HIGHWAY CAPACITY

THE LEVEL OF SERVICE CONCEPT

by

Professor Charles J. Keese
Executive Officer, TTI, and Professor
of Civil Engineering

Dr. Charles Pinnell
Associate Research Engineer and Associate
Professor of Civil Engineering

Dr. Donald R. Drew
Associate Research Engineer and Associate
Professor of Civil Engineering

Presented by

Professor Charles J. Keese

at the

35th Annual Meeting of the Institute of Traffic Engineers
"A World Traffic Engineering Conference"
Statler Hilton Hotel, Boston, Massachusetts
Session VI, October 21st, 2-4:30 pm

Bulletin 34

TEXAS TRANSPORTATION INSTITUTE
Texas A&M University
College Station, Texas

HIGHWAY CAPACITY

THE LEVEL OF SERVICE CONCEPT

This report is based on a number of research studies con-

ducted by the Texas Transportation Institute in cooperation with

the Texas Highway Department and Bureau of Public Roads. The

contributions of the members of the staffs of these organizations

are hereby acknowledged. The valuable contributions of other

researchers are also acknowledged.

INTRODUCTION

During the past 50 years, or so, motor vehicle transportation has pro-
gressed dynamically from a beginning existence and is now a major economic
force in our society. In its development, emphasis was placed on providing
ever better conditions for vehicle use. Just a few years ago, highways and
streets were built for the purpose of providing for mobility during all kinds of
weather. These roads made automobile transportation more practical and ec-
onomical and thus more popular and people soon became much more dependent
upon it.

In the years prior to World War II, the main purpose of the street or high-
way, except for a relatively few locations, was to provide all-weather routes
for motor vehicle transportation. Traffic volumes were relatively low and
major emphasis was placed on access to adjoining property.

There is evidence, however, that studies and applications of the capacity
concept date back to the early 1920's. In fact the basic level of service con-
cept is not new. There is a significant body of literature dating back more
than thirty years illustrating that early practitioners appreciated the fact
that capacity and quality of service should both be considered. To quote one
of the early practitioners of this concept, "The idea of measuring and es-
timating traffic flow capacities had to be sold and it took a long time. It did
not begin to be generally accepted as a useful tool in highway design, op-
erations and traffic control, regulation and management, until after World War

II."*

The formation of the Highway Capacity Committee in the Highway Research Board about 1944 brought together under the chairmanship of the late O. K. Normann, some of the individuals who were involved in developing the early capacity concepts.

This early work provided the basis for the method of measuring capacity reported in the 1950 edition of the Highway Capacity Manual. This contribution of the Capacity Committee of the Highway Research Board has served vitally in modern highway design.

For the most part, the Manual was based on two capacity levels -- possible and practical. "Possible" represented the maximum hourly volume under prevailing roadway and traffic conditions and "practical," "the maximum number of vehicles than can pass a given point on a roadway or in a designated lane during one hour without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the driver's freedom to maneuver under prevailing roadway and traffic conditions."[1]

Under proper application, this concept served most satisfactorily for a number of years and those who developed it are certainly due a great amount of credit for the fine research and common sense that went into this masterpiece.

For rural conditions, the basic concepts of the 1950 Manual are probably still quite applicable. The desire for higher speeds and better conditions however, makes it desirable to review the concept even for strictly rural conditions.

---

*Guy Kelcey (Edwards and Kelcey, Engineers and Consultants), letter to Charles J. Keese, Executive Officer, Texas Transportation Institute, September 14, 1965.

In urban conditions, the desire for a more dependable and higher quality of service has brought about a change in the application of "capacity" in the design and operation of streets and highways. This new application has been termed the Level of Service Concept and attempts to take into account speed and travel time, traffic interruption, freedom to maneuver, safety, driving comfort and convenience, and vehicular operational costs.

Since the latest work of the capacity committee is not yet published, it would be inappropriate to publish any part now being considered. Yet the relationship between capacity and level of service is so fundamental to such practical aspects of traffic engineering as planning, design, and operations, that it is important that the distinction between these terms be appreciated.

In effect, the revised Manual will not be a major departure from the 1950 Manual as far as "Capacity" is concerned. It will, however, attempt to provide those responsible for the design and operation of traffic facilities with a range of service volumes (rather than one "practical" capacity) which will be related to various conditions of operation or levels of service. The highest quality or level of service can then be provided based on economics and other management and engineering controls.

The new Manual takes into account several characteristics for which data were not available in the preparation of the previous Manual. For one thing, the variation of flow or peak flows within the hour has been considered. The short period peak rate of flow (5-to-15-minute period) is accounted for through the application of a peaking factor[2,3,4,5] (the ratio of the whole hour volume to the highest rate of flow occurring during a 5-minute or 15-minute interval within the hour). The consideration of these short-peak flows will much more accurately relate traffic flow (volume) to traffic operation or conditions experienced by the motorists.

Both capacity and level of service are functions of the physical features of the highway facility and the interaction of vehicles in the traffic stream. The distinction is this: a given lane or roadway may provide a wide range of levels of service, but only one possible capacity. The various levels for any specific roadway are a function of the volume and composition of traffic. A given lane or roadway designed for a given level of service at a specified volume will operate at many different levels of service as the flow varies during an hour, and as the volume varies during different hours of the day, days of the week, periods of the year, and during different years with traffic growth. In other words, fluctuations in demand do not cause fluctuations in capacity, but do affect changes in the quality of operation afforded the motorist. In a very general way then, highway planning, design and operational problems become a case of whether a certain roadway (capacity) can handle the projected or measured demand (volume) at an acceptable level of service (speed, etc). Because of both observed and theoretical speed-volume rela-

tionships on freeway facilities, which shall be considered later, it is possible to anticipate to some degree just what level of service can be expected for a given demand-capacity ratio. The obvious weakness lies in the fact that most of the qualitative factors affecting level of service have not yet been related directly to traffic volume.

Greater dependency on motor vehicle transportation has brought about a need for greater efficiency in traffic facilities. The motorist is no longer satisfied to be "out of mud." In fact, fewer and fewer folks remember the days of unpaved roads. The freeway is an outgrowth of the demand for highways which provide higher levels of service. The place that motor vehicle transportation plays in our society demands that dependable service be provided by traffic facilities and the popularity or attraction to the freeway illustrates this point. It is very important that the engineer clearly understands the factors affecting efficiency or level of service of our highways and streets.

An individual street or highway is a part of a network or system of facilities that together provide for access and movement of motor vehicle transportation. Various types of streets and highways in the system serve different functions and are generally grouped according to the two major functions of access and traffic movement. Those designed primarily for land access are generally referred to as local streets; collector facilities serve both access and traffic movement more or less equally. Major arterials must accommodate greater volumes of traffic for longer distances and therefore movement is primary but the land access function is also served. The freeway type facility is designed primarily for traffic movement with access not being a function of this type of facility.

The higher types of facilities, major arterials and freeways, are generally the types requiring careful analysis of capacity and level of service. The intersection is the major capacity problem on at-grade facilities such as major streets. The freeway epitomizes the level of service concept since it is designed to provide a high quality of traffic movement.

This publication deals with the two major problems of the application of the level of service concept to the design and operation of freeways, and discusses capacity design of high-type signalized intersections.

As was mentioned earlier, since the Revised Highway Capacity Manual has not yet been published, it would be improper to publish its contents. This publication, therefore, in discussing capacity and level of service, attempts to add to the material which will be published in the Manual. Although some of the methods are different from those presented in the Manual, they are felt, at this point at least, to complement rather than conflict with the Manual.

- 4 -

FREEWAY LEVEL OF SERVICE

Planning and Design Criteria

Almost any engineering design problem may be described as a systematic attempt to resolve a capacity-demand relationship at an acceptable level of service. For example, pavement engineers design the materials of the highway layers to withstand the shear stresses due to anticipated vehicular loads. However, the mere fact that the strength of the materials (capacity) exceeds the load stresses (demand) does not guarantee an acceptable level of service to the motorist. The pavement deflection, smoothness, texture, and color contrast also affect the driver's comfort, safety, and convenience and, as such, are level of service factors that must be considered.

The traffic engineers' problem in freeway design is basically one of estimating the parameters defining the traffic demand and capacity and/or level of service of the facility. He must provide a sufficient number of traffic lanes and other geometric features to handle anticipated traffic loads at an acceptable level of service. To design for adequate roadway conditions, the engineer must have an understanding of, and reliable data pertaining to, present and future traffic conditions.

No phase of highway development requires more careful analysis and study than the urban freeway. Only through comprehensive knowledge of the origin and destination of traffic in the metropolitan area can the route be properly located and designed. Planning criteria, with regard to freeways, are based on the probable usage of the proposed facility and the resultant effect upon adjacent arteries. Traffic desires must be considered in coordination with the development of new subdivisions, industrial areas, business areas, and changes in zoning. Urban freeways often block and cross existing streets, and thus affect the services rendered by the streets in different areas. The location of interchanges also affects the continuity of streets as well as the land-use characteristics.

Freeway design criteria are expressions of the major controls for which the freeway is designed. The three major controls are traffic volume -- its distribution and composition, design vehicle, and design speed. It is generally accepted[1] that the basic design designation for a proposed freeway should include the: (1) average daily traffic ADT (current year and future design year), (2) design hourly volume DHV (future design year), (3) directional distribution D, (4) percentage of trucks during the design hour T, (5) design speed during the peak and off-peak periods, (6) degree of control of access (i.e., full or partial) and the design vehicle (i.e., P, SU, etc.). These designations indicate the broad services for which the facility is being designed. However, as a complete basis for rational geometric design, these designations are inadequate.
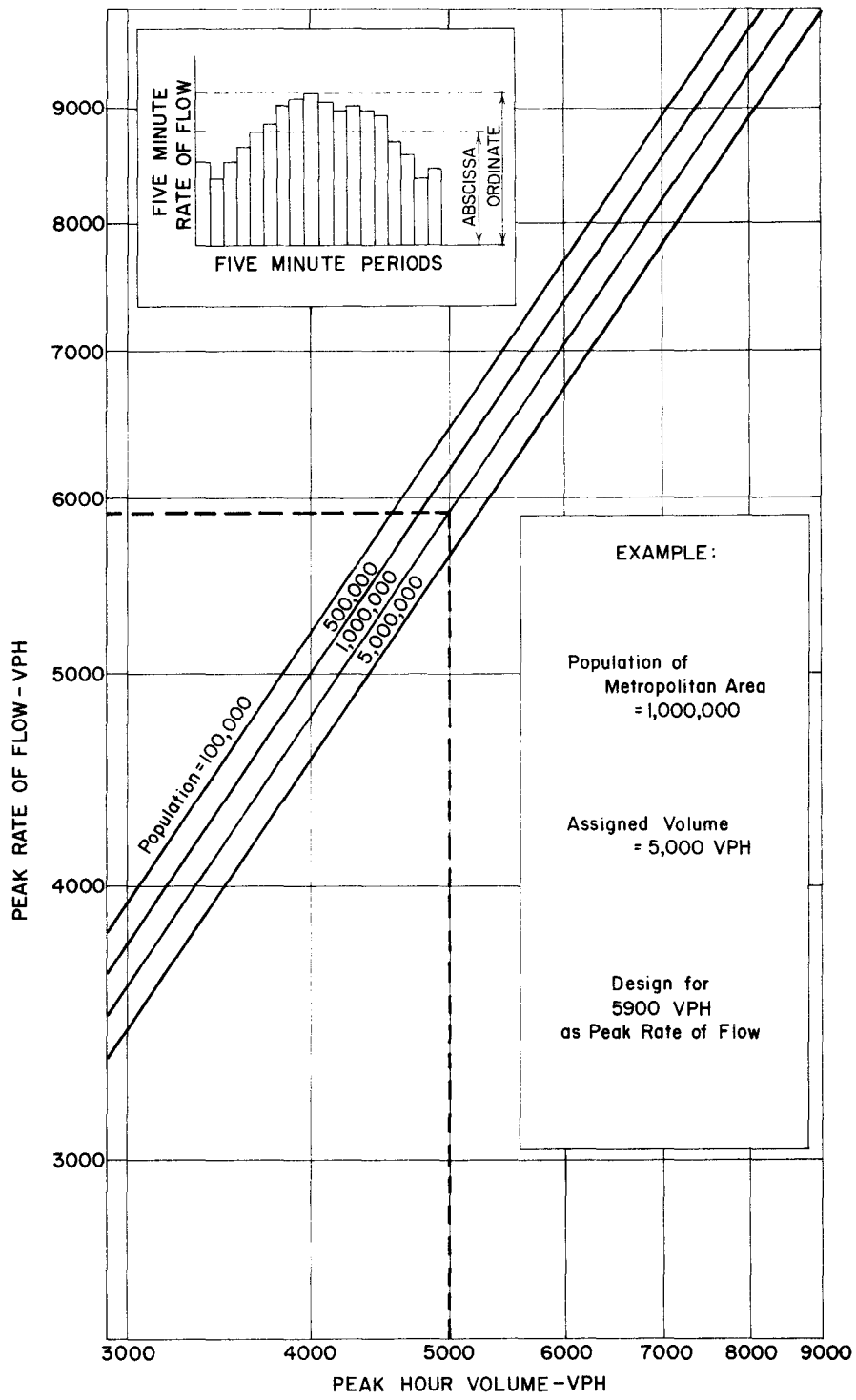
Peak Period Demand and Lane Distribution

Traffic flows within the peak hour are by no means constant but are known to be continuously variable. Secondly, although the design and markings of freeways tend to separate the traffic stream into lanes of flow, the total freeway demand at any point will seldom be equally distributed among the freeway lanes. The need to consider peak rates of flow within the peak hour and the transverse distribution of traffic on a freeway has been recognized for many years.[2, 3, 4, 5]

The analysis of rates of flow within the peak hour or "peaking" characteristics is a logical extension of the seasonal, weekly, and daily cyclic volume patterns which have been explored in the past in the selection of representative design hourly volumes. Although many characteristics related to trip generation such as the geographical and time concentrations of trips, character of the freeway (radial, circumferential, etc.), character of the supporting street system, and area served are suspected of having marked effects on the peaking characteristics; efforts to correlate peaking with these parameters have not been successful. It has so far only been possible from the data available to establish the relationship of peaking to the population of the city or urban area (Figure 1). These statistically significant curves, based on data from 132 peak periods from studies in 31 cities in 18 states, illustrate that the peak period factor (highest 5-minute rate of flow divided by peak hour volume) varies inversely with population.[6] Since the frequency distribution of per cent error involved in using Figure 1 to estimate peak rates of flow was normally distributed with a standard deviation of 5%, the probability of any design volume being exceeded can readily be predicted as shall be explained later.
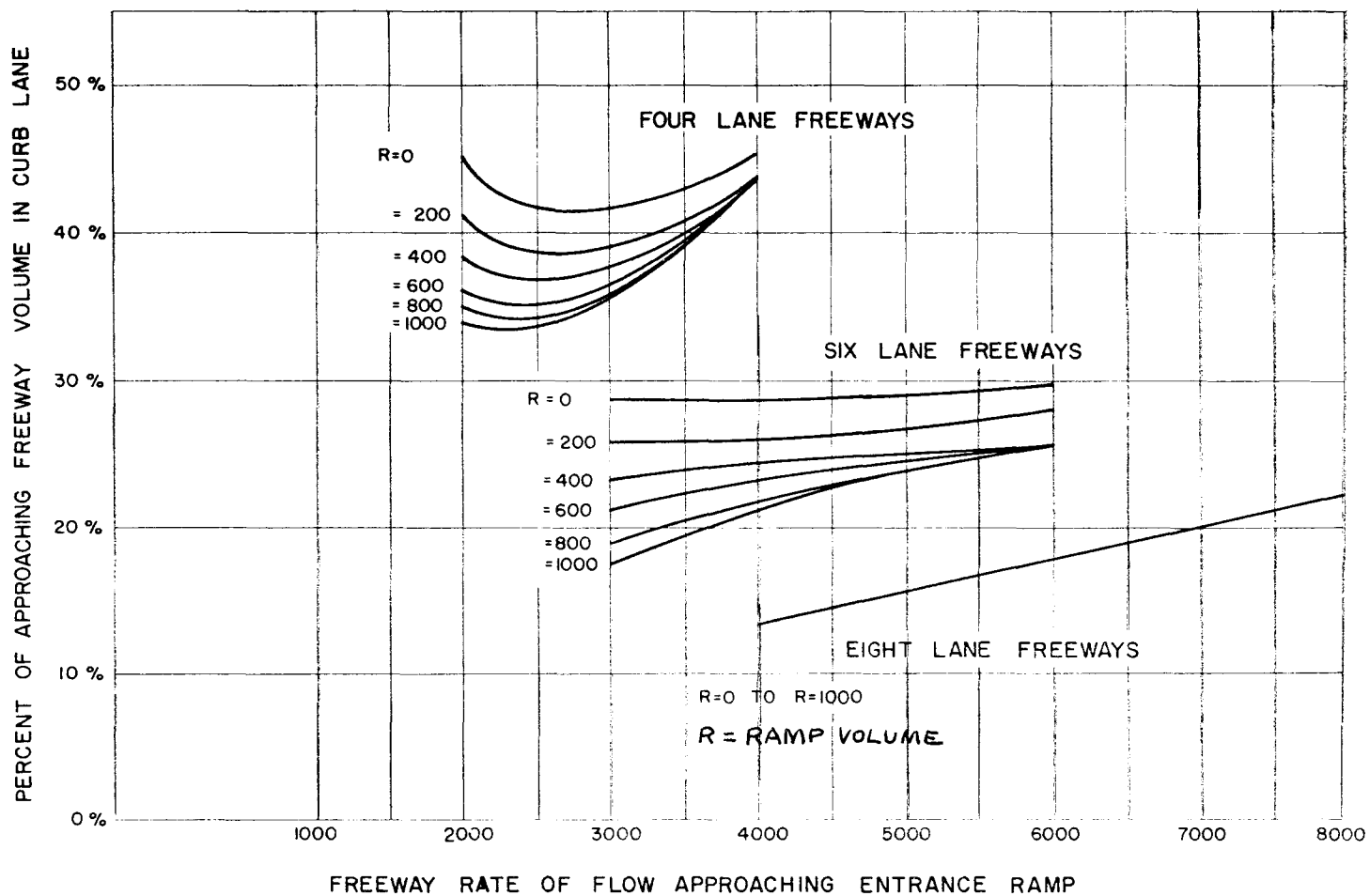
It has been illustrated in Figure 1 how the rate of flow for the highest 5-minute interval can be determined from the rate of flow for the whole peak hour. These flows represent the demand on all lanes and as such are inadequate for making a capacity-demand analysis at critical sections of the freeway. Such critical sections often exist adjacent to ramps and, if a certain level of service is to be assured the motorists, it is necessary to give close attention to such areas. Because the merging problem directly involves traffic in the outside lane and the entering ramp traffic, the per cent of total freeway traffic using the outside lane is a desirable parameter.

