Notes on Freeway Capacity

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•THE FIRST part of this paper deals with rural freeways, and strictly speaking is not a discussion of capacity. However, it is a discussion of operating characteristics at volumes less than capacity which will result in a "level of service," subjectively determined to be desirable for rural or long distance conditions.

Both design capacity and possible capacity of a uniform segment of an urban freeway are then discussed. A knowledge of these values is necessary to determine the basic number of lanes in the design of a freeway, and to review conditions on an existing freeway where traffic congestion occurs. Traffic flow cannot be increased by revising the design of one segment of freeway or interchange if the downstream freeway leg is operating at capacity, and delay cannot be reduced unless traffic flow is increased or diverted to another route.

A knowledge of capacity is necessary to recognize and pinpoint the bottlenecks. Because traffic often flows smoothly at a bottleneck, many observers make serious mistakes in identification and pinpointing. Conversely, even when a bottleneck is identified and a cure is proposed, it is necessary to know whether the upstream freeway can furnish enough flow to take advantage of the increased capacity and whether some new bottleneck will make its appearance at a downstream location.

The terminology, "Urban Freeways," does not mean that these capacities are not valid under rural conditions. Given the same geometry, driver, and vehicle characteristics, the capacity of a freeway is the same in a city, suburbs, or rural areas.

"Analysis of Interchanges" presents a procedure for reviewing the design or operation of a given geometric layout to be sure that it will work. Ramp capacity and weaving and merging capacities are defined and analyzed. The procedure may seem complicated at first, but weaving is a complicated problem. It is hoped that practicing designers will produce simplified tables, charts, and nomographs to aid in the solution of problems for specific cases, as well as for the general case. However, the complexity of the problem means that oversimplification must be avoided.

The discussion of "level of service" is necessarily subjective to some degree. Values in Table 1 were agreed to by the HRB Highway Capacity Committee in January 1962. These values will replace the "Rural Practical Capacity" values of the 1950 edition of the Highway Capacity Manual when the new edition is published.

Other values and all figures are based on extensive observations and intensive study of California Freeways during the past seven years (1955-62). Observations are continuing with the objective of refining the given values and filling in the blank spots.

The effects of grades, coupled with the proportion and speed distribution of slower vehicles, are not wholly understood, but Figure 2 represents the best available estimate of these effects. Research is under way which may cause some future changes in this figure. However, it is now based on enough facts and study to warrant the statement that it is far better than any individual opinion or summation of opinions. Effects of weather and lighting conditions are not treated at all, and this also represents a deficiency in present knowledge.

With these exceptions and others specifically pointed out this report may be considered authentic.

The Subcommittee on Definitions of the Committee on Highway Capacity adopted the following definitions at the January 1962 meeting of the Highway Research Board:

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The possible capacity is the maximum number of vehicles that can pass over a given section of roadway in one direction during one hour under specified traffic conditions.

Design capacity is the number of vehicles that can pass over a given section of roadway in one direction during one hour under specified traffic conditions and operating at a level of service. The level of service should be based on an engineering evaluation of the probability of traffic interruptions, on desired speed of operation as determined by trip purpose, type and location of the facility, the cost of vehicle operation, and by the cost of building, maintaining and operating the highway.

A design capacity is a volume generally selected for design purposes which will provide a desirable level of service.

RURAL FREEWAYS

On rural freeways, where most trips are long, the traffic volume during the design hour should be low enough to provide a reasonable degree of freedom of maneuver and absence of tension on the part of the drivers. This volume is quite low in comparison with the capacity of the freeway.

Even at extremely low volumes, there will be occasions where the projected time-distance graphs of three cars driving at steady speeds on a 2-lane one-way roadway will all reach a given point on the road at one time and a certain amount of adjustment of speed is required. The aggregate of such adjustments is negligible, in terms of psychological annoyance, up to values to be discussed. On grades, the aggregate or cumulative adjustments or conflicts are more frequent, but if the grades are short or if they are long distances apart, the cumulative tension for the trip is not increased very much. On the other hand, the capacity of any grade should never be exceeded.

On 4-lane freeways, with two lanes in each direction, it was found that at about 1,400 vehicles per hour (vph) in one direction on a level grade, the faster group of drivers began to be reluctant to use the right-hand lane for fear of being "trapped" behind a slow vehicle while an entire platoon of fast vehicles passes the slow vehicle. When rates exceed this number, this effect begins to be significant and the trapped vehicles will begin to break into the platoons passing in the left lane.

Curves showing speed versus traffic volume are not sensitive enough to pinpoint this effect. The fast platoons in the left lane are traveling 55 to 65 mph and the slow vehicles in the right lane are traveling 45 to 55 mph. The average speed of all vehicles is very slightly less than it is during low-volume flow. An observer standing at one location will note that long intervals go by between platoons, during which all cars are free moving, and then a platoon will go by in which the headways in the left lane are very short. It does not look like heavy flow, but about 50 percent of the drivers will be in a state of tension, driving bumper-to-bumper.

When there are three or more lanes in one direction, the probability of being trapped in the slow lane is reduced to negligible proportions at hourly volumes of less than 1,500 per added lane. It follows that for a given level of freedom, a freeway having three or more lanes in one direction will allow for a higher average hourly lane volume.

Table 1 may be used as a guide for determining the traffic volume which will result in practically unrestricted flow on various widths of freeway. Values are shown both for passenger cars only and for a normal percentage of trucks or slow vehicles. This percentage rarely exceeds 50 percent during the peak hour.

TABLE 1

PRACTICALLY UNRESTRICTED FLOW ON LEVEL GRADES, RURAL LONG-DISTANCE FREEWAYS[a]

No. of Lanes One Direction	Hr. Vol. —One Direction	
	No Trucks	5% Trucks
2	2,000	1,700
3	3,500	3,000
4	5,000	4,400

[a]Very high level of service.

The values given in Table 1 are not capacity volumes.[1] The only reason for listing them is to evaluate a quality of flow that will be acceptable for long-distance travel with almost complete absence of tension and to show the effect of additional lanes for this quality of flow. In deciding what value to use for design capacity, as previously defined, the length of highway involved, the distribution of individual trip lengths, and the cost of providing a given level of service should all be taken into account.

Effect of Grades

On sustained grades (more than ½ mi), the right lane will be pre-empted by trucks, and if it is desired to maintain a quality of flow on the grade equal to the quality on the level, it is necessary to add a climbing lane whenever the one-way volume exceeds 1,000 vph. However, because of economic factors, it may not always be desirable to do this.

There is a certain amount of platooning even on level roads at the volumes given in Table 1. When a plus grade is introduced, these platoons become more serious controls on capacity. The frequency of these platoons or bunches, the speed at which they move, and the possible capacity of the roadway itself are functions of (a) number of slow vehicles, (b) speed of slow vehicles (rate of grade), and (c) length of grade. If the grade is short and there are few trucks, there is a certain probability that there will be no trucks on the grade. If the grade is longer, there will be a greater probability that trucks on the grade will be encountered. Also, if the grade is steeper (and thus trucks slower), trucks will be on the grade a greater proportion of the time. Research linking these variables is now under way but is not complete.

For the time being, it may be assumed that grades of less than 2 percent and less than ½ mi can be disregarded, when considering flow rates less than possible capacity. Grades between 2 and 3 percent will form queues, but they will move fast enough so that high rates of flow can be maintained and the queues will not accumulate.

Pending the results of current research, the freeway capacity chart (see Fig. 2) may be used as a guide.

URBAN FREEWAYS

Fundamental Considerations

On a level urban freeway, when traffic flow is heavy enough to raise any questions regarding capacity, individual headways between vehicles vary from 0.5 sec up. In other words, in a very short interval of time and for a very few vehicles, the rate-of-flow in one lane or one file of vehicles can be as much as 7,200 vph. However, on the whole it is found that any 100 vehicles traveling through a significant distance, such as a quarter-mile or more, will not accept average headways of less than 1.8 sec, which is a rate-of-flow of 2,000 vph.[2] Some drivers in the total stream will accept lesser headways and these drivers tend to drive in the left-hand or median lane. For example, lane volumes in the median lane on many freeways consistently reach 2,200 vph. This does not mean, however, that all the vehicles in the stream (on all lanes) are willing to accept such short headways.

For design purposes this value (2,000 vph) should be reduced by 10 percent which results in the following rule: The basic fact about freeway traffic flow is that average

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[1] The Highway Capacity Committee used the following to describe Table 1: "(This table) may be used as a guide for determining the traffic volume which will result in a level of service where most of the cars will be affected by other vehicles in the stream, but the conflict is not unreasonable, even for long trips."

[2] Observations of extreme rates-of-flow exceeding this value have frequently been made, but rates higher than 2,000 vph cannot be considered dependable. The rate of 2,000 is capacity in the same sense that 4,000 psi is the compressive strength of a given concrete mix, even though the batch might produce individual cylinders varying from 3,500 to 4,500 psi at failure.

headways of less than 2 sec should not occur except during short intervals such as when a slug of traffic from a surface street traffic signal enters a freeway in about 30 seconds. During any 5-min interval, enough space should be provided so that no more than 150 vehicles will pass a point in one file. This may be referred to as a rate of 1,800 vph per lane for short periods.

Peak Hour Factor

In describing traffic flow, the motorist considers that failure occurs when traffic comes to a stop. This is a good enough definition for the traffic engineer and highway designer. A stipulated rate-of-flow for a 5-min period can insure that this will not occur, and with a 10-percent margin for error, this rate-of-flow is 1,800 vph per lane (average of all lanes).

