QUEUING MODELS FOR AIR

POLLUTION ANALYSES

by

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1.1 Purpose and Scope

Section 19-508 of the General Statutes of the State of Connecticut required that a proposed development generating vehicular traffic be subject to a review to determine its contribution as an indirect source of air pollution. The original requirements covered all developments and placed final responsibility for the review of the permit application with the Connecticut Department of Environmental Protection (ConnDEP). The portion of the application consisting of traffic analysis was reviewed by the Connecticut Department of Transportation (ConnDOT). Since the review of the queuing (waiting line) models represented an additional responsibility outside the normal operation of the Division of Traffic, and because of the potential controversy surrounding many proposals, it was felt that the University of Connecticut might be of assistance in developing a set of general guidelines that would reinforce the objectivity of the process. Thus, the present project through the Joint Highway Research Advisory Council (JHRAC) was originated.

Subsequent to the initial formulation of the project, the general requirement for the indirect source permit was modified so that, at present, it only applies to the direct construction of highways and airports. As a result of this change in requirements it is felt that certain portions of the original proposal should be modified. Since the only projects presently subject to the review process are those of ConnDOT,

* Superscripts in parentheses refer to references listed at the end of the report.
the need for generality of the guidelines is somewhat reduced. However, the need for understanding of and agreement with the guidelines by both agencies remains.

While the original JURAC proposal envisioned guidelines that might be put in rather simplistic form and might be applied to any of a host of submitted models, it now appears that the reduced audience would benefit by more attention to specific models and a deeper understanding of the theory. It is hoped that such an approach will aid both agencies in this area and provide them with a handy means of training new or transferred personnel.

A simple representation of the process under consideration is shown in Figure 1. While the present study concentrates on the second activity shown, it is obvious that (a) the conclusions of activity 2 can be no more accurate than those of activity 1, and (b) the conclusions of activity 2 need be no more accurate than that called for by the accuracy of activity 3*. As a result, activities 1 and 3 have also been examined and, thus, the present report gives an overall description of the entire process from systems planning through emissions modeling. Flow charts are presented and critical decision points are identified.

A detailed critique of queuing, including both classical and simulation models is given. The conditions under which each is applicable and an

* While this approach is quite logical for the present stage, potential increases in the accuracy of the predictions of activity 1 or the requirements of activity 3 suggest that the assumptions underlying activity 2 should be clearly identified so that a balance between the accuracy of the activities can be maintained.
ACTIVITY:

1. Prediction of general traffic demands on specific links and intersections

2. Detailed analysis of traffic on specific links and intersections, Queuing analysis

3. Prediction of carbon monoxide emissions

FUNCTION OF:

ConnDOT
Bureau of Planning and Research

ConnDOT
Bureau of Planning and Research

ConnDEP
Air Compliance Section

FIGURE 1.1 THE OVERALL EMISSIONS MODELING PROCESS
analysis of sensitivity to input parameters are presented. Since the ultimate use of the output of the queuing analysis is in the emissions modeling process, the appropriate equations are discussed in some detail.

Finally, Appendix A is intended for the use of Commdot personnel engaged in traffic-related analyses. A step-by-step procedure is given along with monographs for the solution of the appropriate equations and bounds of acceptable error. Several example problems are given as an aid in explaining the process.

1.2 Literature Review
1.2.1 Systems Planning

There is, of course, much literature available on the transportation planning process. Among the general references, Creighton (32) has become somewhat of a classic in the field. A relatively new text by Bickev et al. (39), although uneven, does discuss recent developments especially in the environmental area. The work of Brutcn (9) is particularly easy introductory reading on the topic as are sections of Faquette, Ashford and Wright (84) and Salter (92). A more comprehensive and sophisticated approach, especially as regards social and economic effects and trade-offs, is taken by Hutchinson (57). Sections of Gaff (45) take an extremely sophisticated approach to network take an extremely sophisticated approach to network modeling.

Government publications are also of value, particularly in providing detailed descriptions of techniques. In the planning area basic documents include those on Origin and Destination Surveys (105), Trip Generation (106),
Trip Distribution (99), Modal Split (102) and Traffic Assignment (104).

Reports by ConnDOT describe the process as applied in the State of Connecticut. The most comprehensive of these is PACER (27), a description of the State's "Environmental Action Plan" as required by Section 109 (h) of Title 23, United States Code. The topic is also briefly discussed in various environmental impact statements (21) and indirect source permit applications (26).

1.2.2 Queuing

Some of the earliest (and still highly pertinent) work on queuing theory was done at the turn of the century by A. K. Erlang. Erlang was interested in telephone switchboard delays and studies of these systems were the major applications of queuing theory until the mid-1930's. Since that time, an extensive literature has developed in the general field of operations research as applied to industrial and other problems.

In one of the pioneering traffic-related papers Adams (1) examined the pedestrian queuing problem. He assumed both pedestrian and vehicular arrivals followed a random distribution; and found these assumptions to be valid through field observations. Adams postulated that a certain critical gap must be present in traffic before a pedestrian will cross the street. In 1950, Eadie (87) further developed Adams' concept of a critical gap, suggesting that the interval between any two vehicles be broken into a period during which a pedestrian will cross and a period during which the trailing vehicle is unacceptably close. The latter period he termed
a "block" and the former an "antiblock". A simpler, related concept was
advanced by Oliver (82) in 1962. Additional work on the pedestrian problem
was performed by Tanner (97), Miller (75), and Underwood (98).