In a study[6] of forty-nine study sites located on fourteen different freeways in ten states, variables which were found to significantly affect the per cent of total freeway volume in the outside lane were freeway volume, entrance ramp volume, upstream ramp volume, distance to upstream ramp, downstream exit ramp volume, and distance to downstream exit ramp. Figure 2 shows the relationship between the per cent of the total freeway volume in the outside lane and two of the parameters -- total freeway volume and entrance ramp volume. The monographs shown in Figure 3, Figure 4, and Figure 5
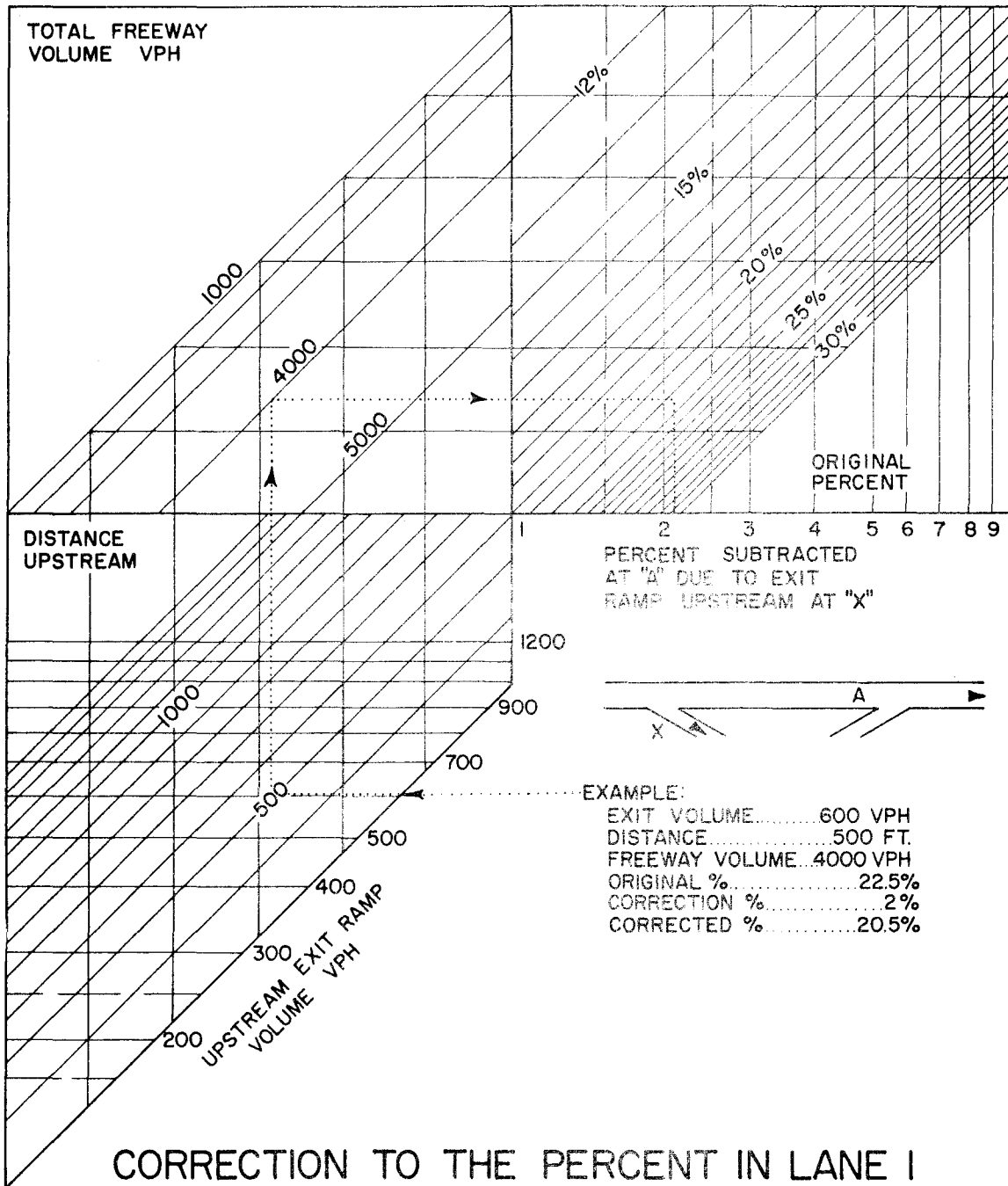
DETERMINATION OF THE RATE OF FLOW FOR THE
HIGHEST 5-MINUTE INTERVAL FROM THE RATE OF
FLOW FOR THE WHOLE PEAK HOUR

FIGURE I

FOUR LANE FREEWAYS

R=0
= 200
= 400
= 600
= 800
= 1000

SIX LANE FREEWAYS

R = 0
= 200
= 400
= 600
= 800
= 1000

EIGHT LANE FREEWAYS

R=0 TO R=1000

R = RAMP VOLUME

PERCENT OF APPROACHING FREEWAY VOLUME IN CURB LANE

50 %
40 %
30 %
20 %
10 %
0 %

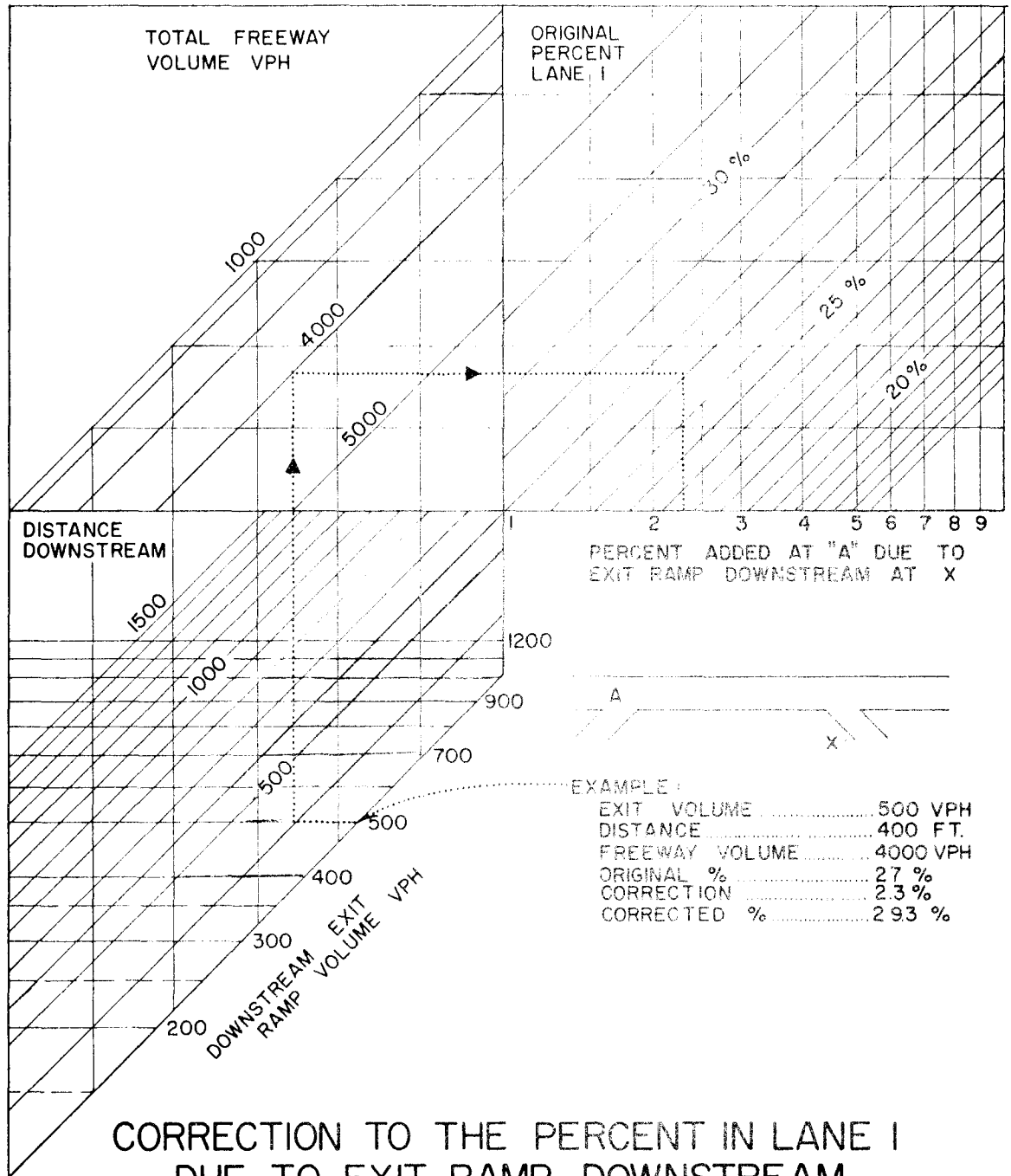1000   2000   3000   4000   5000   6000   7000   8000

FREEWAY RATE OF FLOW APPROACHING ENTRANCE RAMP

RELATIONSHIP BETWEEN PERCENT OF TOTAL FREEWAY
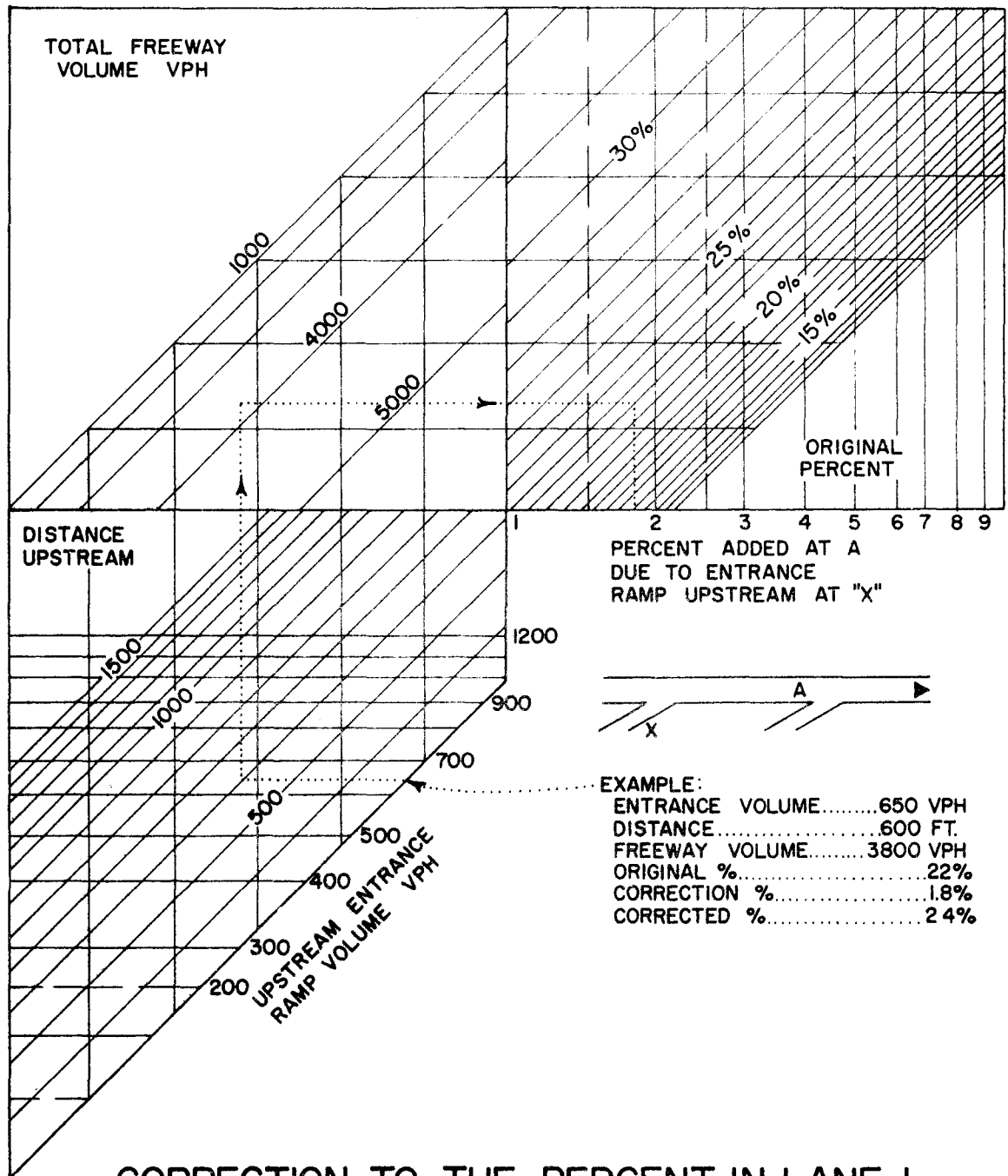VOLUME IN THE OUTSIDE LANE AND FREEWAY AND
RAMP VOLUMES

FIGURE 2

TOTAL FREEWAY
VOLUME   VPH

12%

15%

20%

25%

30%

1000

4000

5000

ORIGINAL
PERCENT

DISTANCE
UPSTREAM

1   2   3   4   5 6 7 8 9

PERCENT   SUBTRACTED
AT "A" DUE TO EXIT
RAMP UPSTREAM AT "X"

1200

1000

900

700

500

500

400

300

200

UPSTREAM EXIT RAMP
VOLUME VPH

X   A

A

EXAMPLE:
EXIT VOLUME..........600 VPH
DISTANCE................500 FT.
FREEWAY VOLUME...4000 VPH
ORIGINAL %...............22.5%
CORRECTION %.............2%
CORRECTED %..........20.5%

CORRECTION TO THE PERCENT IN LANE I
DUE TO EXIT RAMP DOWNSTREAM

FIGURE 3

TOTAL FREEWAY
VOLUME VPH

ORIGINAL
PERCENT
LANE I

30 %

25 %

20 %

1000

4000

5000

DISTANCE
DOWNSTREAM

PERCENT ADDED AT "A" DUE TO
EXIT RAMP DOWNSTREAM AT X

1500

1000

1200

900

700

500

500

400

300

200

DOWNSTREAM EXIT
RAMP VOLUME VPH

A

X

EXAMPLE:
EXIT VOLUME .... 500 VPH
DISTANCE .... 400 FT.
FREEWAY VOLUME .... 4000 VPH
ORIGINAL % .... 27 %
CORRECTION .... 2.3 %
CORRECTED % .... 29.3 %

CORRECTION TO THE PERCENT IN LANE I
DUE TO EXIT RAMP DOWNSTREAM

FIGURE 4

TOTAL FREEWAY
VOLUME VPH

DISTANCE
UPSTREAM

ORIGINAL
PERCENT

PERCENT ADDED AT A
DUE TO ENTRANCE
RAMP UPSTREAM AT "X"

EXAMPLE:
ENTRANCE VOLUME.........650 VPH
DISTANCE....................600 FT.
FREEWAY VOLUME.........3800 VPH
ORIGINAL %.......................22%
CORRECTION %..................1.8%
CORRECTED %...................2 4%

CORRECTION TO THE PERCENT IN LANE I
DUE TO ENTRANCE RAMP UPSTREAM

FIGURE 5

represent the relationships between the per cent of the total traffic in the outside
lane and the volume "on," and distance to, an upstream exit ramp, the volume
"off," and distance to, a downstream exit ramp, and the volume "on," and distance
to, an upstream entrance ramp, respectively. Distances are referenced to the
ramp nose in each case. A downstream entrance ramp was considered to have
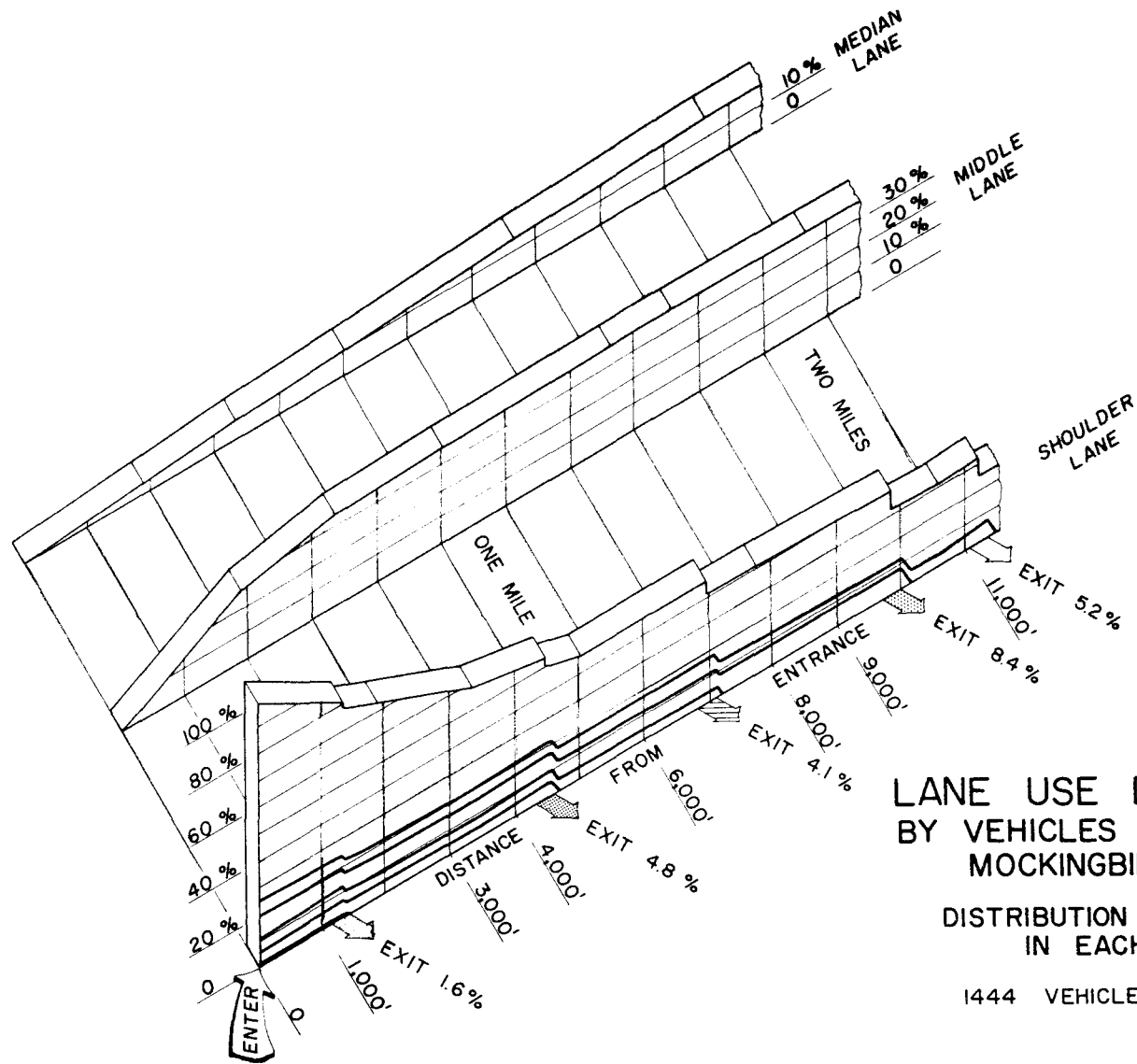no effect on the per cent of traffic in the outside lane.