However, the rate-of-flow for the highest 5-min interval of an hour is always higher than the rate-of-flow for the whole hour. This is because there is a natural statistical variability among the 12 five-min intervals, and there is also a variation in demand, owing to office and factory closing times, etc., within an hour, despite the metering effect of the surface street system.

The ratio of the rate-of-flow during the highest five minutes to the rate-of-flow during the whole hour is called the peak hour factor (PHF). For example, if there are 165 vehicles in the peak 5 minutes and 1,800 in the whole hour, the PHF factor would be $165 \div 150$, or 1.1.

In large metropolitan areas, the peak 5-min rate-of-flow within an hour will be about 1.1 times the rate for a whole hour. For example, if the total hour volume were 1,800 vehicles per lane, the maximum 5-min rate-of-flow within the hour would be about 2,000 vph.

In smaller urban areas the peak 5-min rate-of-flow usually does not exceed 1.3 times the total hour rate.

It follows that if the volume in a large metropolitan area (PHF = 1.1) is predicted to be 1,800 per lane in a whole hour, and in a smaller area (PHF = 1.3) 1,500 per lane in a whole hour, the peak flow rates for short periods at both locations (and thus the probability of failure) will be about the same.

Urban Capacities

The preceding leads to the capacities given in Table 2 for a uniform segment of freeway, or "straight pipe" condition. These values are considered acceptable hourly operating volumes under "average" conditions. Average conditions are as follows:

1. Nearly level grade line (less than 2 percent).
2. About 3 percent trucks.
3. Absence of high-volume ramps in the vicinity which means straight pipe distribution of traffic among the lanes.

Acceptable volumes would be higher in the presence of one of the following factors:

1. Downhill grade line.
2. Less truck volume.
3. An "expanding" situation downstream. An expanding situation could be either the addition of a lane to the freeway, branch connection where the total number of lanes is increased and both legs have more than adequate capacity, or any other factor providing increased capacity.

Acceptable volumes would be lower in the presence of one of the following:

1. Sustained uphill grade line.
2. More truck volume.
3. Other factors causing mal-distribution of traffic.

Actually, average conditions may be considered hypothetical, and accepting operating conditions are not determined by the average, but rather by the sections of least capacity. For this reason, it is important for the engineer to exercise judgment and provide

TABLE 2

FREEWAY CAPACITY (HOUR VOLUME)[a]

Lane	Capacity (vph)										
	2 Lanes[b]			3 Lanes[b]			4 Lanes[b]			5 Lanes[b]	
	1.1 PHF	1.2 PHF	1.3 PHF	1.1 PHF	1.2 PHF	1.3 PHF	1.1 PHF	1.2 PHF	1.3 PHF	1.1 PHF	1.2 PHF
1 (rt.)	1,400	1,300	1,200	1,400	1,300	1,100	1,300	1,200	1,100	1,200	1,100
2	1,800	1,700	1,500	1,700	1,500	1,400	1,600	1,500	1,400	1,600	1,400
3	—	—	—	1,800	1,700	1,600	1,800	1,600	1,500	1,800	1,600
4	—	—	—	—	—	—	1,800	1,700	1,500	1,800	1,700
5	—	—	—	—	—	—	—	—	—	1,800	1,700
Total	3,200	3,000	2,700	4,900	4,500	4,100	6,500	6,000	5,500	8,200	7,500

[a] Queues will not develop and delay will be negligible.
[b] One direction.

a balanced design. The effectiveness of many miles of excellent design may be lost if adequate capacity is not provided for one or two short lengths.

Bottlenecks

Although Table 2 is useful in determining the basic number of lanes by freeway sections, it is not sufficient information to design an urban freeway. During the peak hours, operating conditions on urban freeways are a function of possible capacity of bottlenecks in the system which may or may not be dependent entirely on the number of lanes.

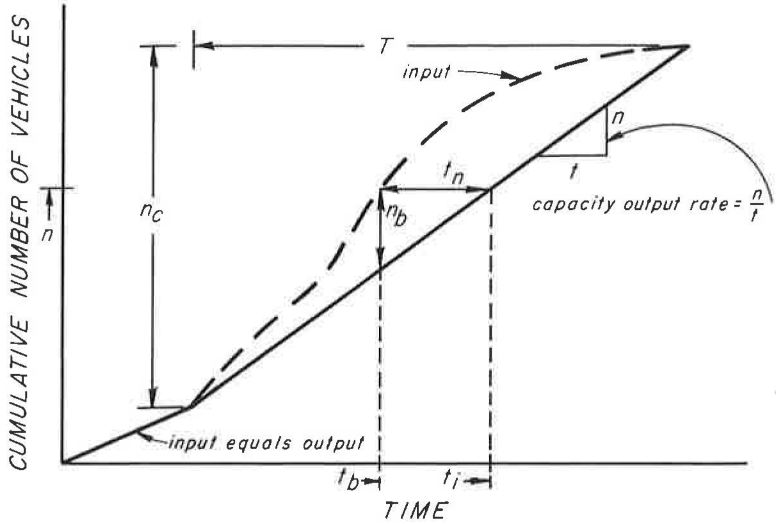
The traffic volume on an urban freeway will change at every entrance and exit ramp. Because of this, the ratio of demand to capacity varies from interchange to interchange. It is impossible to design a freeway so that this ratio will stay constant. Therefore, it is almost pointless to set up a lane-volume value in cars per hour to provide a given quality of flow along any significant length of highway. Driving along an urban freeway, even in a straight pipe condition between interchanges that are two or more miles apart, the individual driver encounters various instantaneous changes in conditions of flow. In one instant, he will be in the crest of a wave, and the next he might be in the trough.

When the input exceeds the capacity of a bottleneck, the freeway upstream from the bottleneck becomes a storage area and rate-of-flow in terms of cars per hour has no meaning. The rate-of-flow upstream of the bottleneck is independent of the geometric conditions at this location since it is bound to be equal to the rate-of-flow at the bottleneck.

Furthermore, when a bottleneck is operating at capacity, the speed of traffic upstream is also independent of geometric conditions on the upstream leg. The speed of traffic under such circumstances is a function of the excess of input over output and the length of time that the input rate has exceeded the output rate (Fig. 1).

When traffic is not backed up from a bottleneck, the average speed decreases somewhat as the rate-of-flow increases. The difference in speed is not significant in urban area capacity problems and should not be used as a criterion for determining acceptable operation. It should never be a consideration in establishing design speed. Design speed should be governed by operating conditions desired during off-peak hours. High standards of horizontal and vertical alignment will result in better operating conditions at very high volumes (even though speeds may be lower than design speed), and in greater safety at all hours of the day.

There are several conditions which can cause a bottleneck. The most frequent condition occurs where traffic is added to the mainline of the freeway without adding lanes to the mainline. This can occur at any entrance ramp along the freeway, and at a given total volume, is more likely to occur if the entering traffic is confined to a few high-volume ramps instead of several low-volume ramps. Another condition which can cause a bottleneck is a reduction in number of lanes. Other bottlenecks occur where the freeway begins an uphill grade.



n_b = No. of cars being delayed at time t_b, which is desired arrival time of nth car (nth car will arrive at upstream end of queue before this).

t_n = Delay suffered by nth car.

n_c = No. of cars delayed during entire period that queue exists.

Area between two curves = total delay in vehicle-minutes.

T = Total length of time that congestion lasts. Note that t_n, the delay suffered by any one car, is only a small fraction of T. Congestion may last 2 hours and the maximum delay to any one vehicle may be as little as 5 or 10 minutes.

If the average space headway in the queue is d, speed of nth car is a function of d, n_b, and t_n. Note that speed is dependent on capacity, not the other way around.

Figure 1. Relation between capacity and delay.

The problem is to define the locations of the bottlenecks and to provide adequate possible capacity at those locations. If this is done, the quality of service in between will take care of itself.

In a long straight pipe condition, traffic tends to distribute among the available lanes so that values such as given in Table 2 will apply. However, in the vicinity of bottlenecks, it is often found that distribution among the lanes does not follow the general pattern.

Bottleneck problems in general may be categorized as grade problems, where slow vehicles cause mal-distribution of traffic among the lanes, and merging and weaving problems at interchanges.

Grades

Figure 2 shows various levels of service as affected by long grades and a normal percentage of trucks. Although the precise effect of grades is not known, this may be used as a guide in evaluating grade problems for the time being or until further research requires a change.

