic management strategies are implemented to make the most effective and efficient use of the freeway system. Activities that reduce capacity include incidents

(including traffic accidents, disabled or stalled vehicles, spilled cargo, emergency or unscheduled maintenance, traffic diversions, or adverse weather), construction activities, scheduled maintenance activities, and major emergencies (such as earthquakes or flooding). Activities that increase demand include special events. Freeway traffic management strategies that mitigate capacity reductions include incident management; traffic control plans for construction, maintenance activities, special events, and emergencies; and minor design improvements (e.g., auxiliary lanes, emergency pullouts, and accident investigation sites). Freeway traffic management strategies to reduce demand include plans for incidents, special events, construction, and maintenance activities; entry control/ramp metering; on-freeway HOV lanes; HOV bypass lanes on ramps; traveler information systems; and road pricing.

### Capacity Management Strategies

Incident management

Incident management is the most significant freeway strategy generally used by operating agencies. Incidents can cause significant delays even on facilities that do not routinely experience congestion. It is generally believed that more than 50 percent of freeway congestion is the result of incidents. Strategies to mitigate the effects of incidents include early detection and quick response with the appropriate resources. During an incident, effective deployment of management resources can result in a significant reduction in the effects of the incident. Proper application of traffic control devices, including signage and channelization, is part of effective incident management. Quick removal of vehicles and debris is another part. Incident management may also include the use of accident investigation sites on conventional streets near freeways for follow-up activities.

Lane control signals

Lane control signals are a way to convey the status of individual freeway lanes to motorists. By providing positive guidance through incident sites and maintenance activities, the capacity of the location, as well as safety, may be maintained.

Geometrics

Geometric adjustments can be made to a freeway to enhance overall freeway capacity. Examples include the use of auxiliary lanes, the addition of capacity through the use of narrow lanes or shoulders to eliminate isolated bottlenecks, and the reconfiguration of ramps or ramp geometry.

Construction and maintenance

Construction and reconstruction activities also reduce freeway capacity. These reductions can be minimized through the use of effective traffic control plans. Maintenance activities are similar to construction and reconstruction activities except that the duration tends to be shorter. Scheduling this capacity reduction during periods of lower demand can mitigate the effects of the reduction.

### Demand Management Strategies

Ramp closure and metering

The number of vehicles entering the freeway system is the primary determinant of freeway system performance. Entry control is the most straightforward way to limit freeway demand. Entry control can take the form of temporary or permanent ramp closure. Ramp metering, which can limit demand on the basis of a variety of factors that can be either preprogrammed or implemented in response to measured freeway conditions, is a more dynamic form of entry control. Freeway demand can be delayed (changed in time), diverted (changed in space to an alternative route), changed in mode (such as HOV), or eliminated (the trip avoided). The difficult issue in assessing ramp-metering strategies is estimating how demand will shift as a result of metering.

HOV alternatives

HOV alternatives such as mainline HOV lanes or ramp meter bypass lanes are intended to reduce the vehicle demand on the facility without changing the total number of person trips. Assessing these types of alternatives also requires the ability to estimate the number of persons who make a change of mode to HOV. In addition, it is necessary to know the origin and destination of the HOV travelers to determine what portions of the HOV facility they can use, since many HOV facilities have some form of restricted access.

Special events result in traffic demands that are based on the particular event. These occasional activities are amenable to the same types of freeway traffic management used for more routine activities such as daily commuting. In the case of special events, more planning and promotion are required than are typically needed for more routine activities.

Road pricing is a complex and evolving freeway traffic management alternative. Initially, road pricing involved a user fee to provide a means to finance highways. More recently, toll roads have been built as alternatives to congestion. Now, congestion-pricing schemes are being implemented to manage demand on various facilities or in some cases to sell excess capacity on HOV facilities. The congestion-pricing approach to demand management is to price the facility such that demand at critical points in time and space along the freeway is kept below capacity by encouraging some users during peak traffic periods to consider alternatives. Nontraditional road pricing schemes are still in their infancy, so little information is currently available on their effects compared with more traditional toll roads, which view tolls only as a means to recover facility costs.

### PERFORMANCE MEASURES

Performance measures for freeway facilities can be summarized by the user in the form of time-space domain contour maps. The most common contour maps are based on speed and density. The contours on the maps join points of similar traffic performance values. For example, the valleys in the speed contour maps indicate time-space regions of lower-speed operations, whereas the ridges in the density contour maps indicate time-space regions of higher-density operations. Careful selection of speed and density contour threshold values associated with capacity operations will clearly indicate boundaries between undersaturated and oversaturated flow conditions. Other contour threshold values can be selected to further identify different levels of undersaturated and oversaturated flow conditions.

Contour maps can also be constructed for volume-to-capacity ratios and congestion status. The volume-to-capacity ratio contour map is helpful in identifying bottlenecks ( $v/c$  values of 1.00) and segments operating close to capacity ( $v/c$  values  $> 0.90$ ). Congested portions of the freeway are identified by negative  $v/c$  ratios. The main purpose of the congestion status contour map is to provide the shapes and locations of congested regions. The vertical projection of the congested region denotes the duration of the congestion, whereas the horizontal projection of the congested region denotes the geographic extent of the congestion. An interesting means of summarizing the congestion status map is to calculate the area of the congested region on the contour map, which results in units of distance-hours of congestion.

Aggregating the estimated traffic performance measures over the entire length of the freeway facility provides facilitywide estimates for each 15-min time interval. Average and cumulative distributions of speed and density for each time interval can be determined, and patterns of their variation over the connected 15-min time intervals can be assessed. Trip times, vehicle kilometers (or person kilometers) of travel, and vehicle hours (or person-hours) of travel can be computed, and patterns of their variation over the connected 15-min time intervals can be assessed.

Aggregating the estimated segment traffic performance measures over the study time duration 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 their variation over connected freeway segments can be compared. Trip times, vehicle kilometers (or person kilometers) of travel, and vehicle hours (or person-hours) of travel can be assessed for each segment and compared.

The user can aggregate the estimated traffic performance measures over the entire time-space domain to provide an overall assessment of the entire freeway facility over the study time period. Average speeds, average trip times, vehicle kilometers (or person kilometers) of travel, and vehicle hours (or person-hours) of travel can be used to assess the overall traffic performance.

Special events

Road pricing

Time-space domain contour maps are effective tools for displaying performance measures

Frequency distributions are useful in displaying performance measures

## VI. REFERENCES

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