Intuitively, one would think that a driver's choice of lane would be based
on more than ramp configuration and relative volumes. Drivers entering a
freeway for relatively short trips would be expected to tend to remain in the out-
side lane whereas drivers making longer trips on the freeway would be more
flexible. The results of a "Lights On" study[6] verified that a definite relation-
ship exists between trip length and outside lane utilization. Figure 6 depicts
three-dimensionally how the vehicles entering at the Mockingbird ramp on
the North Central Expressway in Dallas were distributed over the three lanes
as they travelled toward the Central Business District.

The total volume of traffic on the freeway has an effect on the lane usage
of entering traffic. As the total volume increases, entering vehicles are more
restricted to the outside lane and motorists view a temporary lane change
less worthwhile in view of the fact that another lane-change opportunity
must be found to return to the outside lane prior to exiting. Figure 7 shows the
relationship between the trip length and the per cent of the entering traffic
in the outside lane. The upper portion of Figure 7 pertains to only light total
freeway traffic volumes of less than 3000 vehicles per hour, one way. The
lower portion of Figure 7 corresponds to conditions of moderate to heavy total
freeway traffic volumes of over 3000 vehicles per hour, one way. Although
the observation points are widely spaced, the restrictive effect of the higher
volumes is noticeable. It is also apparent that vehicles travelling less than
three miles cannot be expected to reach that "steady-state" lane distribution
which is characteristic of "through" vehicles.

Determination of Service Volume

Greater dependency on motor vehicle transportation has brought about
a need for greater efficiency in traffic facilities. The ability to accommodate
vehicular traffic is a primary consideration in the planning, design, and
operation of streets and highways. It is, however, not the only consideration.
The individual motorists, for example, seldom interprets the efficiency of
a facility in terms of the volume accommodated. He evaluates efficiency
in terms of his trip -- the service to him.

The original edition of the Highway Capacity Manual[1] defined three
levels of roadway capacity -- basic capacity, possible capacity, and prac-
tical capacity. It was considered of prime importance that traffic volumes
be accurately related to local operating conditions so that particular agencies
could decide on the "practical" capacities for facilities within their juris-
diction. The manual recognized that "practical" capacity would depend on
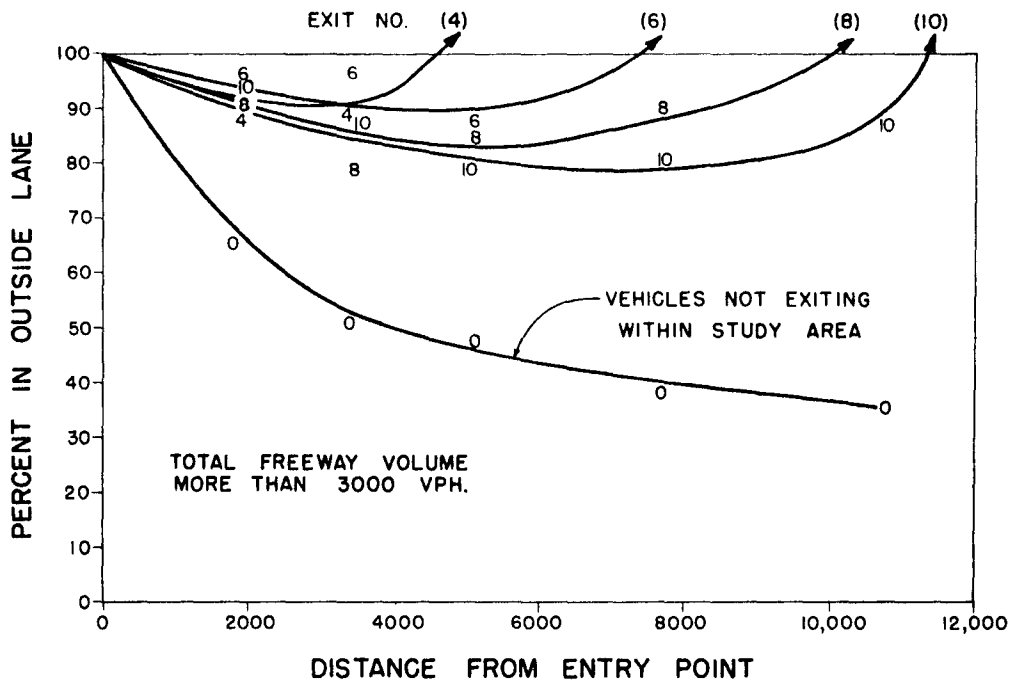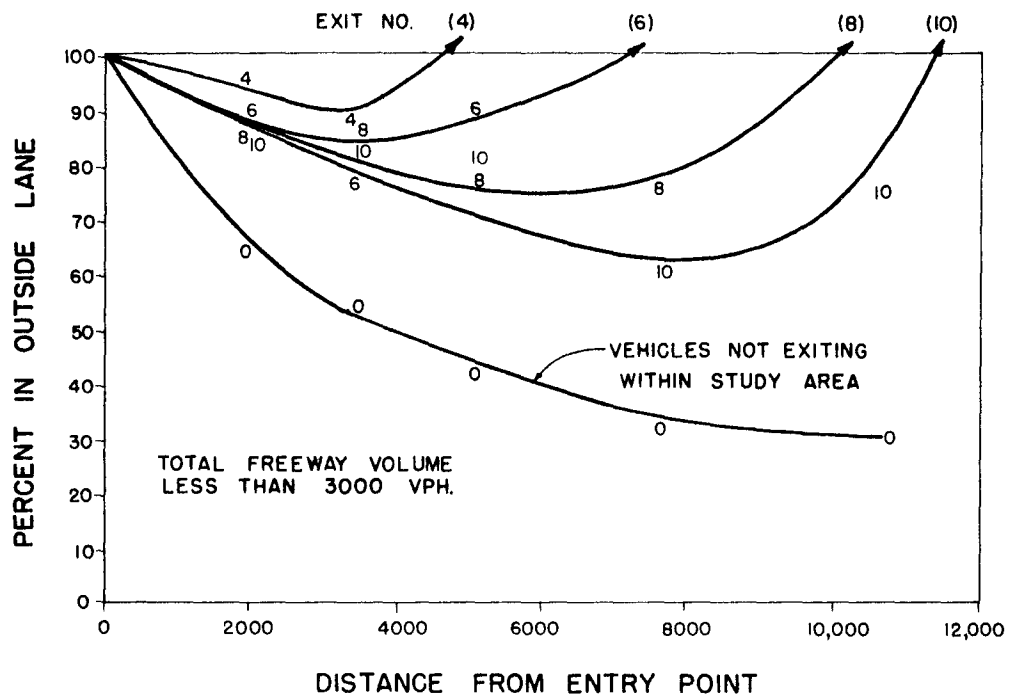the basis of a subjective evaluation of the quality of service provided.

LANE USE DISTRIBUTION
BY VEHICLES ENTERING AT
MOCKINGBIRD RAMP

DISTRIBUTION BY PERCENT
IN EACH LANE

1444 VEHICLES IN SAMPLE

FIGURE 6

OUTSIDE LANE USE RELATION
WITH TRIP LENGTHS

FIGURE 7

The present Capacity Committee of the Highway Research Board has elected, in the new edition, to define a single parameter -- possible capacity -- for each facility. Possible capacity is simply the maximum number of vehicles that can be handled by a particular roadway component under prevailing conditions. The practical capacity concept has been replaced by several specific "service volumes" which are related to a group of desirable operating conditions referred to as levels of service.

Ideally, all the pertinent factors -- speed, travel time, traffic interruptions, freedom to maneuver, safety, comfort, convenience, and economy should be incorporated in a level of service evaluation. The Committee has, however, selected speed and the service volume-to-capacity "v/c" ratio as the factors to be used in identifying level of service because "there are insufficient data to determine either the values or relative weight of the other factors listed."

Six levels of service, designated A through F from best to worst, are recommended for application in describing the conditions existing under the various speed and volume conditions that may occur on any facility. Level of Service A describes a condition of free flow; level of Service E describes an unstable condition at or near capacity; level of Service F a condition of forced flow. Levels of Service B, C, and D describe the zone of the stable flow with the upper limit set by the zone of free flow and the lower limit defined by level of Service F. Although definitive values are assigned to these zone limits for each type of highway in the new manual, no explanation is given as to how these values were obtained. This is in no way intended as a criticism since it is recognized that the function of any manual is essentially that of a handbook and therefore, should not include a methodical discussion of the facts and principles involved and conclusions reached for every value between its covers.

The authors feel that much of the designer's dilemma can be attributed to the inability to relate capacity and level of service. There is no universally accepted procedure for measuring either. If these values cannot be related quantitatively for an existing facility, there can be little hope for the designer to relate them for a facility that is still on the drafting board! In an attempt to provide a rational explanation of this capacity-level of service relationship an energy-momentum model based on the hydrodynamic analogy of a one-dimensional compressible fluid has been developed.[7, 8, 9, 10] Some aspects of this model will be discussed briefly.

A single stream of traffic offers a striking analogue to the flow of a compressible fluid in a constant area duct. Both consist of discrete particles: individual molecules in the case of a fluid, and individual vehicles in the case of the traffic stream. Just as the fluid equation of state can be derived from the microscopic law of interaction of two molecules, the traffic equation of state can be derived from the car-following law governing the motion of two cars.[11] The correspondence between the classical hydrodynamic and

the traffic system is summarized in Table 1.

Because, in a classical system, the conservation of momentum equation serves to establish the form for momentum, the quantity ku is defined here as the "momentum" of the traffic stream. It is apparent that "momentum" is equivalent to traffic flow and therefore the flow oriented parameters (u_m, k_m, and q_m) in Table 1 are based on optimizing this momentum (setting $dq/du = dq/dk = 0$ to obtain u_m and k_m).

It is well known that the kinetic energy of a fluid is $(1/2) \rho v^2$. However because of the generalized equation of motion utilized in the formulation of the traffic system model, the "kinetic energy" of the traffic stream will be defined as $E = \alpha k u^2$ where α is a constant. Differentiating E with respect to concentration and speed, $dE/dk = dE/du = 0$, gives the appropriate energy parameters (u'_m, k'_m, and q'_m).

In Table 2, the service volume ranges for Levels of Service A through F are described in terms of the energy-momentum parameters. The divisions between (1) free and stable flow and (2) between unstable flow and forced flow are defined by the two points obtained by equating kinetic energy to internal energy (see Figure 8). The dividing point between stable and unstable flow is obtained by either maximizing the kinetic energy or minimizing the kinetic energy of the stream. These three points serve to establish the four level of service zones defined by the revised Highway Capacity Manual.

The significance of Figure 8 is that it provides a rational basis for defining level of service and relating it to the other traffic variables -- speed, flow and density. The relationship between level of service and traffic volume (flow) is analogous to the relationship in classical hydrodynamics between energy and momentum. Efficiency in a classical system is measured by the ratio of useful energy to total energy or E/T. Optimum operation occurs when lost energy I is at a minimum. In a traffic system, this concept of efficiency is manifest by maximizing the kinetic energy of the stream as a whole and minimizing the acceleration noise of the individual vehicles (internal energy).

Application to Design and Operation

The level of service approach establishing levels of operation from free flow to capacity which has been established in the revised Capacity Manual is designed to allow the engineers and administrators to provide the highest level of service economically feasible. The momentum-energy analogy derived in the previous section is an effort to explain the capacity-level of service relationship rationally and quantitatively. It must be recognized, however, that highway traffic represents a stochastic phenomenon. Therefore, any high-
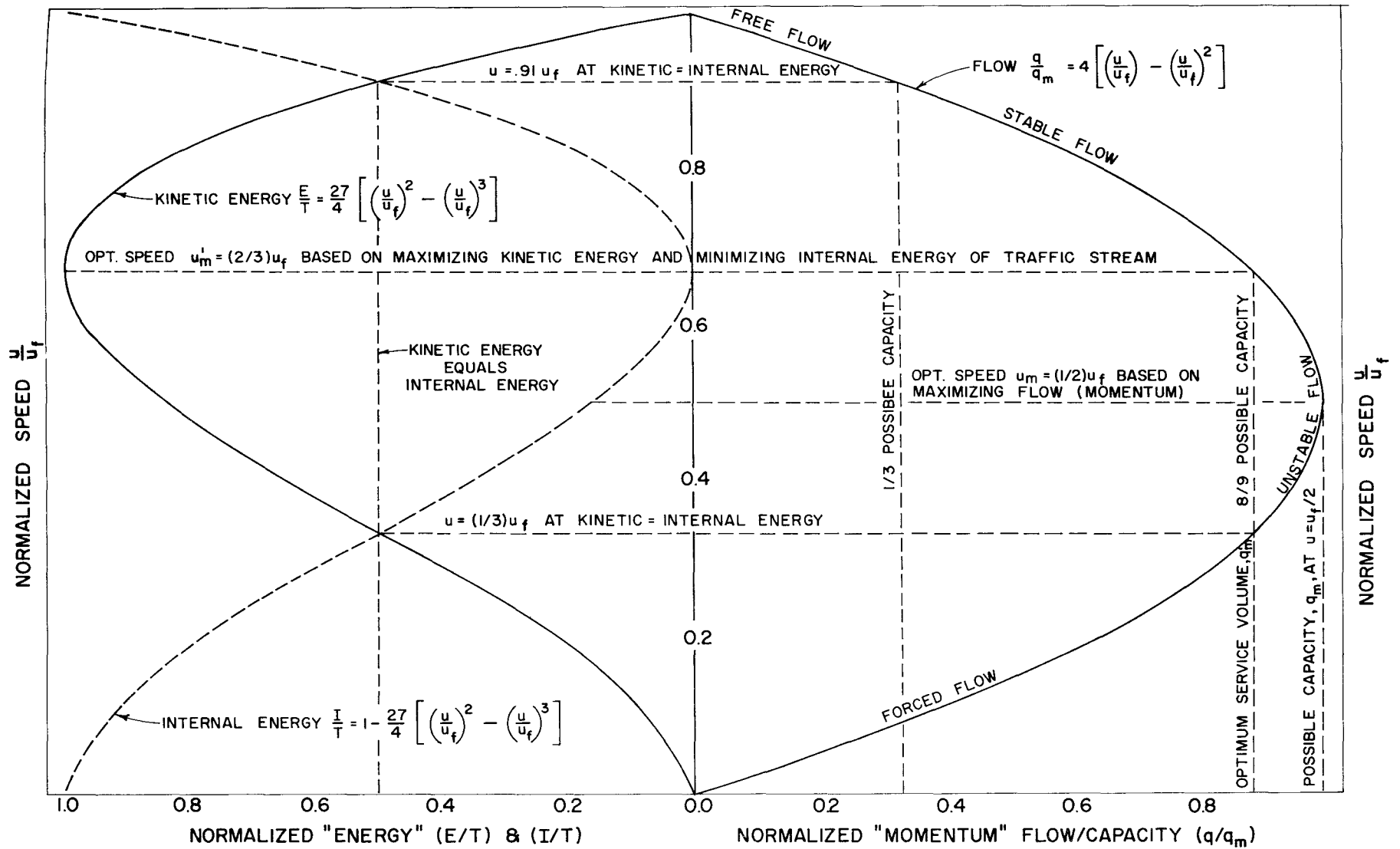
Characteristic	Hydrodynamic System	Traffic System
Continuum Discrete Unit	1-dimensional compressible fluid Molecule	Single-lane traffic stream Vehicle
Variables	Mass density, ρ Velocity, v	Concentration, k Speed, u

		Hydrodynamic System	Traffic System
Equations	Continuity	$\dfrac{\partial \rho}{\partial t} + \dfrac{\partial(\rho v)}{\partial x} = 0$	$\dfrac{\partial k}{\partial t} + \dfrac{\partial(ku)}{\partial x} = 0$
	Motion	$\dfrac{dv}{dt} + \dfrac{c^2}{\rho}\dfrac{\partial \rho}{\partial x} = 0$	$\dfrac{du}{dt} + c^2 k^n \dfrac{\partial k}{\partial x} = 0$
	Momentum	$\dfrac{\partial(\rho v)}{\partial t} + \dfrac{\partial(\rho v^2 + \rho c^2)}{\partial x} = 0$	$\dfrac{\partial(ku)}{\partial t} + \dfrac{\partial(ku^2 + k^{n+2}c^2/n+2)}{\partial x} = 0$
	State	$P = cPT$	$q = ku$

Parameters	Critical velocity, v_c Critical flow, Q_c Shock wave velocity, U Momentum, ρv Kinetic energy, $\rho v^2/2$ Internal energy, E Friction factor, f	Critical speed, u_m Critical concentration, k_m Capacity flow, q_m Shock wave velocity, U Flow, ku Kinetic Energy, αku^2 Optimum speed, u'_m Optimum concentration, k'_m Optimum flow, q'_m Internal energy, σ Natural noise, σ_N

TABLE 2

LEVELS OF SERVICE AS ESTABLISHED BY ENERGY-MOMENTUM CONCEPT

Level of Service Zone		Zone Limits Upper (See Figure 8)	Lower	Description
Free Flow	A	u_f	$.91u_f, .35q_m$	Speeds controlled by driver desires and physical roadway conditions. This is the type of service expected in rural locations.
Stable Flow	B	$.91u_f, .35q_m$	$.83u_f, .55q_m$	Speed primarily a function of traffic density.
	C	$.83u_f, .55q_m$	$.75u_f, .75q_m$	The conditions in this zone are acceptable for freeways in suburban locations.
	D	$.75u_f, .75q_m$	u'_m, q'_m	The conditions in this zone are acceptable for urban design practice. The lower limit u'_m, q'_m represents the critical level of service.
Unstable Flow	E_1	u'_m, q'_m	u_m, q_m	A small increase in demand (flow) is accompanied by a large decrease in speed leading to high densities and internal friction.
	E_2	u_m, q_m	$.33u_f, q'_m$	This type of high density operation can not persist and leads inevitably to congestion.
Forced Flow	F	$.33u_f, q'_m$	0	Flows are below capacity and storage areas consisting of queues of vehicles form. Normal operation is not achieved until the storage queue is dissipated.