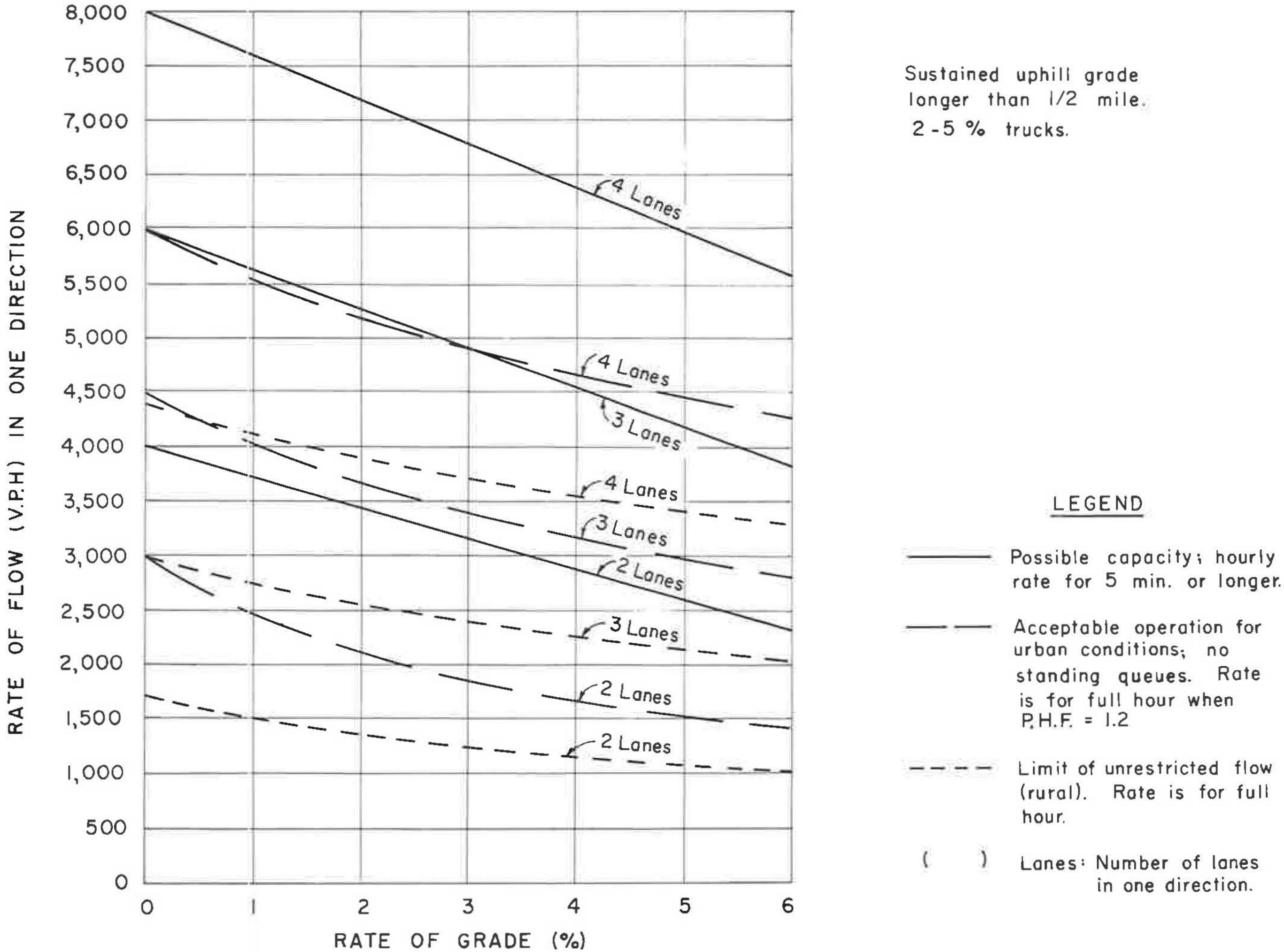


Figure 2. Freeway capacity.

ANALYSIS OF INTERCHANGE CAPACITY

The analysis of interchange capacity is essentially the analysis of conditions at ramp terminals.

Ramp Capacity

The rate-of-flow that an on- or off-ramp proper (turning roadway) can handle is about the same as a freeway lane or about 1,800 vph. Whether the ramp volume can be accommodated at the intersection with the surface street is a separate problem and should be analyzed as a regular street intersection problem.

When capacity is a consideration, any on-ramp roadway more than 1,000 ft long should be 2 lanes wide even when it is funneled to 1 lane at the merge. This allows passing and breaking up of queues and large gaps, thus permitting a more even arrival rate at the freeway and at higher speeds.

On an off-ramp, the amount of 2-lane roadway (or wider) beyond the exit nose is dependent primarily on capacity requirements at the surface street connection and storage space required.

The freeway terminals of ramps should be of standard design. The standard entrance ramp must provide (a) adequate merging distance for high speeds as well as low speeds at every location, (b) in combination with the approach ramp, adequate length for entering cars to accelerate from any turning speed, and (c) adequate merging distance for low volumes as well as high volumes.

Freeway to freeway connections are essentially the same as ramps and can be analyzed in the same manner. The turning roadway may be of a higher standard to permit higher speeds, but the terminals would be the same. The connections would be different only if the exit or entrance volumes were so high as to require dropping or adding a lane to facilitate 2-lane exits or entrances.

Two-lane ramp connections to the freeway are not generally used unless a lane is added or dropped, but in some cases, they are desirable even when a lane is not added or dropped. This could be the case when the ramp and freeway peak occur at different times. If 2-lane entrance ramp terminals are used, a parallel lane should also be provided for a substantial distance, in addition to the standard ramp taper, so that a portion of the ramp traffic will have a chance to move to the left before the remainder has to merge. Conversely, 2-lane exit ramps require a parallel deceleration lane in order to provide sufficient volume to utilize the lanes.

Calculating Weaving and Merging Capacities

As a first step in the design of a length of freeway, the number of lanes required is determined from the predicted hourly volume for the design year. For example, if the one-way hourly volume is predicted to be 6,000 vehicles, 4 lanes would be provided since an average of 1,500 vehicles per lane is within the limits of acceptable operations for 4 lanes (Table 2).

As a second step, flow by lanes must be checked in the vicinity of ramps. The following stipulations must be met (assuming grades of less than 3 percent and about 3 percent trucks):

1. Rate-of-flow in the right lane or auxiliary lane of a freeway or in a single-lane ramp should not exceed 1,800 vph.
2. Number of weaving vehicles should not exceed 2,100 vph in any 500-ft segment of a weaving section.
3. Average rate-of-flow across all lanes should not exceed 1,800 vph per lane.

As long as demand rate-of-flow (for 5 to 15 min) does not exceed the given limits, queuing or shock waves will not occur and operation upstream of the critical section will take the characteristics of straight pipe flow.

The described procedure only determines whether a certain volume level and traffic pattern will give acceptable operation; it does not evaluate quantitatively how much better operation would be for a certain lower volume level. The method is intended to be used to check a critical section to insure that it will work and not become a bottleneck for the predicted volume levels and traffic patterns or at least so that the limitations of the section will be realized.

Under normal conditions of straight pipe flow where there are no high-volume ramps in the vicinity, the lane distribution at near capacity conditions could be expected to be approximately as given in Table 2. Capacities might be reduced because traffic desires might be such that the general straight pipe distribution will not occur and an inordinate number of vehicles will try to use a single lane. Problems such as this occur, for example, at heavy volume ramps where a substantial portion of the traffic wants to be in the right lane and there is not enough traffic that will use the efficient high-capacity left lanes. (However, solving this problem by using left-hand ramps should not be attempted.)

Therefore, after the basic number of lanes and geometric design have been determined through the use of total-volume flow rates, lane distributions should be checked at any point where a bottleneck condition might be suspected.

Because rates-of-flow within an hour are higher than the flow for the full hour, the short-time rates of flow should be used in checking a section of freeway for its adequacy. Converting the full-hour volume to short-time flow rates is done by applying the PHF. All of the volumes or flow rates in the following refer to short-time rates.

Merging operation will be smooth as long as total ramp and adjacent lane rate-of-flow does not exceed 1,800 vph, provided that the entrance ramp terminal is long enough and has a gradual taper.

Maximum combined flow-rates for a merge of a particular ramp and adjacent freeway lane have been observed as high as 2,000 and 2,200 vph. However, it is not recommended that this value be anticipated in design procedures, since there are certain conditions of geometric design and traffic characteristics (which are difficult to predict or evaluate) that can prevent its attainment. A dependable figure is 1,800 vph which can be counted on under almost all circumstances, with normal truck percentages and grades of less than 3 percent.

Merging operation will vary considerably depending on the relative proportion of traffic on the ramp and adjacent lane. The smaller the number of ramp vehicles compared to adjacent lane vehicles (with the sum of the two being 1,800 vph), the better the merging operation. Entering ramp vehicles tend to move at slower speeds than freeway vehicles and often tend to arrive in platoons because of signal control. Thus, they are not as well spaced as freeway traffic, which causes higher instantaneous merging flow than would occur if ramp traffic arrived randomly. This also means that in most instances, two ramps of 400 vph each, will operate better than one ramp with a rate-of-flow of 800 vph.

In any case, regardless of the relative volumes, a combined flow rate of 1,800 vph will result in satisfactory operation. Operating conditions when this criterion is met will be such that average speeds (over the entire length of the merging area) will be between 30 and 40 mph.

Many times on a heavy-volume ramp the rate-of-flow on the ramp itself for 30 sec or a minute will be 1,800 vph, even though the flow rate over 5 or 10 min is only 800-1,000 vph. When this platoon arrives at the freeway, and if there are any vehicles in the adjacent freeway lane (as there almost always will be), severe reductions in speed will occur. If two cars arrive at the same spot at the same time, one will have to adjust its speed. It is a statistical certainty that will will happen at a ramp at almost any volume level—not as frequently at the lower volumes, but it will occur. This type of operation at ramps must be expected and not considered a failure in freeway operation. It cannot be designed out by assuming lower design capacities. Failure occurs when the queue does not dissipate, i.e., when the queue is continuous for several minutes.

This value, 1,800 vph (or an average headway of 2 sec) in any 5-min interval, is also the key for testing weaving lanes. In addition, the weaving that will take place in a short length must be checked. No more than 2,100 vph weaving should be permitted in any 500-ft segment of roadway, regardless of the number of lanes provided. (Weaving vehicles are defined as those that must actually cross paths; at least two lanes must be available and all weaving vehicles must cross the line—"crown line"—separating the two lanes.)