FREE FLOW

u = .91 u_f AT KINETIC = INTERNAL ENERGY

FLOW $\frac{q}{q_m} = 4\left[\left(\frac{u}{u_f}\right) - \left(\frac{u}{u_f}\right)^2\right]$

STABLE FLOW

KINETIC ENERGY $\frac{E}{T} = \frac{27}{4}\left[\left(\frac{u}{u_f}\right)^2 - \left(\frac{u}{u_f}\right)^3\right]$

OPT. SPEED $u'_m = (2/3)u_f$ BASED ON MAXIMIZING KINETIC ENERGY AND MINIMIZING INTERNAL ENERGY OF TRAFFIC STREAM

KINETIC ENERGY
EQUALS
INTERNAL ENERGY

OPT. SPEED $u_m = (1/2)u_f$ BASED ON
MAXIMIZING FLOW (MOMENTUM)

1/3 POSSIBEE CAPACITY

8/9 POSSIBLE CAPACITY

UNSTABLE FLOW

u = (1/3)u_f AT KINETIC = INTERNAL ENERGY

0.8

0.6

0.4

0.2

INTERNAL ENERGY $\frac{I}{T} = 1 - \frac{27}{4}\left[\left(\frac{u}{u_f}\right)^2 - \left(\frac{u}{u_f}\right)^3\right]$

FORCED FLOW

OPTIMUM SERVICE VOLUME, q_m

POSSIBLE CAPACITY, q_m, AT u=u_f/2

NORMALIZED SPEED $\frac{u}{u_f}$

NORMALIZED SPEED $\frac{u}{u_f}$

1.0   0.8   0.6   0.4   0.2   0.0   0.2   0.4   0.6   0.8

NORMALIZED "ENERGY" (E/T) & (I/T)

NORMALIZED "MOMENTUM" FLOW/CAPACITY (q/q_m)

QUANTITATIVE APPROACH TO LEVEL OF SERVICE USING THE "TOTAL
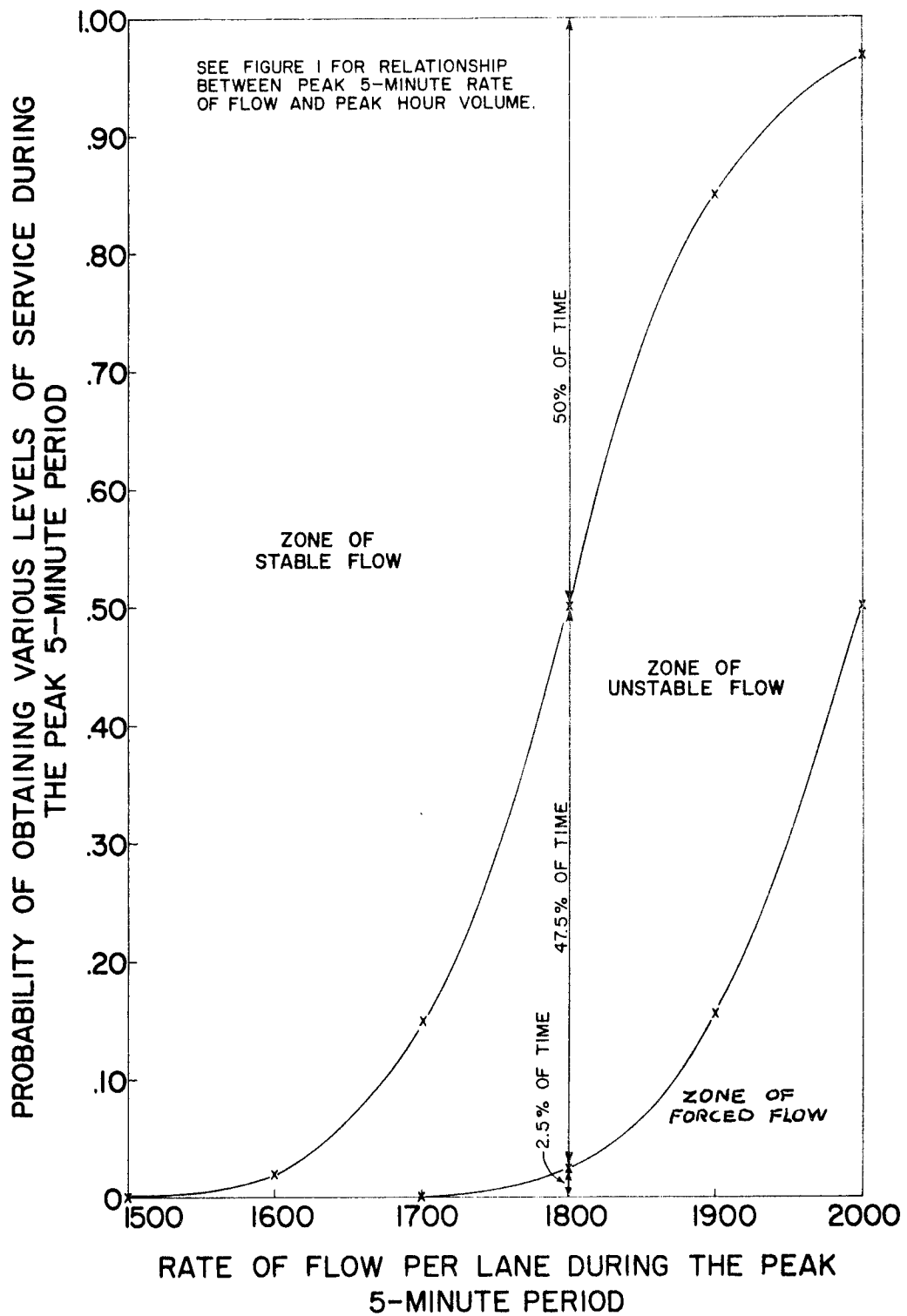ENERGY" – MOMENTUM ANALOGY

FIGURE 8

way facility, designed to accommodate traffic, must be designed with the realization that it is highly probable from time to time demand will exceed capacity.

Since congestion may last much longer than that interval in which demand exceeds capacity, it is important that precautions be taken to prevent this. Based on a stipulated rate-of-flow for a 5-minute period, a designer can ensure that congestion will not occur to whatever degree of confidence desired. This is illustrated by the design curves relating level of service to a 5-minute rate of flow (Figure 9). Thus, assuming a possible capacity of 2000 vph per lane, a service volume of 1800 has a probability of .50 of guaranteeing "stable flow" during the peak 5-minute period. On the other hand, there is a 50% chance of "unstable flow" occurring; and a 2.5% chance of "forced flow" can lead to congestion, due to the statistical variability of vehicle headways. It is interesting to note that a relatively small reduction of 100 in the service volume, to a flow of 1700 vph, greatly increases the probability of maintaining "stable flow." These curves represent an attempt to put such a decision in the hands of highway administrators and designers. After this choice is made, the service volume to be used for design for the peak hour may be obtained from Figure 1, depending on the population of the city.

Figure 9 was obtained by determining the probability of getting observed rates of flow greater than the predicted values shown in Figure 1 based on a standard deviation of 5% in the normally distributed error. The determination of freeway design service volumes is summarized in Table 3. The first four columns are taken directly from Figure 9; the last four columns utilize the peaking relationships expressed in Figure 1. The organization of Table 3 is useful in that it provides the designer with confidence limits in determining the number of main lanes needed on a freeway.

After the determination of the number of freeway lanes, the operating conditions at critical locations of the freeway must be investigated for the effect on capacity and level of service. Unless some designated level of service is met at every point on the freeway, bottlenecks will occur and traffic operation will break down. Critical locations on a freeway are manifest by either sudden increases in traffic demand, the creation of inter-vehicular conflicts within the traffic stream, or a combination of both.

The traffic demand on a freeway can only change at entrance or exit ramps. Two of the most critical points on a freeway will be upstream from an exit ramp and downstream from an entrance ramp, where traffic demand will necessarily be at a maximum. Operating conditions at exit ramps are generally similar to the operating conditions described at an upgrade, but can be much more severe where there is a backup from the exit ramp onto the main roadway proper. Many exit ramps problems could be avoided by providing for the speed reduction on the ramp rather than on the shoulder lane of the freeway. Even

PROBABILITY OF OBTAINING VARIOUS LEVELS OF SERVICE DURING THE PEAK 5-MINUTE PERIOD

RATE OF FLOW PER LANE DURING THE PEAK 5-MINUTE PERIOD

SEE FIGURE I FOR RELATIONSHIP BETWEEN PEAK 5-MINUTE RATE OF FLOW AND PEAK HOUR VOLUME.

ZONE OF STABLE FLOW

ZONE OF UNSTABLE FLOW

ZONE OF FORCED FLOW

50% OF TIME

47.5% OF TIME

2.5% OF TIME

DESIGN CURVES RELATING LEVEL OF SERVICE TO FLOWS DURING THE PEAK 5-MIN.

FIGURE 9

TABLE 3

FREEWAY CAPACITY WITH CONFIDENCE LIMITS

Peak 5-Min Flow (VPH)	Approx Probabilities of Various Types of Flow in Peak 5-Min			Freeway Design Service Volume (Total Hourly Vol./Lane) Population of Metropolitan Area			
	Stable	Unstable	Forced	100,000	500,000	1,000,000	5,000,000
1500	1.00	0.00	0.00	1100	1200	1300	1300
1600	0.98	0.02	0.00	1200	1300	1300	1400
1700	0.85	0.15	0.00	1300	1400	1400	1500
1800	0.50	0.48	0.02	1400	1500	1500	1600
1900	0.15	0.69	0.16	1500	1600	1600	1700
2000	0.03	0.47	0.50	1500	1600	1700	1800

where long parallel deceleration lanes are provided, they are not used because of the unnatural maneuver involved. Unfortunately, the close spacing of interchanges and use of frontage roads favor the use of short slip-type ramps. Where a high exit volume slip ramp is used, definite consideration should be given to placing yield signs on the frontage roads, or streets into which the ramp exits, thus preventing back-up from the exit ramp onto the freeway.

Entrance ramps may create two potential conflicts with the maintenance of the adopted level of service of a roadway section. First, the additional ramp traffic may cause operation changes in the outside lane at the merge. This condition, of course, will be aggravated by any adverse geometrics, such as high angle of entry, steep grades, and poor sight distance. Second, the additional ramp volume may change the operating conditions across the entire roadway downstream from the on-ramp. This is particularly true where there is a downstream bottleneck.

The last "critical location" to be considered is the weaving section. Weaving sections often simplify the layout of interchanges and result in right-of-way and construction economy. The capacity of a weaving section is dependent upon its length, number of lanes, running speed and relative volumes of individual movements. When large volume weaving movements occur during peak hours, approaching the possible capacity of the section, probable results are traffic stream friction, reduced speeds of operation, and a lower level of service. This can sometimes be avoided by the use of additional structures to separate ramps, reversing the order of ramps so as to place the critical weaving volumes on frontage roads, and the use of collector-distributor roads in conjunction with cloverleaf interchanges.

Ramp weaving sections should be designed, checked and adjusted so that their capacity is greater than the service volume used as the basis for design. This is consistent with the level of service concept used in determining the number of main lanes and checking the merging capacities at entrance ramps. The determination of minimum length of weaving section between ramps to meet the controlling level of service is illustrated in Figure 10. These relationships were obtained by considering the outside lane use relation with trip length (Figure 6 and 7). Referring to Figure 10, the maximum number of vehicles that an exit, R_2, cannot exceed, $Q-R_1$, plus the number of entrance ramp vehicles that change lanes within the merging section.

The freeway motorist expects to have his needs anticipated and fulfilled to a much higher degree than on conventional roads. Hopefully, this expectation would be fulfilled by the application of level of service considerations to rational geometric design. More often than not, however, actual traffic and travel patterns differ from the projected values making constant freeway operation attention after construction a must.

DETERMINATION OF MINIMUM LENGTH OF WEAVING SECTION TO MEET THE
DESIGN LEVEL OF SERVICE

FIGURE 10

The significance of the energy-momentum model as applied to freeway operations is that if any two of the traffic variables -- density k, speed u, flow q or acceleration noise σ -- are measured, the following traffic parameters can be calculated: k_m, u_m, q_m, k'_m, u'_m, and q'_m. The determination of these parameters enables the engineer to document operation on an entire freeway over a sustained period of time. Such a description could be obtained by measuring speed and density from aerial photos[6] (Figure 11 and 12), or speed and acceleration noise with a test vehicle equipped with a recording speedometer (Figure 13).[7] The contour maps are for the Gulf Freeway Surveillance and Control Project, illustrating how operation during the entire morning peak can be described rationally for a 6-mile high volume urban freeway.

SIGNALIZED INTERSECTIONS

The at-grade arterial street system is a vital part of an urban area's transportation facilities. From a capacity viewpoint the signalized intersection is the key element of this system and thus the item of main consideration. Since signalization is required at the intersection points of two at-grade arterials and often at freeway-arterial intersections, the signalized intersection can often be the major bottleneck in a transportation system. It is, therefore, extremely important to utilize a high degree of capacity technology in the planning and design of such facilities.

Planning

The planning process requires the evaluation of existing facilities and a determination of their traffic carrying ability. The Highway Capacity Manual provides a good basic guide for making such studies. The manual provides extensive coverage of various types of intersections which operate under different conditions of parking restrictions, turn demands and street widths. This material makes it possible to determine a reasonable estimate of the traffic carrying ability of almost any existing intersection.
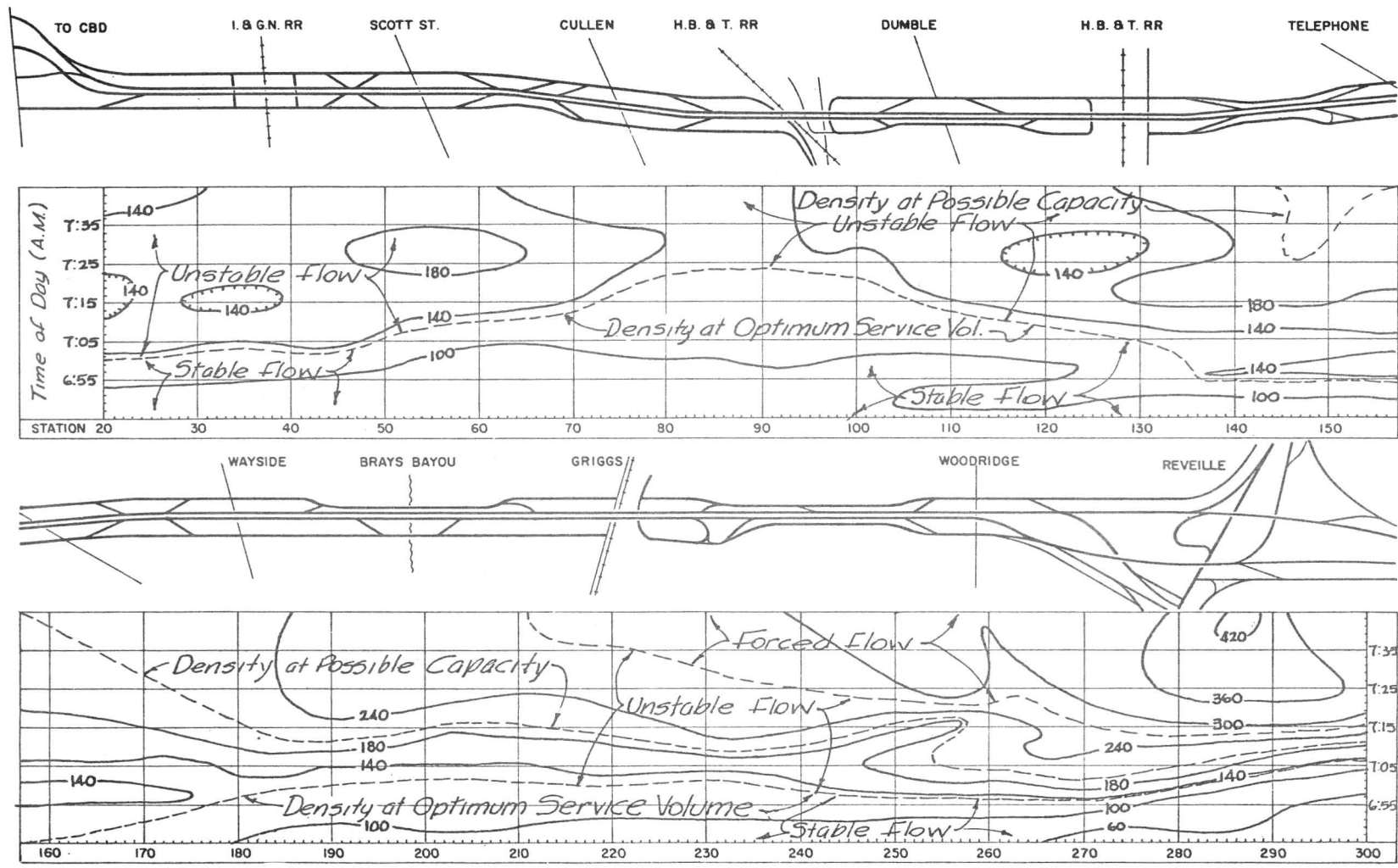
Intersection Design