Possible capacity of a 500-ft length is about 2,300 vph but as in the case of possible capacity for merging traffic, it should not be counted on. Under most circumstances, 2,100 vph weaving in 500 ft can be reasonably expected. Speed and acceptable weaving volume are not directly related. Assuming a lower speed will not make the acceptable weaving volume higher. A given weaving volume will operate much more smoothly at high speeds than at low speeds.

Ordinarily, if the 1,800 vph in any one lane requirement is met, weaving volume will not be a control when the length available for weaving is 1,500 ft or more.

Examples of Procedure

The following examples illustrate the procedure and basic facts which are used to determine the lane distribution on a critical portion of the freeway so that the described procedure can be accomplished.

An 8-lane freeway with an on- and off-ramp is assumed, as shown in Figure 3. One-way traffic upstream of the on-ramp is at a rate of 5,500 vph. It will be developed that with 5,500 vph on the main line approaching the on-ramp merge, including 700 going to the off-ramp, 1,200 of the 5,500 will be in the right lane at the nose of the on-ramp. Since an auxiliary lane is not provided, all of the on-ramp vehicles must merge with this 1,200. Since rate-of-flow in a merging lane should not exceed 1,800 vph, 600 vph is the maximum rate-of-flow that may enter from the on-ramp.

If the off-ramp were a greater distance away from the on-ramp, then not all of the 700 off-ramp vehicles would be in the right lane, thus leaving room for more on-ramp vehicles. The improved distribution of traffic across all lanes would result in a higher capacity on the freeway between the on- and off-ramp.

If the ramps were 2,000 ft apart, then about 550 of the 700 off-ramp vehicles would be in the shoulder lane, thus leaving room for an additional 150 vehicles from the on-ramp (Fig. 4).

It is now assumed, in the case where the ramps are 1,000 ft apart, that the on-ramp has a demand of 1,200 vph. As illustrated, only 600 can be absorbed efficiently be-

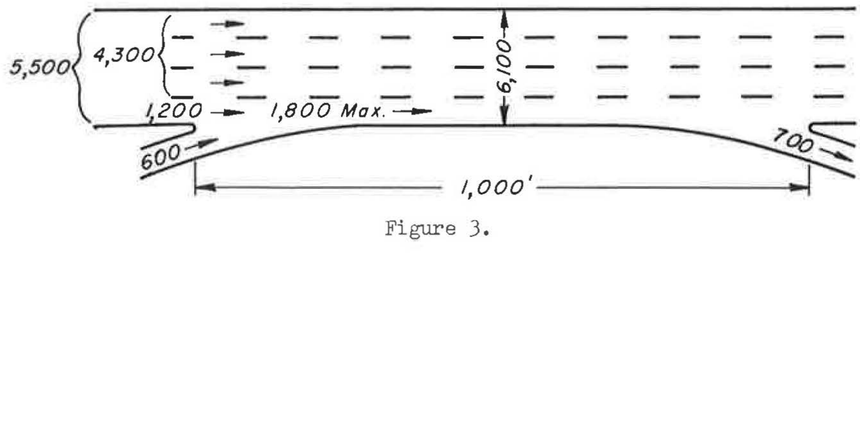


Figure 3.

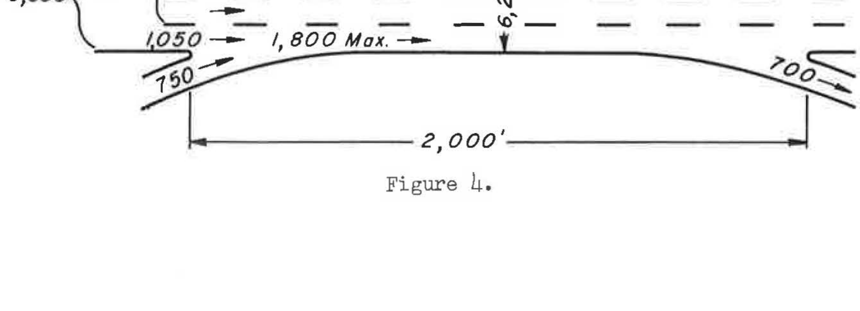


Figure 4.

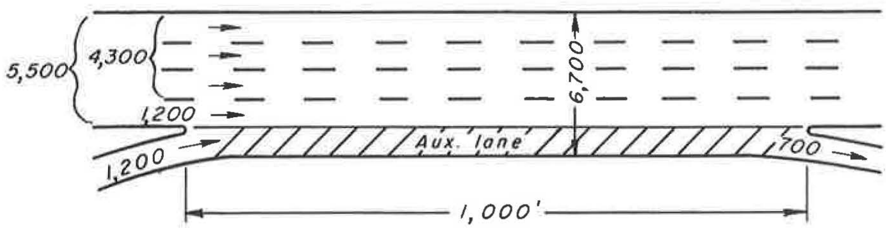


Figure 5.

cause there are 1,200 in the right lane already.[3] But if an auxiliary lane is provided between the two ramps, then the off-ramp vehicles can move to the right before on-ramp vehicles have to merge into the main stream. The on-ramp can absorb 1,200 vph, because lane changing is such that there will be no more than 1,800 vph at any point in the auxiliary or right lane. Therefore, by adding the auxiliary lane the capacity of the ramp is greatly increased (Fig. 5).

As previously stated, the principle is that traffic volume in a merging or weaving lane at any point should not exceed 1,800 vph.

The basic problem in implementing this procedure is to know how traffic will distribute across the freeway lanes.

Distribution of Traffic by Lanes

Traffic at a point on a freeway can be divided into three segments:

1. Through traffic—traffic not involved in ramp movements within a distance of 4,000 ft.
2. On-ramp traffic—traffic which has entered the freeway a certain distance upstream of the point or section under study. This distance is a variable to be put into the problem.
3. Off-ramp traffic—traffic destined for an off-ramp a certain distance downstream of the point or section under study. This distance is also an input variable.

Under most conditions, when capacity volumes are approached, each of these segments, which make up the total freeway flow, will be distributed in accordance with the curves in Figures 6, 7, and 8 (or Fig. 9 in lieu of 7 and 8).

The distributions presume the existence of demand for near-capacity volumes in the right lane at the point being considered. Unless there are about 1,800 vph total, in the right lane, the distribution is not necessarily valid. For example, assuming through traffic at a certain point on a 4-lane section (one-way) is 6,000 vph, Figure 6 would place 10 percent or 600 in the right lane. This is true provided that ramp vehicles will bring the total volume in the right lane at this point close to 1,800 vph. If ramps are so far removed from this point that little ramp traffic would be assigned to lane 1, then the 10 percent of the through traffic assigned to lane 1 would be too low. However, if the volume in the right lane comes out to be considerably less than 1,800 vph, then the section is obviously satisfactory and the actual distribution is of no significance. That is to say, the figures are valid when checking capacity conditions. For situations where volume is well below capacity, they are irrelevant.

The figures were developed from examination of actual cases operating satisfactorily. Additional research is being conducted to further verify and refine them, and to extend their range of application. Several examples comparing calculated volumes in the right lane with actual observed volumes are given in Appendix A.

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[3]The fact that only 600 can be absorbed efficiently does not mean that only 600 will get on the freeway. With a demand of 1,200, the difference of 600 will be partly waiting in a queue on the ramp, and partly in a queue on the freeway. The freeway flow will have broken down with long irregular queuing, mostly in the right lane but with spill-over queuing and stop-and-go operation in adjacent lanes. This type of operation results in hazardous lane-changing upstream.

The figures are intended for use with single-lane on- and off-ramps with or without an auxiliary lane between them. They will also be used for the more complex situations involving 2-lane ramps and branch connections. However, they may require some modifications and are currently under study. This procedure should not be used for left-hand ramps.

Limited observation indicates that the combined rate-of-flow for the left lane and a left-hand on-ramp of 1,800 vph will provide acceptable operation as in the standard right-side ramp. However, when the average volume on all lanes is 1,800 vph, smooth flow on the freeway between interchanges requires that the left lane be carrying high-volume rates of 2,000 vph or more. Left-hand ramps would cut this to 1,800. The difference could not be made up in the other lanes as volume rates in the right-hand lane would still be limited to 1,800 vph to maintain good operation. This capacity re-

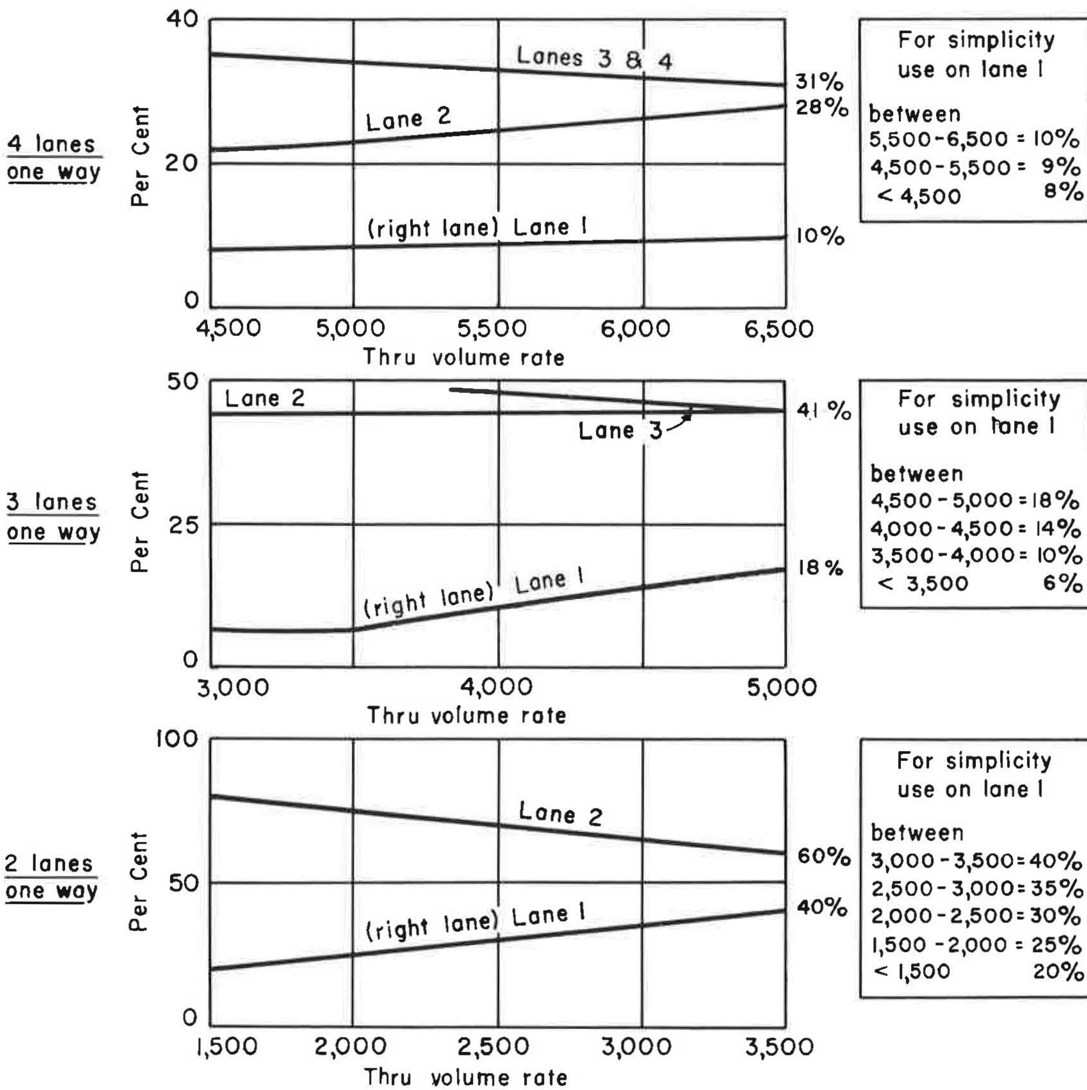


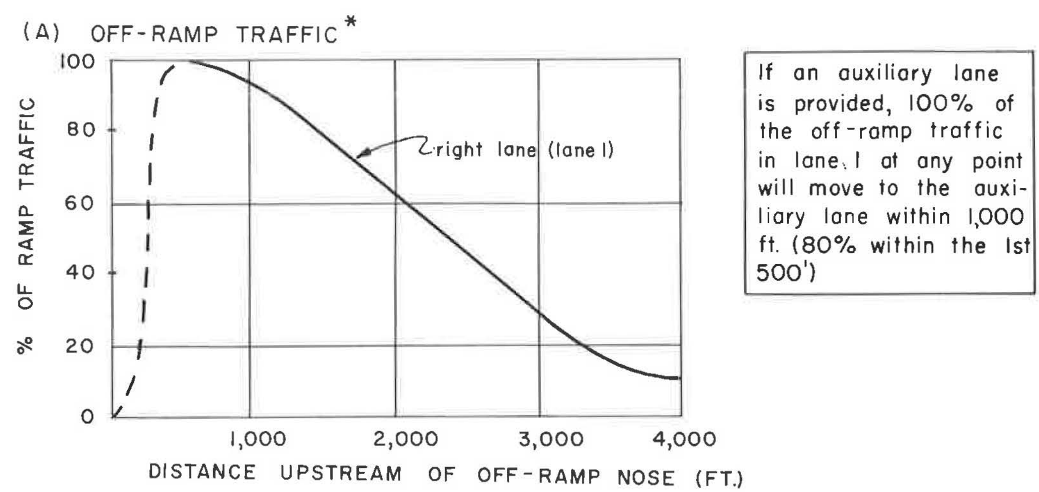
Figure 6. Distribution by lane of one-way through traffic, not involved in a ramp movement within 4,000 ft (percentages are not necessarily distributions under free flow or light ramp traffic, but under pressure of high volumes in right lanes).

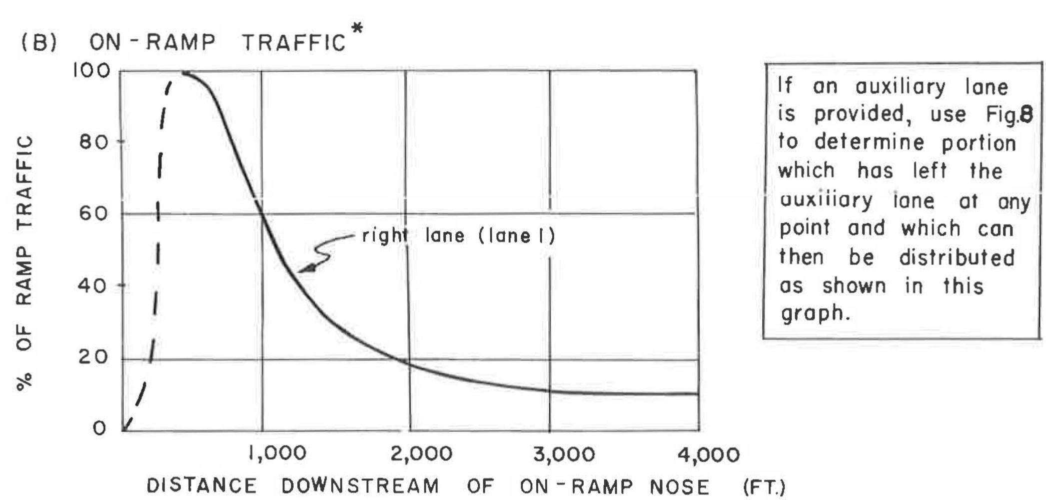
duction is in addition to other undesirable operational characteristics of left-hand ramps.

Figure 6 indicates the number of through vehicles that will stay in the right lane even though they are not involved in a ramp movement and are likely to be forced to adjust their speeds because of ramp maneuvering and statistical distribution of ramp traffic headways.

For example, assume 4 lanes one-way and 6, 300 vph through traffic (which is defined as traffic not involved in a ramp movement within 4, 000 ft). Reading from the graph, 10 percent, or 630 vph, will be in the right lane.

Figure 7 (A) shows the percentage of the off-ramp traffic in the right lane at any distance upstream of the ramp. The curve indicates that in the case of a conventional off-





* The percentage of ramp traffic in the right lane cannot be less than the percentage of thru traffic in the right lane (from Fig. 6). If the % comes out less, ramp traffic should be considered as thru traffic.

Figure 7. Percentage of ramp traffic in right lane (percentages are not necessarily distributions under free flow or light ramp traffic, but under pressure of high volumes in right lanes).

ramp (no auxiliary lane—a standard taper), 100 percent of the off-ramp traffic will be in the right lane at a point 500 ft upstream of the off-ramp nose. At a point 2,000 ft upstream of the nose, 63 percent of the off-ramp traffic will be in the right lane.

Figure 7 illustrates an important point in connection with an ordinary off-ramp. Because there is always some through traffic in the right lane, it would not be possible to supply 1,800 vph to an off-ramp even though the ramp might handle it. But if a parallel lane were added (an auxiliary lane in effect), 1,800 could be supplied to a ramp. For example, assume the following conditions: off-ramp demand is 1,800 vph, 350 vph going through in the right lane, and a parallel lane 1,500 ft long. At the beginning of the parallel lane (1,500 ft upstream of the off-ramp nose), 79 percent of the ramp traffic or 1,420 (0.79 × 1,800 vph) would be in the right lane. This combined with the 350 vph thru volume, a total of less than 1,800 is satisfactory. Then off-ramp traffic as it

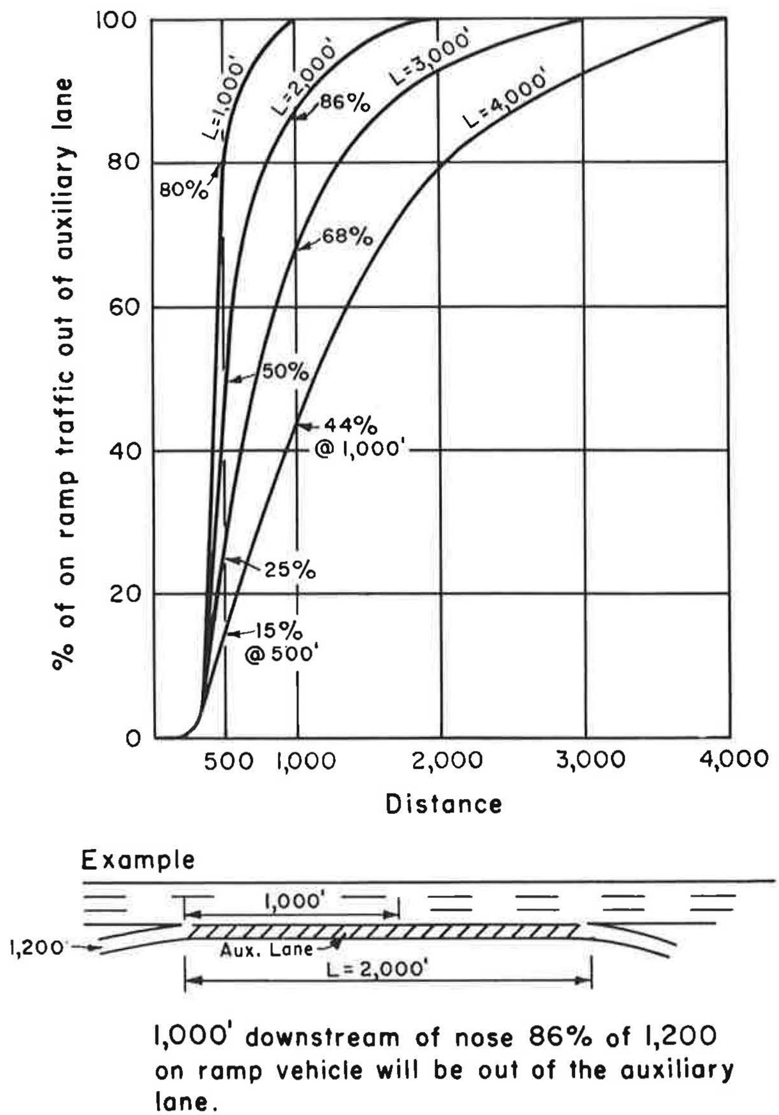


Figure 8. Percent of on-ramp traffic leaving auxiliary lane at any point for a given length of auxiliary lane (L).