Since the Highway Capacity Manual deals with many variable conditions it is difficult to apply this procedure to the design of a new facility. Since most new facilities where signalization and capacity are a problem are of the major arterial-major arterial or freeway-major arterial type the most pertinent design problem concerns what might be termed a "high-type" facility. That is, a facility where vehicle conflicts are time separated, pedestrian conflicts and parking are minor considerations and where effective lane designations exist on all approaches. It is extremely important to be able to adequately design this type of intersection or to develop modifications or redesigns which assure that the capacity provided is sufficient to meet traffic demands.
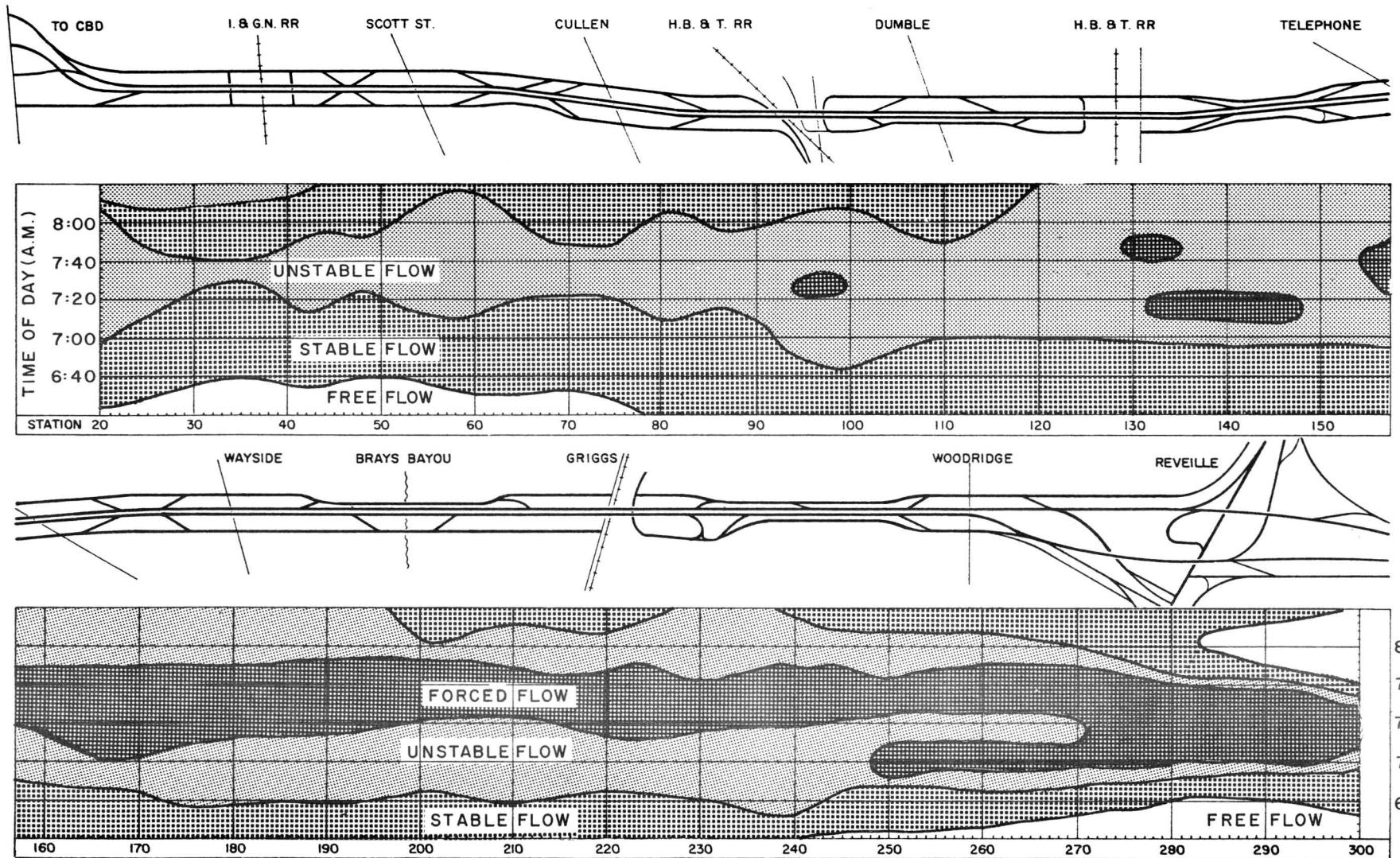
SPEED CONTOURS (TOTAL INBOUND TRAFFIC)

FIGURE 11

DENSITY CONTOURS (3 LANE TOTAL)

FIGURE 12

LEVEL OF SERVICE CONTOURS
TUESDAY – INBOUND

FIGURE 13

Capacity-Design Procedure

The Texas Transportation Institute has developed a capacity-design procedure which has proven to be simple in application and very effective in the results produced. This technique was developed through a research effort sponsored by the Texas Highway Department and the U.S. Bureau of Public Roads and was reported in articles by Pinnell, Drew, and Capelle.[12, 13]

The technique was best formulated by Drew[13] considering the time-space diagram shown in Figure 14. The formulation is as follows:

$$\Sigma V = \frac{3600}{C} \left[\frac{c + \Sigma O - \phi(K-D)}{D} \right] \qquad (1)$$

Where the symbols are as defined in Figure 14.

The important concept here is that the volume represented is the highest single lane volume (critical volume) from each phase movement. The summation of the critical volumes from each phase movement (ΣV) yields a capacity which is related to cycle length and intersection characteristics as indicated in the formulation.
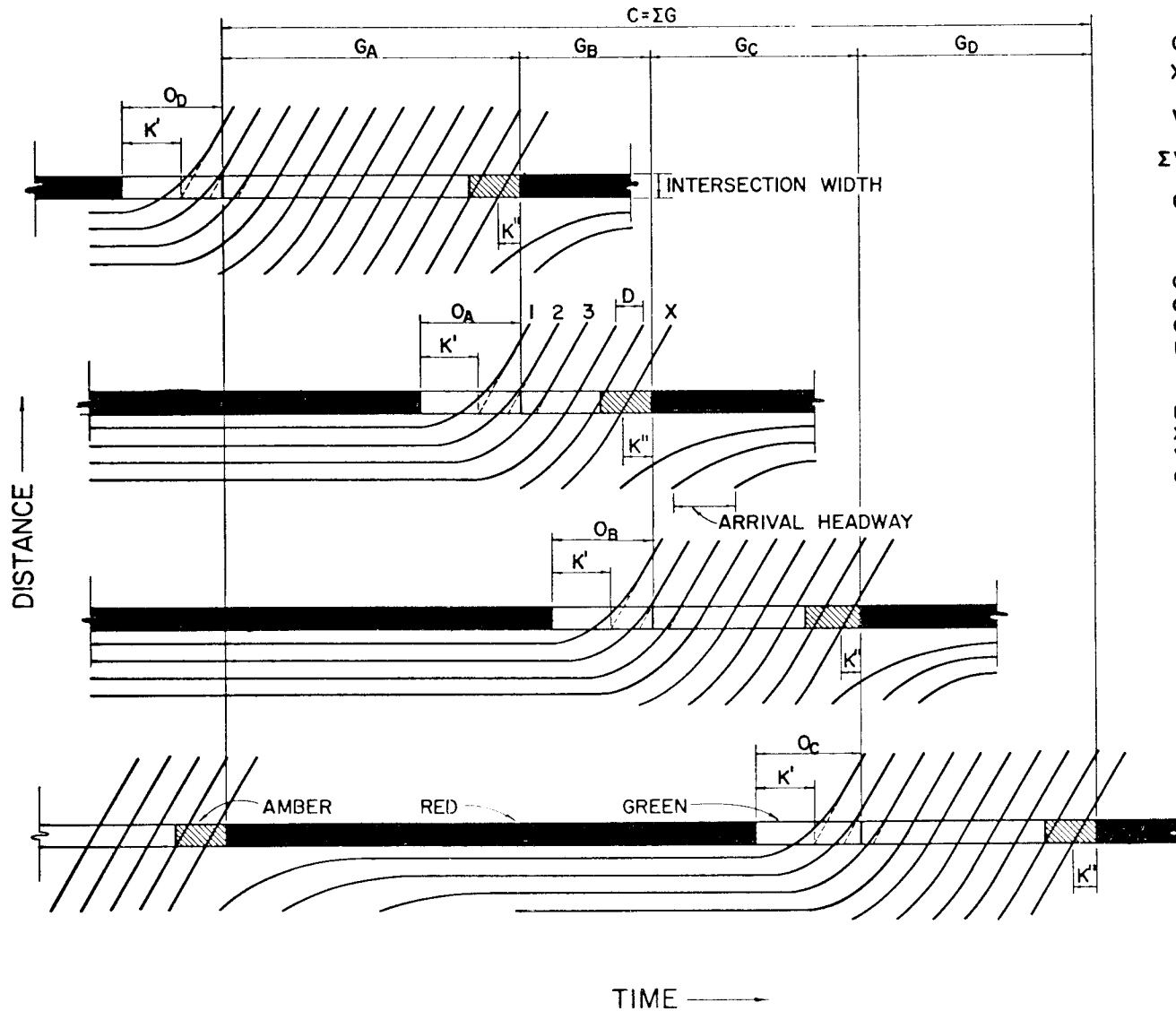
If the phase movements cannot be overlapped (two phase movements moving simultaneously) then the term ΣO drops out. This is the case for most standard intersections. However, if the phase movements can be overlapped as in the case of diamond interchanges then ΣO would have a value and add to the capacity.

Introducing the values of K = 6.0 seconds, D = 2.0 seconds and ϕ =4 and rearranging equation 1 the following is obtained:

$$\Sigma V = 1800 - 16 \left(\frac{1800}{C} \right) + \Sigma O \left(\frac{1800}{C} \right) \qquad (2)$$

This equation makes it somewhat easier to visualize the formulation. The first term of equation (2) represents the hourly capacity of a single lane moving with vehicle spacings of one vehicle every two seconds. The second term represents the reduction in hourly capacity resulting from starting delays and amber termination. The third term represents the increase in hourly capacity that can be obtained from overlapping phases if used.

With no overlap it can be seen that the limiting critical capacity is 1800 vehicles per hour and that capacity increases with an increase in cycle length. With overlap it is possible to go above 1800 veh./hr. if ΣO > 16 and the capacity increases with a decrease in cycle length.

- 29 -

$$C = \Sigma G$$

G_A G_B G_C G_D

O_D

K'

INTERSECTION WIDTH

K"

O_A 1 2 3 D X

K'

K"

ARRIVAL HEADWAY

O_B

K'

K'

O_C

K'

AMBER   RED   GREEN

K'

DISTANCE

TIME

$$G + O = (X-1)D + K$$

$$X = \frac{G + O - (K-D)}{D}$$

$$V = \frac{3600}{C} X$$

$$\Sigma V = \frac{3600}{C}\left[\frac{(C+\Sigma O) - \phi(K-D)}{D}\right] \quad \text{EQ.}(1)$$

$$C = \frac{3600[\phi(K-D) - \Sigma O]}{3600 - D\Sigma V} \quad \text{EQ.}(2)$$

G = PHASE LENGTH
C = CYCLE LENGTH
O = OVERLAP PHASE
K = K' + K" = TOTAL TIME LOST / PHASE
   K' = LOST GETTING QUEUE IN MOTION
   K" = CROSSING THE INTERSECTION
D = CONSTANT DEPARTURE HEADWAY
X = LANE CAPACITY PER PHASE
V = CRITICAL LANE VOLUME
ϕ = NO. PHASES PER CYCLE

TIME-SPACE RELATIONSHIP FOR MULTI-PHASE SIGNAL SYSTEMS

FIGURE 14

Design Example

The utilization of the above procedure and the "critical lane" concept are best illustrated through the use of an example design problem. For this purpose, an example using a typical major arterial intersection with no phase overlap is developed.

Example I - Major Arterial Intersection --- The capacity-design procedure of a high-type major arterial intersection can best be described as a step-by-step procedure. This procedure starts with 24-hour volumes as furnished in an urban planning study and moves to a specific lane design. Figure 15 and 16 illustrate steps of the procedure. These steps are as follows:

Step 1 in the design procedure is to determine the three conditions (population, location, and volumes) which affect the magnitude of the peak period. See Figure 15.

In Step 2 the peak hourly volumes are determined from the ADT (Average Daily Traffic). This must be accomplished for the AM and PM peaks. For the purpose of this example, only the PM peak will be considered. See Figure 15.

Step 3 consists of calculating the peak magnitude factor for each approach. See Figure 15.

In Step 4 the peak magnitude factors are applied to the peak hour approach volumes to determine an hourly rate of flow which has been adjusted to reflect the arrivals during the peak period. See Figure 15.

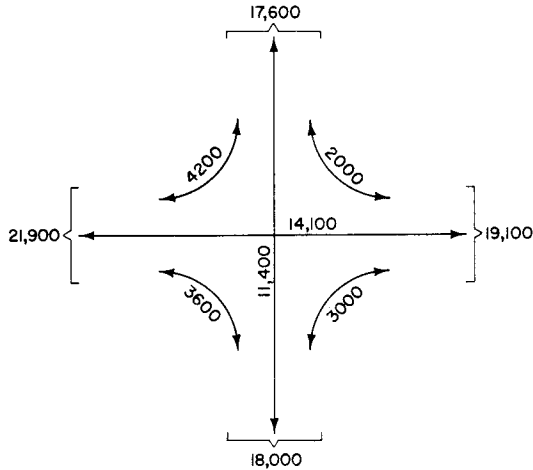In Step 5 all conflicting movements are separated by the signal phasing as shown in Figure 16.

Step 6 consists of testing various design combinations by varying the number of lanes on each approach in order to arrive at a desirable design. Volumes are assigned to each lane assuming equal lane distribution during the peak period. The maximum lane volume required to move on a given phase is called the critical lane volume. The sum of these critical lane volumes for all phases provides the basis for calculating the minimum cycle length using equation (2) transformed into

$$C = \frac{57,600}{3,600 - 2.0 \, \Sigma V}$$

A first design combination might be two lanes and a left turn slot on each approach. The critical lane volumes and the sum of the critical lane volumes (ΣV) are shown in Figure 16.
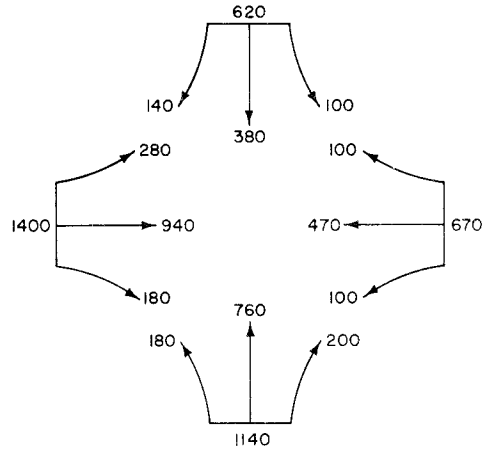
STEP 1: LIST CONDITIONS

(1) High-Type Intersection
(2) Population of City (1981)=280,000
(3) Location of Intersection (1981)
    distance from CBD = 4.0 miles
    distance from City Limits = 2.6 miles

17,600

4200   2000

21,900 ◁   14,100   ▷19,100

11,400

3600   3000

18,000

STEP 2: FIND PEAK HOUR VOLUMES

K=10 % (Peak Hour Factor)
D=67 % (Directional Distribution)

620

140   100

280   380   100

1400   940   470 ◀   670

180   760   100

180   1140   200

NOTE: Only the P.M. peak is considered for the
purpose of this example.

STEP 3: FIND PEAK MAGNITUDE FACTOR
FOR EACH APPROACH

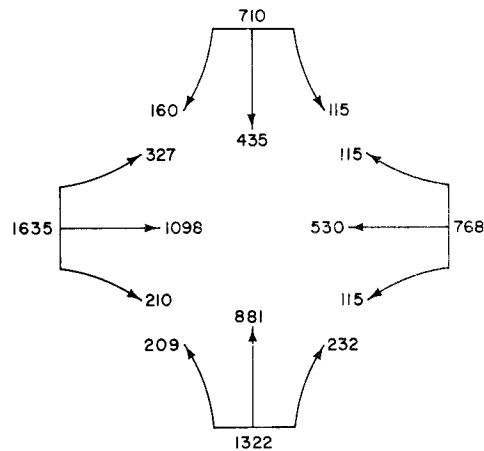$\hat{Y}'=1.225-.000135X_1\pm(0.1X_2'-.00003X_3)$

Where X_1 (pop.÷1000)= 280
       X_2'(ratio dist.) = 4.0÷(4.0+2.6)=6.1
       X_3(south approach)= 1140
       X_3(west approach)= 1400
       X_3(north approach)= 620
       X_3(east approach)= 670