progresses downstream will move into the parallel lane leaving room for the remaining 21 percent of the off-ramp traffic to move to the right lane. This effect has been observed at heavy off-ramps where cars create a parallel lane by riding the shoulder previous to the off-ramp deceleration lane.

Figure 7 (B) shows the percentage of on-ramp traffic in the right lane at any point downstream of the ramp. For example, 500 ft downstream of the on-ramp nose, 100 percent of the ramp traffic will have encroached on the right-hand freeway lane. The whole vehicle may not be in lane 1, but the left side will be close enough to create a headway unit in lane 1. One thousand feet downstream of the nose, 60 percent will be in the right lane with the other 40 percent having moved over to the left if there is room in the other lanes.

If auxiliary lanes between ramps are provided, basically the same system is used. In the case of off-ramp traffic, all off-ramp traffic in lane 1 at any point will move into the auxiliary lane within 1,000 ft (with 80 percent moving over within the first 500 ft). For example, assume an on- and off-ramp 1,000 ft apart with an auxiliary lane. As shown in Figure 7 (A) abscissa 1,000 ft, 95 percent of the off-ramp traffic will be in the right lane at the on-ramp nose. Five hundred feet downstream, 80 of the 95 percent will have moved over to the auxiliary lane leaving 19 plus the remaining 5 percent of the off-ramp traffic (100 minus 95 percent) in the right lane (see Fig. 9).

In the case of on-ramp traffic where an auxiliary lane is provided, Figure 8 should be used in conjunction with Figure 7. Figure 8 shows the manner in which ramp traffic leaves the auxiliary lane. For example, assume adjacent on- and off-ramps 1,000 ft apart. Figure 8 indicates that 500 ft downstream of the on-ramp nose, 80 percent of the ramp traffic will have moved to L_1. The traffic which has moved to the right lane is then distributed using Figure 7, which indicates that 60 of the 80 percent will still be in L_1 1,000 ft downstream of the on-ramp nose (see Fig. 9).

With these three figures, various traffic demands and geometric conditions involving adjacent ramps with or without auxiliary lanes can be checked to determine whether they will operate at acceptable levels, i.e., no more than 1,800 vph in the right lane or auxiliary lane.

Weaving volumes that take place in any 500-ft segment can also be determined from these graphs.

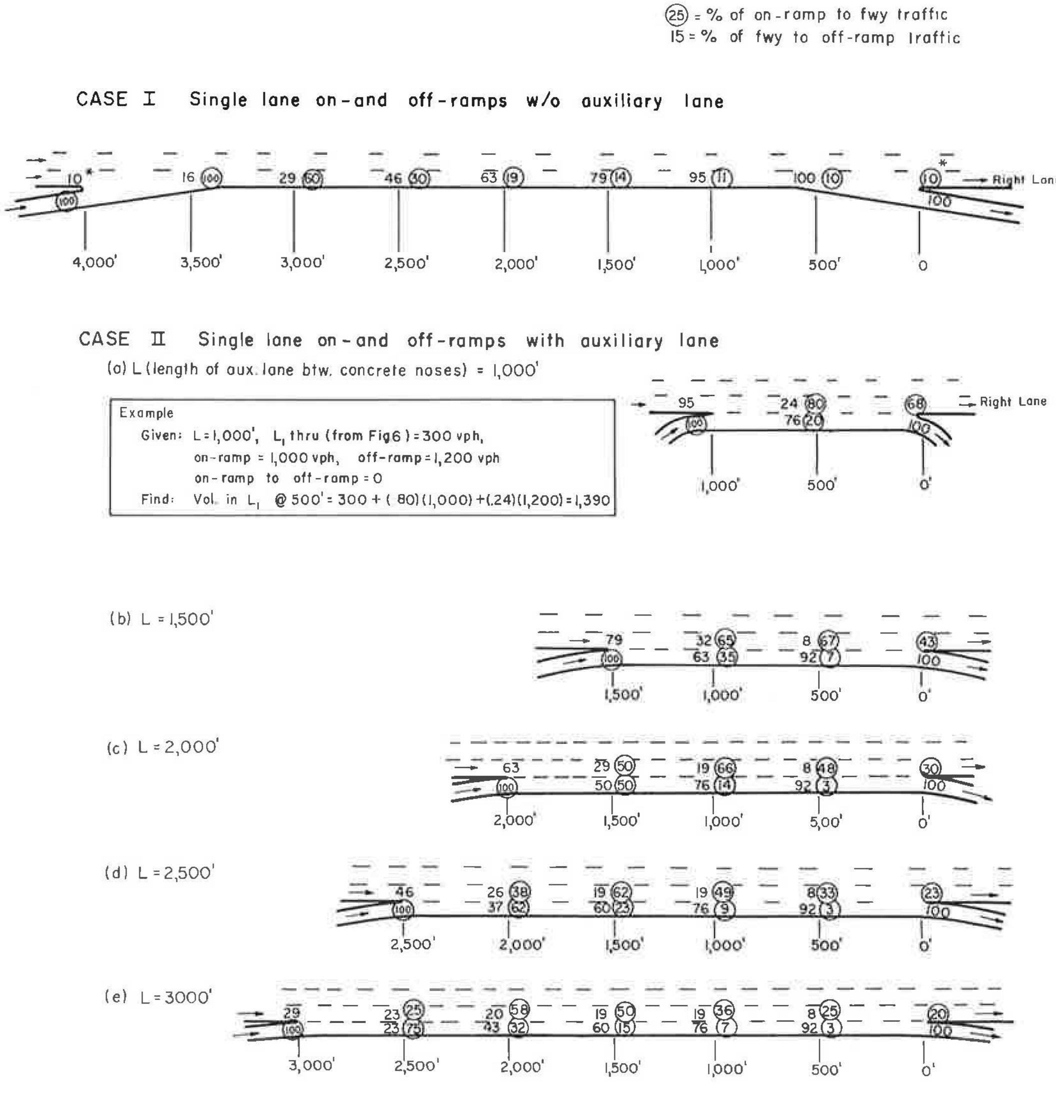
Figure 9 shows the distribution of ramp traffic at 500-ft spacings for several general cases. It is calculated from Figures 7 and 8 and makes it easier to solve general problems. For example, assume on- and off-ramps 1,000 ft apart with an auxiliary lane and the following traffic pattern: L_1 thru = 300 vph; on-ramp = 1,000 vph; off-ramp = 1,200 vph (and no on-ramp to off-ramp traffic). The critical point is at the 500-ft section. At this point, traffic in L_1 will be 300 (L_1 thru) plus 80 percent of the on-ramp traffic or 800, and 24 percent of the off-ramp traffic or about 300—a total of 1,400 which is satisfactory.

The weaving that takes place in a 500-ft section can also be determined. In the same example, in the first 500 ft, 80 percent of the on-ramp traffic will weave with 76 percent of the off-ramp traffic. This would be (0.80) (1,000) + (0.76) (1,200) = about 1,700 vph which is satisfactory.

Obviously, in actual practice there are few weaving sections with lengths that are exact multiples of 500 ft. However, the length of the section under investigation can be rounded to the nearest 500 ft, without exceeding allowable error in estimating the acceptability of traffic operation.

GENERAL REMARKS

To obtain maximum flow and good operation on the freeway, traffic needs a minimum of 600 ft to change lanes. Therefore, in addition to controls imposed by lane distribution of traffic, if vehicles must merge and then move to a second through lane (as in the case of a 2-lane off-ramp), the minimum distance between "paint" noses should be 1,200 ft regardless of the lowness of the weaving volumes. Since the paint nose, or actual confluence point, is offset several feet laterally from the concrete nose, the distance (on a flat taper) between the paint nose and the concrete nose is several hundred ft. The distance between concrete noses is seldom less than 1,800 ft (Fig. 10).



* Minimum % in right lane cannot be less than % of thru traffic in right lane as determined from Fig. 6.

NOTE: These percentages are not necessarily the distributions under free flow or light ramp traffic, but under pressure of high volumes in the right lanes at the point being considered.

Figure 9. Percentage distribution of on- and off-ramp traffic in right lane and auxiliary lane (calculated from Figs. 7 and 8).