P.M. Peak:
    $\hat{Y}$ (P.M.)=1.225-.038-.016+.00003X_3
        $\hat{Y}'$(south approach)=1.160
        $\hat{Y}'$(west approach)=1.168
        $\hat{Y}'$(north approach)=1.145
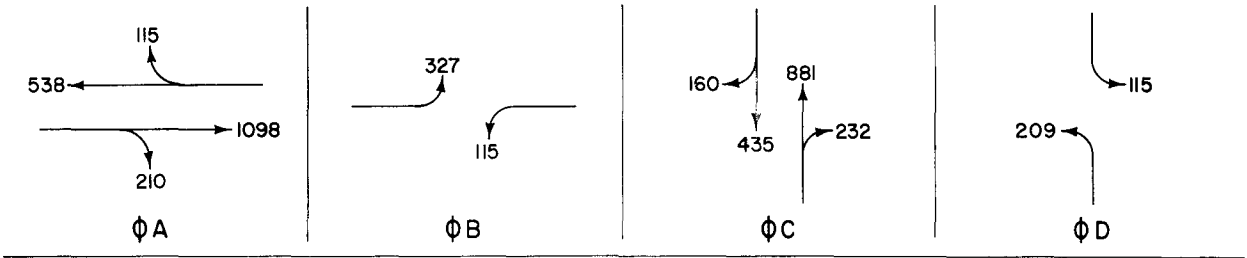 $\hat{Y}'$(east approach)=1.146
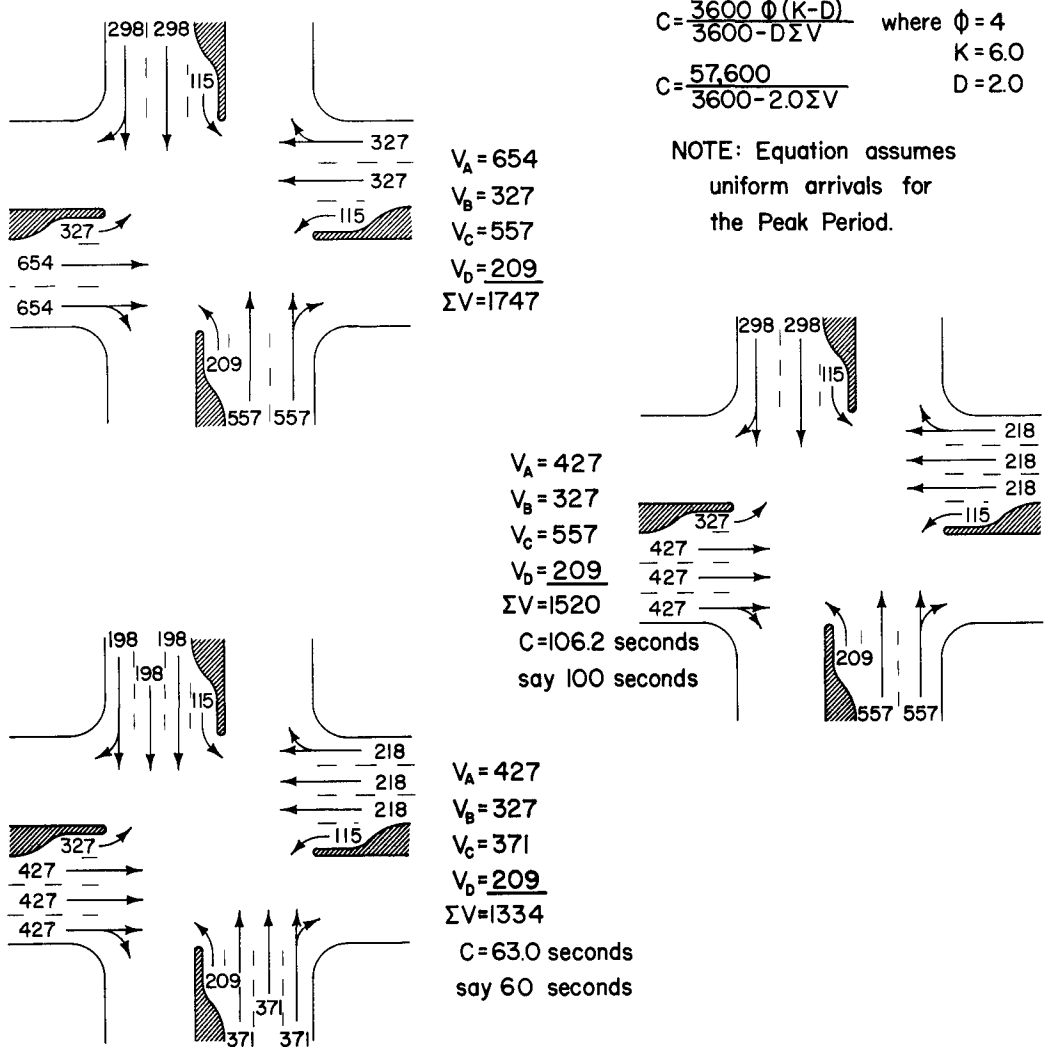
STEP 4: SHOW ADJUSTED HOURLY RATES
OF FLOW FOR PEAK PERIOD

710

160   115

327   435   115

1635   1098   530 ◀   768

210   881   115

209   1322   232

CAPACITY-DESIGN PROCEDURE
(STEPS 1 TO 4)
FIGURE 15

STEP 5: ASSUME PHASING

φA ... φB ... φC ... φD

115
538◄
────► 1098
210

327
115

160◄ 881
435 ►232

115
209◄

φA   φB   φC   φD

STEP 6: ASSUME VARIOUS LANE COMBINATIONS. DETERMINE CRITICAL LANE VOLUMES (V)
AND MINIMUM CYCLE LENGTHS (C).

298│298
115

327
327
115

327
654 ────►
654 ────►

209
557│557

$V_A = 654$
$V_B = 327$
$V_C = 557$
$V_D = \underline{209}$
$\Sigma V = 1747$

$$C = \frac{3600\ \phi\,(K-D)}{3600 - D\Sigma V} \quad \text{where } \phi = 4$$

$$C = \frac{57,600}{3600 - 2.0\Sigma V}$$

where $\phi = 4$
$K = 6.0$
$D = 2.0$

NOTE: Equation assumes
uniform arrivals for
the Peak Period.

$V_A = 427$
$V_B = 327$
$V_C = 557$
$V_D = \underline{209}$
$\Sigma V = 1520$
$C = 106.2$ seconds
say 100 seconds

298│298
115

327
427 ────►
427 ────►
427 ────►

218
218
218
115

209
557│557

198│198
198
115

218
218
218
115

327
427 ────►
427 ────►
427 ────►

209
371
371│371

$V_A = 427$
$V_B = 327$
$V_C = 371$
$V_D = \underline{209}$
$\Sigma V = 1334$
$C = 63.0$ seconds
say 60 seconds

CAPACITY-DESIGN PROCEDURE
(STEPS 5 AND 6)
FIGURE 16

To accommodate this demand the cycle length must be over 500 seconds. This implies an inadequate number of lanes for the volumes involved and therefore other designs must be considered.

A desirable design combination might be three lanes and a left turn bay on each approach (Figure 16). The summation of the critical lane volumes is $\Sigma V = 1334$ and the cycle length is calculated to be 63 seconds, a very reasonable figure. This cycle length may be rounded to a 65 second cycle since an increase in cycle length increases capacity in this case (no overlap).

In Step 7 the average arrivals per cycle (m) are calculated from the second design's critical lane volumes so that the phase lengths may be determined. This is accomplished by entering the graph of Poisson curves (Figure 17) with the value of "m" to determine the phase lengths (G) for the various probabilities of failure (P). Any combination of $G_A + G_B + G_C + G_D$ that equals the assumed cycle length of 65 seconds is acceptable. In this design the following phasing with C = 65 seconds was found to be satisfactory:
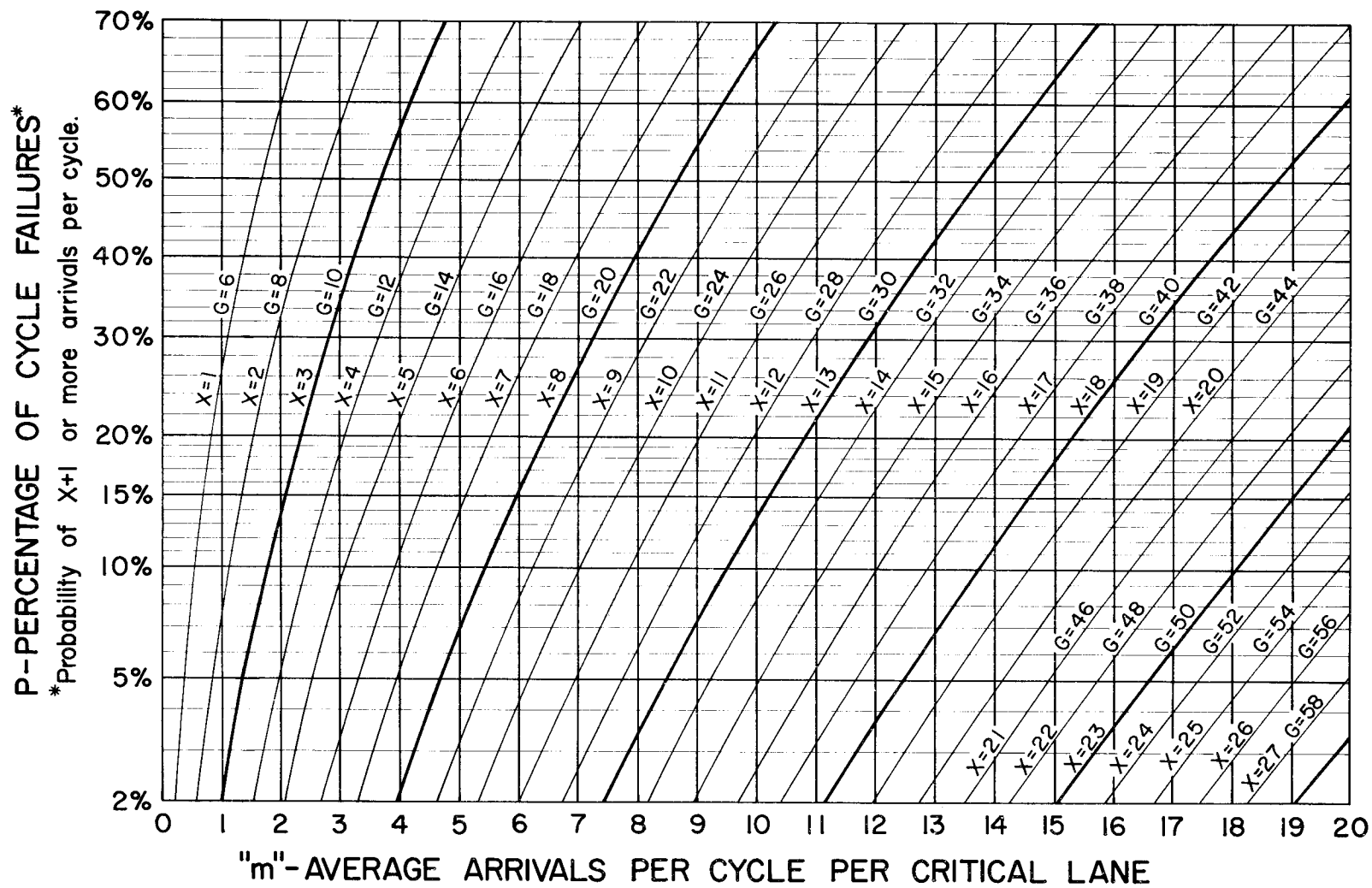
C = 65 seconds

Phase 0	Avg. Arrivals per Cycle "m"	Per Cent of Failure	Phase Length
A	7.7	35%	20
B	5.9	35%	16
C	6.7	35%	18
D	3.7	35%	11

G = 65 seconds OK

Thus the design of the intersection is complete. At this stage, the designer has determined (1) the number of lanes required on each approach, (2) the cycle length to be utilized and (3) the phasing sequence and phase times. This produces what might be termed an operational design since all aspects of the final field operation have been considered in the capacity-design procedure.

Intersection Level of Service

Another major advantage of the capacity-design procedure just reviewed is its versatility in yielding various design or operating conditions which can be related to a predicted level of service. The principal figure of merit is the probability of cycle failures which indicates the operational level of service for the intersection under specific design conditions.

- 34 -

GREEN REQUIREMENTS FOR NON-CONFLICTING MOVEMENTS

FIGURE 17

Studies relating per cent of cycle failure to queue length have produced the curve shown in Figure 18. Here it can be seen that queue length increases rapidly at the point of approximately 35% failure. This breaking point is approximately where average arrivals equal average departures.

The procedure makes it possible to consider various design configurations (different number of lanes) as shown in Figure 15 and also the effect of various cycle lengths as related to per cent failures.

Table 4 in the appendix shows phase arrangements and per cent of failure for cycle lengths ranging from 25 seconds up to 165 seconds considering the design shown in the previous example. From these data the curve shown in Figure 19 which relates cycle length to per cent failure was developed.

The curve shown in Figure 17 illustrates the latitude which the designer may take in designing the intersection. A low percentage of failure can be obtained by increasing the cycle length (i.e., increasing capacity) but cycles in excess of 100 seconds may introduce delays of undesirable length. Short cycles may be utilized but one must consider the increased probability of a breakdown. If a satisfactory compromise cannot be reached then it may be desirable to add additional lanes. In any event, the procedure provides a rational approach and permits the engineer to bring his judgement into play to arrive at a sound estimate of design requirements.

Phase Overlap

A similar technique as previously discussed can be applied to the design of diamond interchanges. The only difference is that the term of equation (1) indicating phase overlap is brought into use and the relationship of capacity and cycle length will change. For overlaps greater than 16 seconds the capacity will increase with a decrease in cycle length. Figure 20 illustrates a plot of per cent failure versus cycle length for a typical diamond interchange.
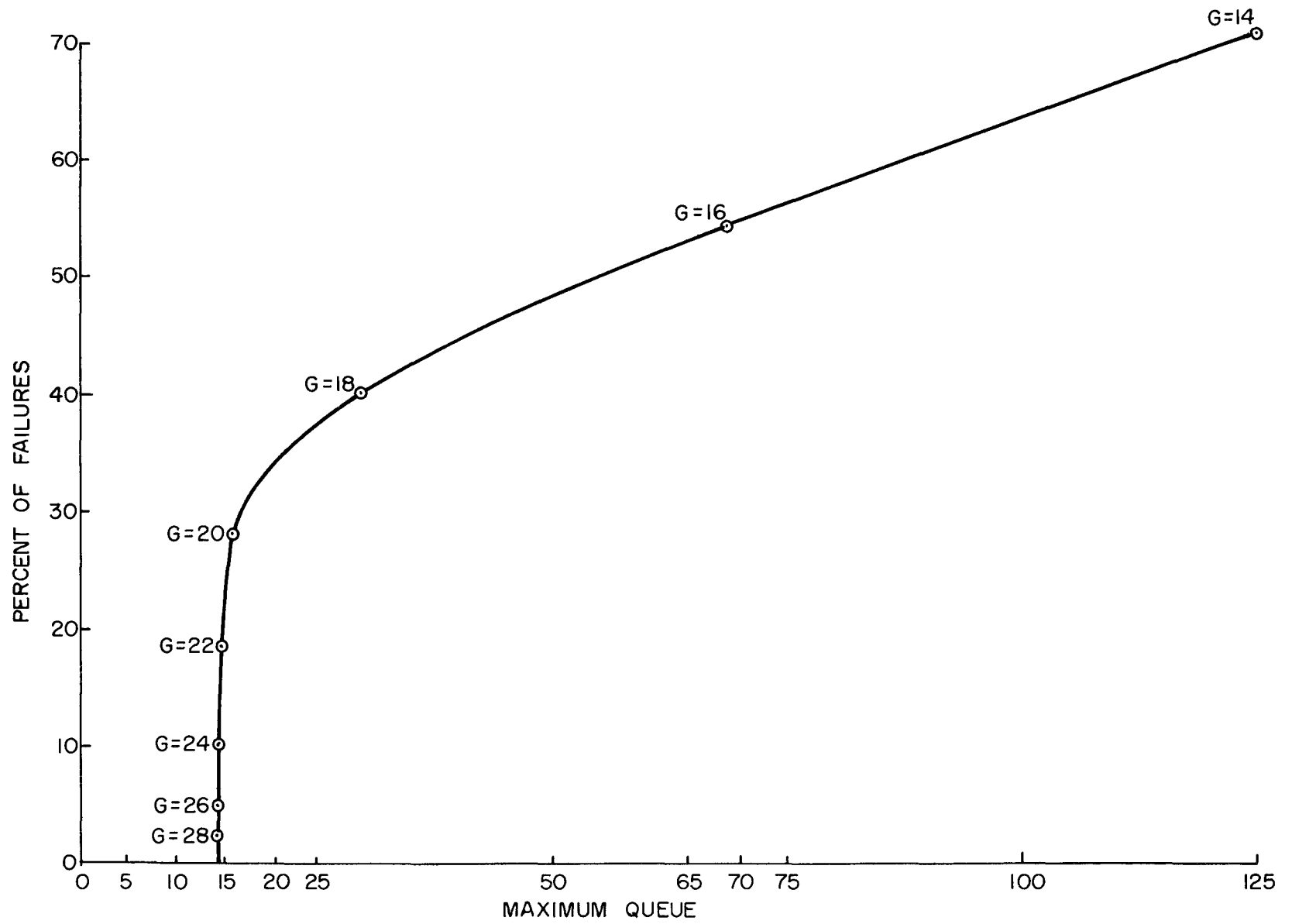
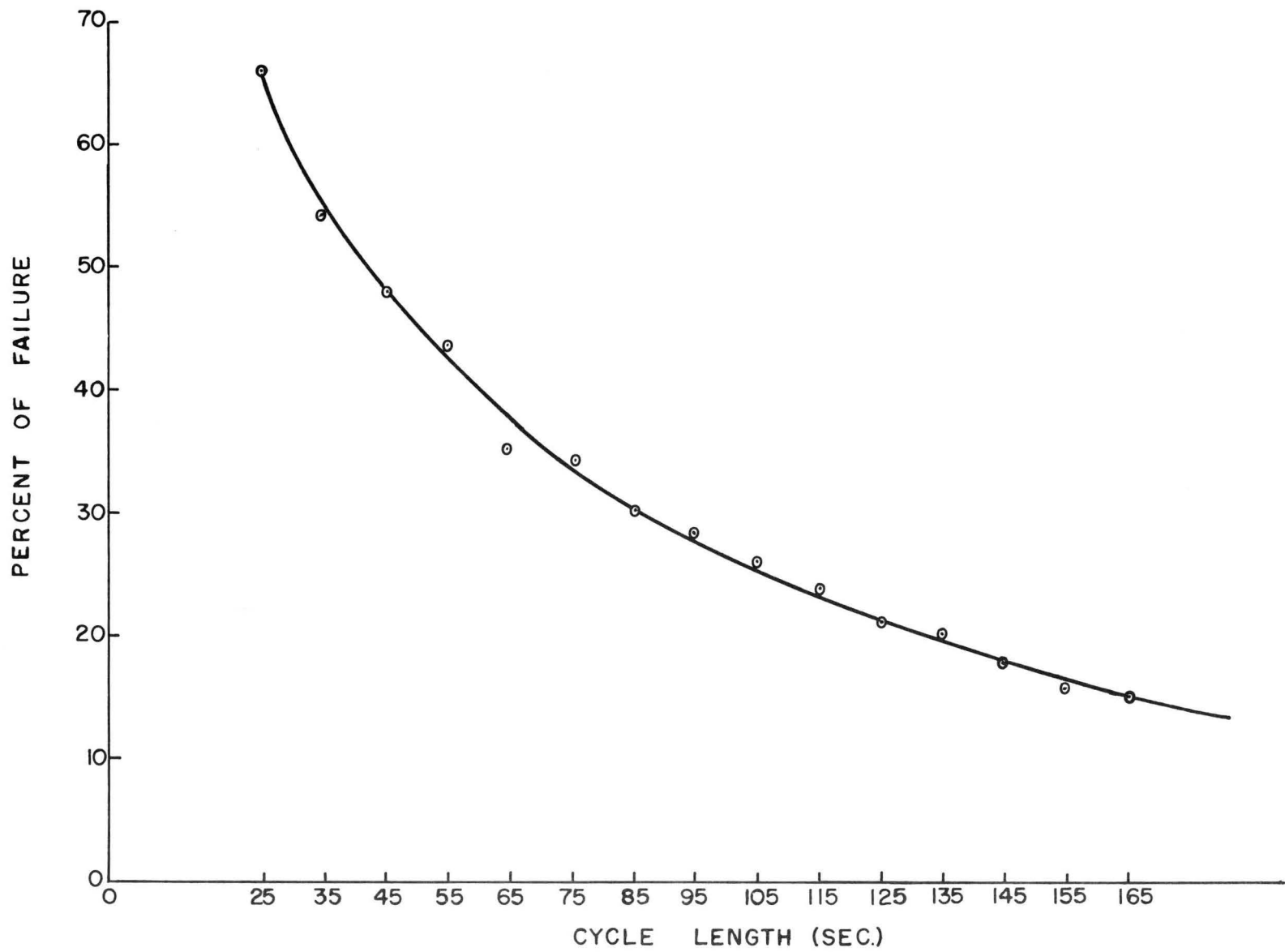FIGURE 18 - CYCLE FAILURES AS DETERMINED BY MAXIMUM QUEUE
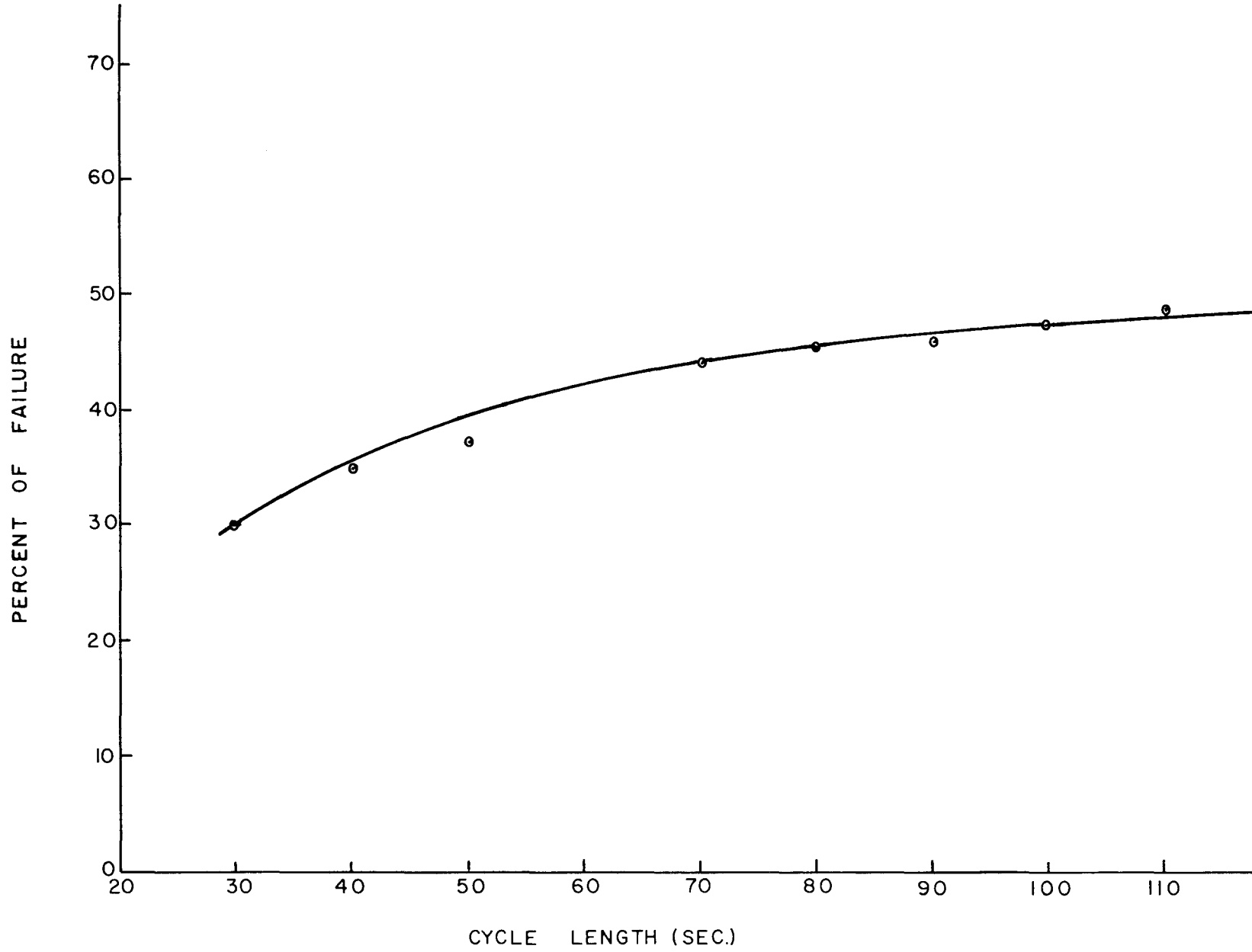
FIGURE 19 — CYCLE FAILURE WITHOUT OVERLAP

FIGURE 20 — CYCLE FAILURE WITH OVERLAP

SUMMARY

Discussion of Design

Highway design is an engineering function -- not a handbook problem. The engineer is faced with the problem of predicting traffic demands in future years and providing facilities that will accommodate that traffic under a selected set of operating conditions or levels of service. Too often highway design has been accomplished by adopting a set of handbook "standards" which when coupled with traffic "guestimates" have resulted in the construction of many seriously inadequate facilities.

Traffic prediction, traffic operation, and design have now developed to the point that it is possible for engineering (the application of science) to produce rather reliable results.

A freeway is not built for some date 20 years in the future. It must go to work the first day and serve efficiently all through its expected life. And, if history is not changed, many will be serving for quite a number of years beyond the "design" year.

The freeway is only one facility in a network of system of streets and highways. It has its place, but the system as a whole must be made to function efficiently. The day has gone when a freeway can be designed within the confines of two parallel right-of-way lines. Likewise, the day has gone when only the 20-year "complete system" can be considered when designing a particular facility. Traffic projections and designs must be made on partial or incomplete systems if desirable service is to be obtained in the years before the whole system is completed. With the modern tools available, the designer should have at his disposal an accurate estimate of traffic demand for each stage of completion of the planned system.
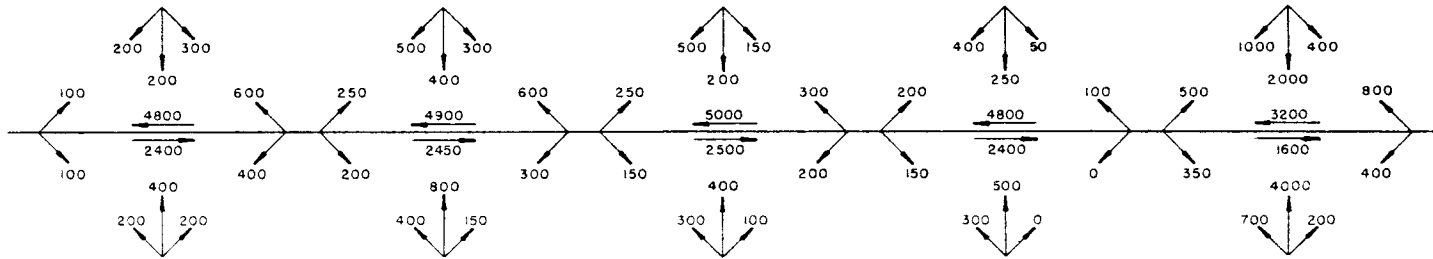
Engineering and management must be coupled in the selection of a level of service for design that is best adapted to the specific need. Economics and other factors will continue to play a major part in facility programming and even in design, but realistic projected service analysis will lead to more realistic priority programming.

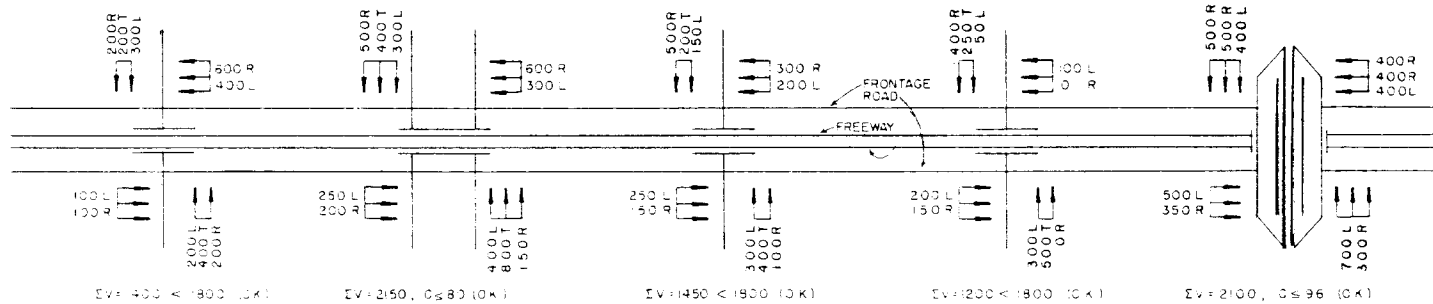Figure 21 illustrates four steps to be followed in the design of a freeway system.

Step 1 - Determine the peak hour volumes through the application of the peak hour and directional distribution factors to the assigned daily traffic volumes. In an actual problem the PM peak would also be checked.

Step 2 - Determine interchange requirements. It is important that this be done before freeway main lane requirements be investigated because the number of ramps depends on the choice of interchange. Thus, a cloverleaf interchange and a directional inter-
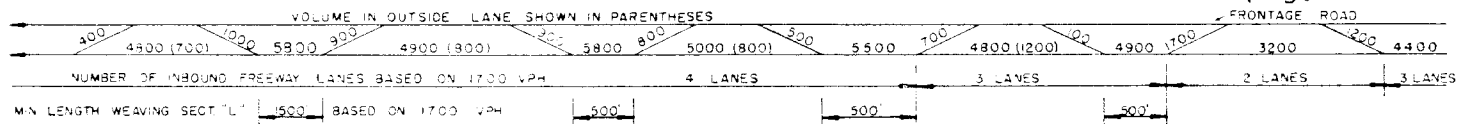
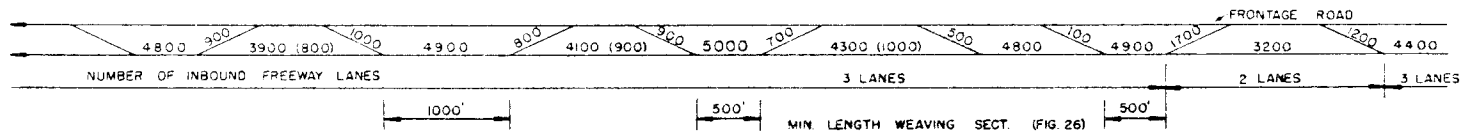STEP 1 — PEAK HOUR VOLUMES (A.M. PEAK) ; DIRECTIONAL DISTRIBUTION = 2:1

STEP 2 — INTERCHANGE REQUIREMENTS (A.M. PEAK) — SEE FIGURES 22 & 23 FOR PHASING & CAPACITY

STEP 3 — MAIN LANE REQUIREMENTS — A.M. PEAK (TABLE 2) & CHECK OF CRITICAL LOCATIONS (Fig. 2 to 5 & 10)

VOLUME IN OUTSIDE LANE SHOWN IN PARENTHESES   FRONTAGE ROAD

4800 (700)   5800   4900 (800)   5800   5000 (800)   5500   4800 (1200)   4900   3200   4400

NUMBER OF INBOUND FREEWAY LANES BASED ON 1700 VPH   4 LANES   3 LANES   2 LANES   3 LANES

MIN LENGTH WEAVING SECT "L"   1500   BASED ON 1700 VPH   500'   500'   500'

STEP 4 — ALTERNATE DESIGN WITH RAMPS REVERSED

                                                                                         FRONTAGE ROAD

4800   3900 (800)   4900   4100 (900)   5000   4300 (1000)   4800   4900   3200   4400

NUMBER OF INBOUND FREEWAY LANES   3 LANES   2 LANES   3 LANES

1000'   500'   MIN. LENGTH WEAVING SECT. (FIG. 26)   500'

FREEWAY   DESIGN   PROCEDURE

FIGURE 21

change may have one or two entrance ramps and one or two exit ramps in each direction; whereas diamond interchanges have one entrance ramp and one exit ramp in each direction. If the interchange is to be signalized, a capacity check is made to see if the planned facilities will handle the traffic with reasonable cycle lengths (See Figures 22 and 23). Should a facility be apparently underdesigned, additional approach lanes may be added or a higher type facility be substituted in its place.
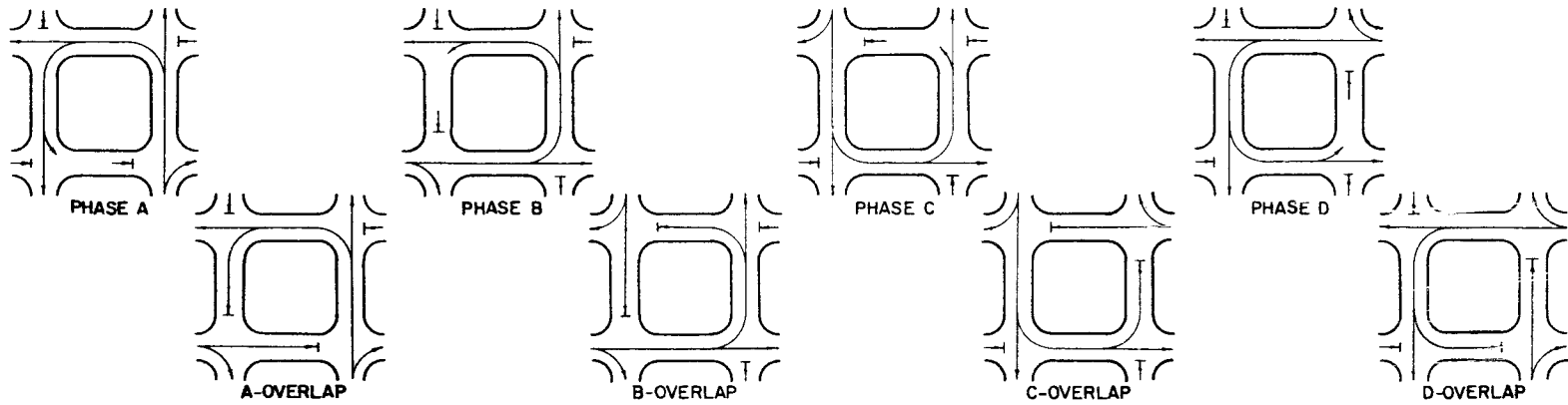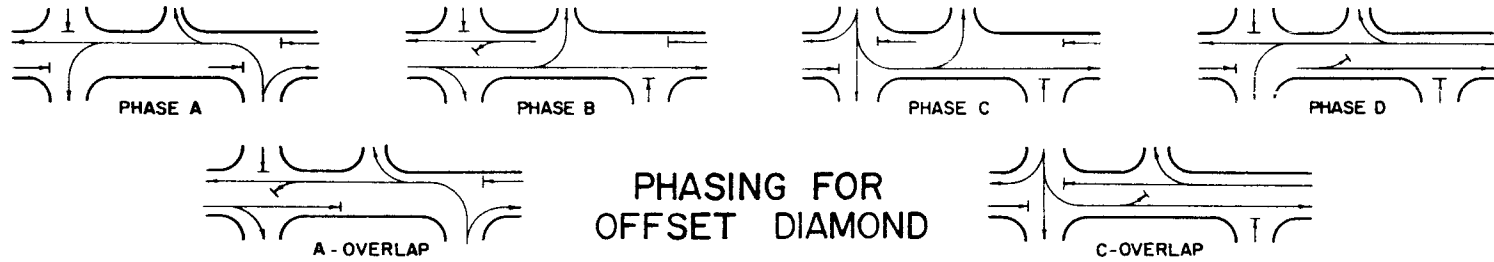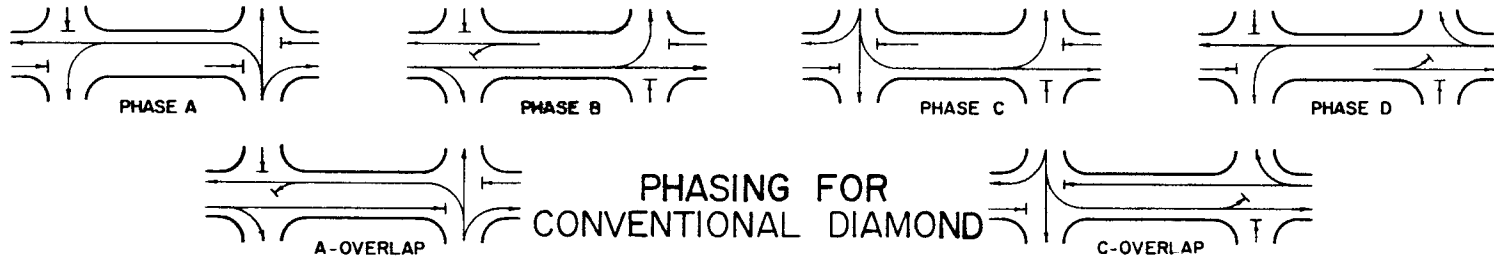
Step 3 - The number of main lanes depends on the service volume value chosen for design. The freeway design service volumes in Table 3 enable the designer to judge what level of service can be expected for a given service volume based on the probability of obtaining various types of flow conditions during the peak 5-minute period. For the purposes of this example a service volume of 1700 vph is chosen. To insure balanced design the operating conditions at critical locations must be checked to insure that the designated level of service is met at every point on the freeway. The critical sections considered in this paper are merging and weaving section. Figure 3 and 5 provide the basis for determining if the merging capacities at entrance ramps are exceeded, where the merging capacity is defined as the service volume chosen in Table 3. Thus, since a total hourly volume of 1700 vph is used as the basis for determining the number of lanes, then 1700 vph would represent the merging capacity in this procedure. Figure 10 provides the basis for determining if weaving sections on the freeway meet the designated level of service.

Step 4 - Alternate designs should always be considered. In Figure 21, one alternative is illustrated by merely reversing the order of entrance and exit ramps, resulting in 3 lanes in each direction instead of 4 lanes.