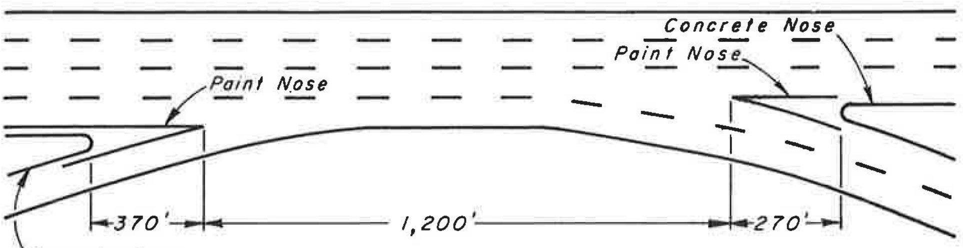


Figure 10.

Because of the length required between entrance and exit ramps, a collector road should be used on all cloverleaf interchanges whenever the weaving volumes exceed 1,200 vehicles an hour. The principle of a cloverleaf with two loops on one side of the freeway is basically incompatible with the principle sometimes expressed as "adequate spacing between interchanges."

If the total distance available for weaving is less than 500 ft, the allowable weaving is less than 2,100 vph. The allowable weaving volume is 1,500 vph when the actual weaving distance is 200 ft. For distances between 200 and 500 ft, the allowable weaving volumes can be assumed to vary linearly.

As an example, in a cloverleaf design where the distance between noses might be 400 ft, the maximum weaving volumes (regardless of the lane distribution factors discussed above) is $1,500 + \frac{200}{300}(2,100 - 1,500) = 1,900$ vph.

SUMMARY

The general procedure for checking weaving and merging capacity is:

1. Establish a given geometric condition.
2. Estimate volumes of the various traffic movements.
3. Use Figures 6 and 9 to determine volume at various check points. At any point in any lane, including the auxiliary lane, the volume should be 1,800 vph or less.
4. Average volume per lane across all lanes should not exceed 1,800 per lane.
5. Number of weaving vehicles in any 500-ft segment should not exceed 2,100 vph. (This ordinarily need not be checked except where the weaving section is 1,000 ft or less.)

If these conditions are met, weaving or merging is workable.

The previous discussion presumes a normal percentage of trucks and relatively level grade. Changes in percentage of trucks or grade will affect the capacities of ramps, and particularly the operational characteristics.

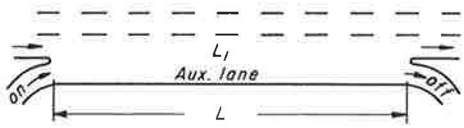
Appendix B gives some examples of the method. The computations can be rather complex in some cases but for general cases figures and tables can be prepared (Figs. 11, 12, and 13).

There are other variables which also affect the critical points on a freeway, but not enough is known about them to incorporate them in the procedure. These variables, which can include alignment, variation in grade, and composition of the traffic, should be considered subjectively in any case. For example, if the procedure shows that a merging lane has a flow rate of about 2,000 vph at some point, but there are very few trucks involved, tangent alignment exists, grade is downhill, or if the number of ramp vehicles is relatively small (500 ±), then perhaps this overloading could be tolerated. On the other hand, if the section is on a plus grade and a curve, then steps probably should be taken to try to reduce the conflict.

As has been noted, 1,800 vph in the right lane or auxiliary lane is below possible capacity and rates of 2,000-2,200 vph have been observed fairly frequently and sometimes operating acceptably. However, there are two reasons for not expecting or designing for this number in all cases:

1. As implied, rates this high are very sensitive to geometric design features and traffic characteristics.
2. Getting such high rates of flow requires that there be no large gaps in the traffic stream. To avoid these gaps (which always occur under free flow conditions), there has to be a constant supply or reservoir of traffic upstream of the merge. Often these extremely high rates are accompanied by some queuing (and thus, stop-and-go driving) upstream of the merge, even though the traffic demand over the short-time period may equal the output at the merge.

General Case - Single lane on- and off-ramp with auxiliary lane.



For L = 1,000'

L_1 thru = 0, ON to OFF = 0	L_1 thru = 300, ON to OFF = 0	L_1 thru = 600, ON to OFF = 0	L_1 thru = 900, ON to OFF = 0
ON OFF	ON OFF	ON OFF	ON OFF
0(vph) - 1800(vph)	0(vph) - 1600(vph)	0(vph) - 1250(vph)	0(vph) - 950(vph)
900........1800	1100........1600	1100........1250	850........950
1000........1700	1200........1500	1200........1000	900........700
1200........1500	1400........1300	1300.........700	1000........400
1400........1300	1500........1200	1400.........400	1100..........0
1600........1100	1600.........900	1500...........0	
1800.........900	1800.........250		
1800...........0	1800...........0		

L_1 thru = 0, ON to OFF = 200	L_1 thru = 300, ON to OFF = 200	L_1 thru = 600, ON to OFF = 200	L_1 thru = 900, ON to OFF = 200
0........1600	0........1600	0........1250	0........950
1100........1600	1100........1600	1100........1250	850........950
1200........1500	1500........1200	1200........1000	900........700
1400........1300	1600.........900	1300.........700	1000........400
1600........1100	1600...........0	1400.........400	1100..........0
1600..........0		1500..........0	

For L = 1,500'

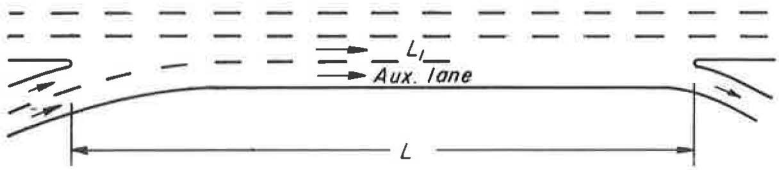
L_1 thru = 0, ON to OFF = 0	L_1 thru = 300, ON to OFF = 0	L_1 thru = 600, ON to OFF = 0	L_1 thru = 900, ON to OFF = 0
ON OFF	ON OFF	ON OFF	ON OFF
0........1800	0........1800	0........1500	0.......1150
1500........1800	1400........1800	1100........1500	800.......1150
1600........1700	1500........1650	1200........1300	1000........800
1800........1500	1600........1450	1400.........900	1200........400
1800...........0	1700........1250	1600.........500	1400..........0
	1800........1050	1800.........100	
	1800...........0	1800...........0	

For L = 2,000'

L_1 thru = 0, ON to OFF = 0	L_1 thru = 300, ON to OFF = 0	L_1 thru = 600, ON to OFF = 0	L_1 thru = 900, ON to OFF = 0
ON OFF	ON OFF	ON OFF	ON OFF
0........1800	0........1800	0........1800	0........1450
	1750........1800	1200........1800	950........1450
		1400........1400	1000........1250
1800........1800	1800........1650	1600.........700	1200.........550
	1800..........0	1800.........100	1400...........0
		1800..........0	

Figure 11. Acceptable ramp volume rates, calculated from Figure 9 (no more than 1,800 vph at any point in right or auxiliary lane; no more than 2,100 vph weaving in a 500-ft segment).

General Case - 2 lane on-ramp, 1 lane off-ramp, with auxiliary lane.



PRELIMINARY ONLY

For L = 1,500'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	1800	0	1800	0	1500	0	1100
700	1800	700	1800	1500	1500	1100	1100
2000	1500	1800	1600	1800	0	1400	0
2500	1100	2000	1400				
2700	0	2200	0				

For L = 2,000'

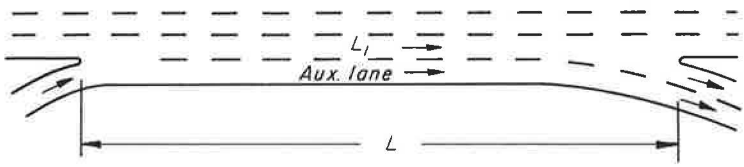
L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	1800	0	1800	0	1800	0	1450
2100	1800	1900	1800	1400	1800	1000	1450
2400	1500	2400	0	1900	0	1500	0
2900	0						

For L = 3,000'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600 ON to OFF = 0		L_1 thru = 900 ON to OFF = 0	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	1800	0	1800	0	1800	0	1800
2800	1800	2400	1800	1800	1800	1100	1800
3600	0	3100	0	2500	0	1800	0

Figure 12. Acceptable ramp volume rates (no more than 1,800 vph at any point in right or auxiliary lane; no more than 2,100 vph weaving in a 500-ft segment).

General Case - 1 lane on-ramp, 2 lane off-ramp, with auxiliary lane.



PRELIMINARY ONLY

For L = 1,500'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300		L_1 thru = 600		L_1 thru = 900	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	2300	0	1900	0	1500	0	1150
1000	2300	1400	1900	1100	1500	800	1150
1800	1500	1800	1050	1800	100	1400	0
1800	0	1800	0	1800	0		

For L = 2,000'

L_1 thru = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600		L_1 thru = 900	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	2850	0	2400	0	1900	0	1450
1150	2850	1300	2400	1250	1900	950	1450
1800	2300	1700	2050	1800	100	1400	0
1800	0	1800	1650	1800	0		
		1800	0				

For L = 3,000'

L_1 thru = 0 ON to OFF = 0		L_1 thru = 300 ON to OFF = 0		L_1 thru = 600		L_1 thru = 900	
ON	OFF	ON	OFF	ON	OFF	ON	OFF
(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)	(vph)
0	3600	0	3300	0	3000	0	2700
1100	3300	1000	3000	900	2750	800	2450
1800	3000	1800	2700	1650	2400	1500	2200
1800	0	1800	0	1800	2200	1800	1700
				1800	0	1800	0

Figure 13. Acceptable ramp volumes (no more than 1,800 vph at any point in right or auxiliary lane; no more than 2,100 vph weaving in a 500-ft segment).

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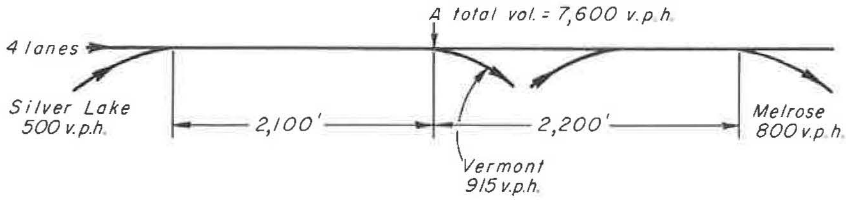
Appendix A

COMPARISON OF CALCULATED NUMBER OF VEHICLES IN THE RIGHT LANE WITH ACTUAL OBSERVED CASES

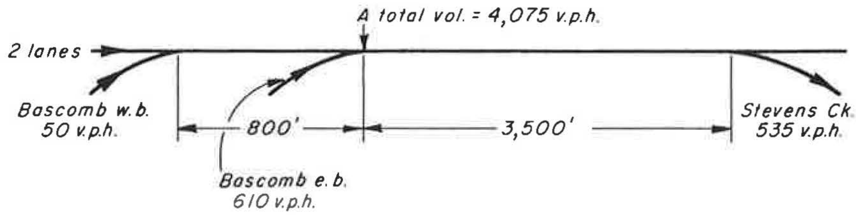
1. Hollywood Freeway at Vermont Avenue.[4]



Thru traffic (not individual in ramp movement with 4,000')
at A = 7,600 - (800 + 915 + 500) = 5,385
% in right lane = 9% = 485
Melrose traffic = 800
% in right lane at A = 60% = 480
Vermont traffic = 915
% in right lane at A = 100% = 915
Silver Lake traffic = 500
% in right lane at A = 18% = 90
Total in right lane at A = 1,970
Actual number observed = 2,003

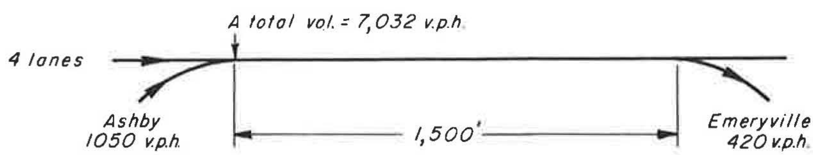
[4] Data are average of 2 observed peak hours (in 1956—average speed all lanes 45).

2. Eastshore Freeway at Ashby Avenue.[5]
(Berkeley, California)



Thru traffic (not involved in ramp movement within 4,000')
at A = 7,032 - (1,050 + 420) = 5,562
% in right lane at A = 10% = 560
Ashby Avenue traffic = 1,050
% in right lane at A = 100% = 1,050
Emeryville traffic = 420
% in right lane at A = 80% = 340
Total in right lane at A = 1,950
Actual number observed = 2,022

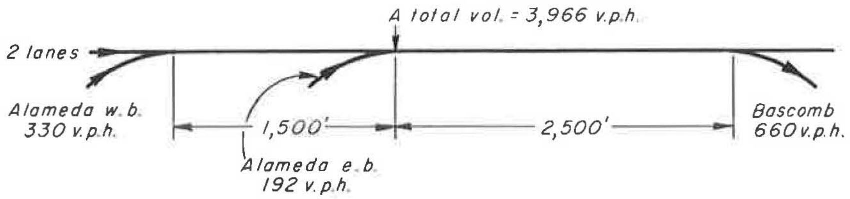
3. San Jose-Los Gatos Freeway at Bascom Avenue (San Jose, California).[6]



Thru traffic at A = 4,075 - (535 + 610 + 50)
= 2,880
% in right lane = 35%
Stevens Creek traffic = 535
% in right lane = 15% (from Fig. 7) but since the % in the right lane is less than that of the through traffic, this traffic should be assumed to be through traffic (i.e., % of ramp traffic in right lane cannot be less than % of thru traffic in right lane).
Recalculate thru traffic =
2,880 + 535 = 3,415
% in right lane at A = 40% = 1,370
Bascom Avenue EB traffic = 610
% in right lane at A = 100% = 610
Bascom Avenue WB traffic = 50
% in right lane = 70% = 35
Total in right lane at A = 2,015
Actual number observed = 2,034

[5] Hourly rate for peak 10 minutes during peak hour. Very smooth flow, no queues though there was one merge causing instantaneous stoppage for 15-20 seconds.
[6] Hourly rate for peak 10 minutes. No stoppages occurred during the period.

4. San Jose-Los Gatos Freeway at the Alameda (San Jose, California).[7]

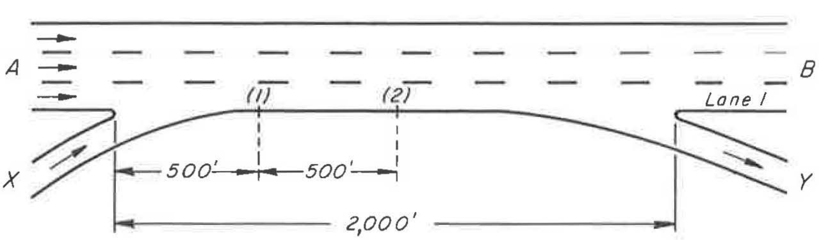


Thru traffic at A = 3,966 - (660 + 192 + 330)
= 2,784
% in right lane at A = 35%
Alameda WB traffic = 330
% in right lane at A = 30% (from Fig. 7) but since the % in the right lane is less than that of the thru traffic, this traffic should be assumed to be thru traffic (i.e., % of ramp traffic in right lane cannot be less than % of thru traffic in right lane).
Recalculate thru traffic =
2,784 + 330 = 3,114
% in right lane at A = 40% = 1,245
Alameda EB traffic = 192
% in right lane = 100% = 192
Bascom Avenue traffic = 660
% in right lane at A = 46% = 305

Total in right lane at A	=	1,742
Actual number observed	=	1,914

Appendix B

Example - 1



Given: (or assumed)

(a) 6-lane freeway
(b) on- and off-ramp 2,000' between concrete noses (no other ramps within 4,000')

[7] Data are hourly rate for peak 10 minutes during peak hour. Operation of merge was very good—capacity was not reached.

(c) Traffic data

A to B = 4,000
X to B = 700
A to Y = 600
X to Y = 0

Find lane volumes

a. Average lane volume = 5,300 ÷ 3 = 1,770

b. Check lane 1 volume at (1)

Thru traffic in right lane (from Fig. 6) = 0.14 x 4,000	= 560
On-ramp traffic in right lane (Fig. 7 or 9) = 100% x 700	= 700
Off-ramp traffic in right lane (Fig. 7 or 9) = 79% x 600	= 470
Total in right lane at (1)	= 1,730

c. Check lane 1 volume at (2)

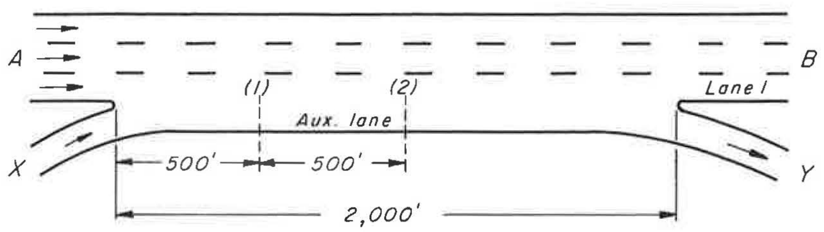
Thru traffic in right lane	= 560
On-ramp traffic in right lane (0.60 x 700)	= 420
Off-ramp traffic in right lane (0.95 x 600)	= 570
	1,550

Comments on the example:

It can be seen that the section would operate satisfactorily and the design would be acceptable since all conditions of the procedure are satisfied. However, a relatively small increase in the volumes or change in traffic patterns could change this fact. It then becomes an economic question whether to build in an extra safety factor by adding an auxiliary lane on this which perhaps might be the most critical section of a freeway. See Example 2 for solution using same volumes with auxiliary lane.

The described procedure only determines whether a certain volume level and traffic pattern will give acceptable operation. It does not evaluate quantitatively how much better operation would be for a certain lower volume level. The method is intended to be used to check a section to insure that it will work and not become a bottleneck for the predicted volume level and traffic patterns.

Example - 2



Traffic:

A to B = 4,000
X to B = 1,300
A to Y = 600
X to Y = 0

a. Average lane volume at (B) = $\frac{5,300}{3}$ = 1,770

b. Check lane 1 volume at (1)

Thru traffic = 0.14 x 4,000 (from Fig. 6)	= 560
On-ramp traffic in right lane = 50% x 1,300 (from Fig. 9)	= 650
Off-ramp traffic in right lane = 0.29 x 600 (from Fig. 9)	= 170
	1,380

c. Check lane 1 volume at (2)

Thru traffic	=	560
On-ramp traffic (0.66 X 1,300)	=	860
Off-ramp traffic (0.19 X 600)	=	110
		1,530

Comments on the example:

As can be seen adding the auxiliary lane greatly increases the ramp capacity.

Usually volumes in the auxiliary lane do not have to be checked unless there is more on-ramp to off-ramp traffic than thru traffic in L_1 which is not likely. Weaving was not checked since the total weave (1,900) is less than 2,100 vph. If it were, however, Figure 9 shows maximum weave takes place in the 1st 500 ft and is 50% of both on-ramp and off-ramp traffic or 950 vph.