The level of service should be "in harmony" along the stretch of freeway being considered. Since operational problems at one point are reflected along the freeway for a distance depending on the volume-capacity relationship, it is not practical to consider a lower level of service at one or more critical points, rather the level of service selected for design should be met or exceeded at the critical or bottleneck point. This concept is referred to as "balanced design" and it is a must for freeways.
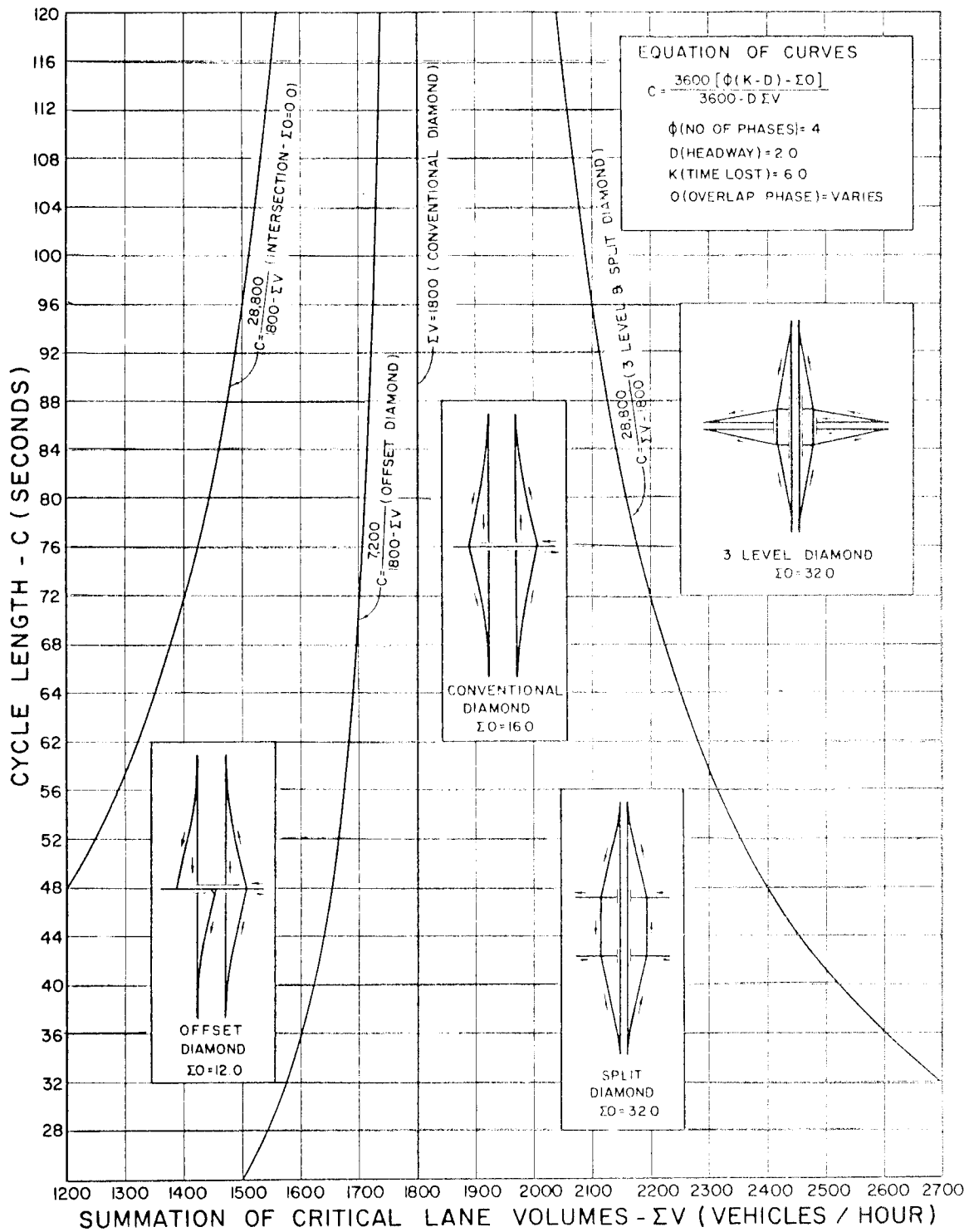
Freeway Surveillance and Control

Freeway design does not always eliminate the need for sound traffic regulation. A reasonably homogeneous traffic stream, particularly with respect to speed, is essential for efficient freeway operations. Pedestrians,

PHASE A PHASE B PHASE C PHASE D

A-OVERLAP **PHASING FOR CONVENTIONAL DIAMOND** C-OVERLAP

PHASE A PHASE B PHASE C PHASE D

A-OVERLAP **PHASING FOR OFFSET DIAMOND** C-OVERLAP

PHASE A PHASE B PHASE C PHASE D

A-OVERLAP B-OVERLAP C-OVERLAP D-OVERLAP

PHASING FOR 3 LEVEL & SPLIT DIAMOND

FIGURE 22

DESIGN CURVES FOR 4 PHASE FACILITIES

FIGURE 23

bicycles, animals and animal-drawn vehicles are excluded from freeways. Motor scooters, nonhighway (farm and construction) vehicles and processions, such as funerals, should generally be prohibited. Towed vehicles, wide loads or other vehicle combinations such as trailers drawn by passenger vehicles which impede the normal movement of traffic should be barred during the peak traffic hours or during inclement weather.

Minimum speed limits are being used more frequently and have been found of great benefit, particularly on high-volume sections. The effect of this type of control is to reduce the number of lane change maneuvers. The effects of slow-moving vehicles on both capacity and accident experience are so pronounced that a greater use of minimum limits appears probable. There is a need to eliminate all vehicles incapable of compatible freeway operation.

Increasing attention has been given to the possibility of and need for using variable speed control on urban freeway sections as a means of easing the accordion effects in a traffic stream as congestion develops. Drastic speed variations might be dampened by automatically adjusted speed message signs in advance of bottlenecks.[14]

A properly designed entrance ramp with provision for adequate acceleration should allow the entering driver adequate distance to select a gap and enter the outside lane of the freeway at the speed of traffic in the lane. These merging areas operate best when there is a mutual adjustment between vehicles from both approaches. "Yield" signs impose rather drastic speed restrictions under the laws of a number of states thus causing operational problems, and are no longer mandatory on the Interstate system. It is generally felt that any speed restriction or arbitrary assignment of right-of-way should be avoided unless inadequacies in the design make it imperative.

It is generally agreed that one key to significant progress in operation of urban freeways lies in improved surveillance technique. In its most basic form, urban freeway surveillance is limited to moving police patrols. Recently, helicopters have been used for freeway surveillance in Los Angeles and other communities. Efficient operation of high density freeways is, however, more than knowing the locations of stranded vehicles; it may require closing or metering entrance ramps, a technique which has been proved effective for freeway traffic control;[2] or it may require excluding certain classes of vehicles during short peak periods. Therefore, what is needed is a reliable, all weather source of surveillance information with no excessive time lag.

Experimentation with closed circuit television as a surveillance tool was initiated on the John C. Lodge Freeway in Detroit. This offers the possibility of seeing a long area of highway in a short or instantaneous period of time, made possible by spacing cameras along the freeway so that a complete picture
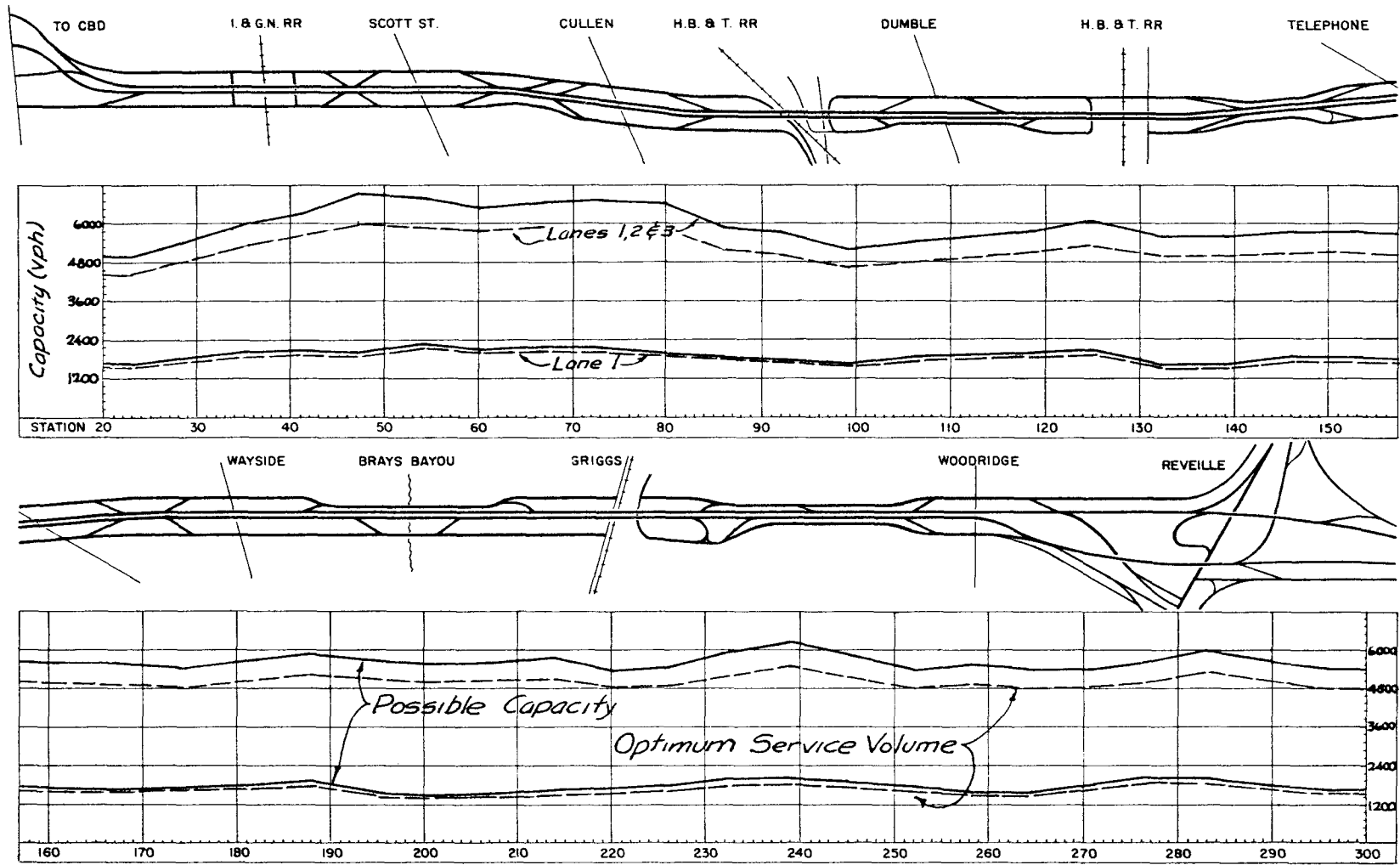
can be obtained of the entire section of roadway. Evaluation of the freeway operation depends mostly on the visual interpretation of the observers. However, many traffic people believe that this is not enough. The Chicago Surveillance Research project, for example is predicated on the assumption that trained observers offered no uniform objectivity. In other words, if an expressway is operating well, this quality can be detected by observing operating characteristics. When the characteristics drop below a predetermined level, action may be taken.

A traffic surveillance system should involve the continuous sampling or basic traffic characteristics for interpretation by established control parameters, in order to provide a quantitative knowledge of operating conditions necessary for immediate rational control and future design. The control logic of a surveillance system, or any system for that matter, is that combination of techniques and devices employed to regulate the operation of that system. The analysis shows what information is needed and where it will be obtained. Then, and only then, can the conception and design of the processing and analyzing equipment necessary to convert data into operational decisions and geometric design warrants be described.

In research conducted during the past year by the Texas Transportation Institute on the Gulf Freeway Surveillance Project, the application of many control parameters to the description and eventual control of freeway congestion was explored. Figures 11 and 12 illustrate the operation of the three inbound lanes of some six miles of the facility during the morning peak hour as obtained from time-lapse aerial photographic studies. Four control parameters, derived in the previous section, are superimposed on the contour maps: (1) the speed at possible capacity, u_m; (2) the density at possible capacity, k_m; (3) the speed at the optimum service volume u'_m; and (4) the density at the optimum service volume k'_m. These parameters afford a rational, quantitative means for describing the level of operation on the facility: stable flow, unstable flow and forced flow.

Figure 24 illustrates continuous profiles of the possible capacity, q_m, and the optimum service volume, q'_m, which were derived by applying the momentum-energy analogy to speed-density data taken from aerials of the facility. Thus, if stable flow is to be maintained on the facility, demand must be kept below the optimum service volume. Use of possible capacity as a basic for ramp metering or control places operation of the facility in the unstable zone of operation, and provides absolutely no safety factor against breakdowns due to statistical variability in demand.

Efforts to measure freeway operational efficiency in terms of traffic "throughput" (momentum) are not consistent with the level of service (energy) concept, since maximum throughput must necessarily be achieved with a high traffic stream density, a low traffic stream speed, and a level of operation typified by "unstable flow." On the other hand, the optimum service volume

CAPACITY PROFILES (LINEAR MODEL)

FIGURE 24

provides for speeds 33% higher, densities 33% lower, a level of operation typ-
ified by "stable flow," and with only a 10% reduction in flow. Actually, because
there is less probability of attaining "forced flow" (congested flow inevitably
accompanied by complete breakdown), the "throughput" from day to day might
very well be higher because of less frequent breakdowns.

Signalized Intersection

A procedure for use within the capacity-design of high-type intersections
and interchanges has been discussed. This procedure has the following de-
sirable features:

(1) It permits consideration of lane design in place of total approach width.

(2) It considers all approaches of the intersection as a single unit in
the design procedure, thus insuring better balance.

(3) It gives consideration to peaking characteristics.

(4) It gives consideration in the design procedure to the phenomeno:
of random vehicle arrivals.

(5) It permits the evaluation of a wide range of possible design and
operating conditions.

It is believed, therefore, that this procedure should be used where ap-
plicable as a supplement to the Highway Capacity Manual procedure for deter-
mining intersection capacity and level of service.

REFERENCES

1. U. S. Bureau of Public Roads, "Practical Applications of Research," Highway Capacity Manual, U. S. Government Printing Office, Washington, D. C., 1950.

2. Keese, Charles J., Pinnell, Charles, and McCasland, W. R., "A Study of Freeway Traffic Operations," Highway Research Board, Bulletin 235, Washington, D. C., 1960.

3. Keese, Charles J., "Improving Freeway Operation," Proceedings of the Western Section, Institute of Traffic Engineers, 1960.

4. Texas Highway Department, Design Manual for Controlled Access Highways, Austin, Texas, January, 1960 (revised December, 1962).

5. Moskowitz, Karl and Newman, Leonard, "Notes on Freeway Capacity," Highway Research Board Record 27, Washington, D. C., 1963.

6. Port Development Department, Planning Division, "Route 3 'Lights On' Traffic Survey," Port of New York Authority, New York, 1960.

7. Drew, D. R. and Keese, C. J., "Freeway Level of Service as Influenced by Volume and Capacity Characteristics," (presented at the 44th annual meeting of the Highway Research Board, January, 1965, Washington, D. C.).

8. Drew, D. R. and Dudek, C. L., "Investigation of a Internal Energy Model for Evaluating Freeway Level of Service," Freeway Surveillance Project Research Report 24-11, Texas Transportation Institute, Texas A&M University, 1965.

9. Drew, D. R., Dudek, C. L., and Keese, C. J., "Freeway Level of Service as Described by an Energy-Acceleration Noise Model," (for presentation at the 45th annual meeting of the Highway Research Board, Jan., 1966, Washington, D. C.).

10. Drew, D. R., "The Energy-Momentum Concept of Traffic Flow," Accepted for publication in Traffic Engineering, 1966.

11. Ryan, D. P. and Breuning, S. M., "Some Fundamental Relationships of Traffic Flow on a Freeway," Highway Research Board, Bulletin 324.

12. Pinnell, Charles and Capelle, Donald G., "Capacity Study of Signalized Diamond Interchanges," Proceedings, 40th Annual Meeting Highway Research Board, 1961.

13. Drew, Donald R., "Design and Signalization of High-Type Facilities,"
    Traffic Engineering, Vol. 33. No. 10, July, 1963.

14. DeRose, Frank, Jr., "Lodge Freeway Traffic Surveillance and Control
    Project," Highway Research Board No. 21, 1963.

APPENDIX

TABLE 4

VARIOUS SIGNAL CYCLE LENGTHS

C = 25 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Per cent of Failure	Phase Length
A	2.9	65	7
B	2.3	65	6
C	2.6	65	7
D	1.5	65	5
			25 OK

C = 35 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Per cent of Failure	Phase Length
A	4.2	54	11
B	3.2	54	9
C	3.6	54	9
D	2.0	54	6
			35 OK

C = 45 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Per cent of Failure	Phase Length
A	5.3	48	14
B	4.1	48	11
C	4.6	48	12
D	2.6	48	8
			45 OK

TABLE 4 continued

C = 55 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	6.5	44	16
B	5.0	44	14
C	5.7	44	15
D	3.2	44	10
			55 OK

C = 75 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	8.9	34	23
B	6.8	34	19
C	7.7	34	20
D	4.3	34	13
			75 OK

C = 85 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	10.1	30	26
B	7.7	30	21
C	8.8	30	23
D	4.9	30	15
			85 OK

TABLE 4 continued

C = 95 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	11.2	28	29
B	8.6	28	23
C	98	28	26
D	5.5	28	17
			95 OK

C = 105 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	12.4	26	32
B	9.5	26	26
C	10.8	26	29
D	6.1	26	18
			105 OK

C = 115 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	13.6	24	35
B	10.4	24	28
C	11.8	24	32
D	6.7	24	20
			115 OK

TABLE 4 continued

C = 125 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	14.8	21	39
B	11.4	21	31
C	12.9	21	34
D	7.2	21	21
			125 <u>OK</u>

C = 135 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	16.0	20	41
B	12.3	20	34
C	13.9	20	37
D	7.8	20	23
			135 <u>OK</u>

C = 145 seconds

Phase ϕ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	17.2	18	44
B	13.2	18	36
C	14.9	18	40
D	8.4	18	25
			145 <u>OK</u>

TABLE 4 continued

C = 155 seconds

Phase φ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	18.4	16	48
B	14.1	16	38
C	16.0	16	42
D	9.0	16	27
			155 OK

C = 165 seconds

Phase -φ	Avg. Arrivals per cycle "m"	Percent of Failure	Phase Length
A	19.6	15	51
B	15.0	15	41
C	17.0	15	45
D	9.6	15	28
			165 OK