Yield sign delay was studied by Weiss and Maradudin (109), Evans,
Herman and Weiss (43) and Dem (40). Stop sign delay has generally been
studied in connection with warrants. One of the earliest of these papers
is by Raff (87). Later contributions included those of Oliver and Bisbee
(83), Major and Buckley (84), Ashworth (3), Kelt (60) and Lewis and Michael (64).

The fixed-time traffic signal has been studied by a number of inves-
tigators. A continuum approach was taken by May (69) and his analysis, while
not accounting for the obvious probabilistic aspects of the problem, is
nevertheless helpful in giving a feel for the operation.

McNeil and Weiss (73), recognizing that under heavy traffic the represen-
tation of arrivals by a simple Poisson process is inadequate, examined a
compound Poisson process. The discussion includes reference to work by
Darroch (36) on the index of dispersion. The compound Poisson distribution
of arrivals is also used by McNeil (72) and by Daley and Jacobs (34).

Modeling of the process of departures allowing for the effect of turning
vehicles was attempted by Newell (79), Darroch (36), Gordon and Miller (49),
and Little (66). Apparently, however, these studies concentrated on the
delays experienced by the turning vehicle and provided little insight into
the delays caused by them.
Equations for expected waiting time are derived in [73] for the cases of Poisson and more general arrivals. An assumption of binomial arrivals is shown to yield a waiting time equivalent to that obtained by Beckmann et al. [5] in 1956. The simple case of constant arrivals (thus deterministic) was considered in one of the earliest papers on the topic, written in 1941 by Clayton [14]. Expected waiting times for a condition of near saturation were treated by Newell [77].

The overflow at a signal with simple Poisson inputs and constant departure times was studied by Haight [52]. Overflow was also studied by Newell [80], Darrock [36], McNeil [72], Kleinecke [62], and Miller [74].

The most widely quoted work on fixed-time signals is that by Webster [108]. In his classic paper he describes a process for selecting an optimal signal timing. Since consideration of delay and hence queue lengths is an important part of Webster's analysis, it is of particular interest in the present report.

Traffic-actuated signals have been studied by Lehoczky [63], who used results from the theory of storage, Tanner [96] and by Darrock et al. [37]. A simple case of the Darrock et al. model is examined by Dunne [61]. As described in Gazis, the expressions derived by Dunne are based on a binomial arrival process with the j intervals defining it coincident with the constant departure times. The Webster equation was used as a basis for the work of Courage and Papamou [29] on delay estimation.

All of the above types of control were reported on by Lepore [25] in
his paper written on queuing analyses as conducted by ConnDOT.

1.7.3 Simulation

With the advent of the high speed digital computer it became possible to simulate the behavior of vehicles on a microscopic scale. The general process is described in Section 4.3. It has been used for a host of reasons in traffic studies. Some of the early investigators who used simulation were Webster (108) in the development of his technique for optimal signal timing, Yell (60) and Lewis and Michael (64). In a recent study, Courage and Papamichou (29) used simulation to check the validity of their formulation. Probably the biggest use of simulation, however, has been in regard to networks of intersections. Much of this work is described by Gibson and Ross (48). A particularly pertinent paper utilizing simulation is that by Cohen (16) in which he uses the technique to analyze carbon monoxide pollution.

There have been several simulation models submitted in connection with the Connecticut indirect source program. In a 1968 paper, May and Pratt (71) describe a computer program, written in FORTRAN, designed to simulate the traffic at an intersection controlled by a fixed-time signal. After selection of an input rate, a random number generator assigns random arrival times for each vehicle. The vehicles are processed one-by-one through the intersection until one hour's worth of traffic data are simulated. The program allows for varying the input rate, the minimum input headway and the signal capacity. Specification of the minimum input headway is necessary in order to avoid the possibility of very small headways. Signal capacity is handled by assigning a minimum output headway. The three outputs provided by the program are a listing of random arrival
times in sequence, a cycle-by-cycle listing of traffic conditions and a summary tabulation.

The computer model described in (71) was modified by Hay and Gyarmati (70) in 1969. The modification consisted of:

1) permitting the use of measured arrival headways or a composite exponential arrival distribution as well as the original random arrival distribution,
2) permitting the processing of vehicles during the amber phase,
3) introducing a decreasing headway queue discharge as well as the original uniform headway queue discharge and
4) permitting the use of measured departure headways.

Among the conclusions were:

...using a composite exponential arrival headway distribution with a uniform or decreasing discharge headway distribution... gave results which were close to using field measurements of arrival and discharge headways.

Using the composite exponential arrival headway distribution as input ... and a decreasing discharge headway distribution ... represented fairly well the field conditions at a signalized intersection."

"Insect 69"(92) is a submodel of the Dynamic Highway Transportation Model (DHTM), developed by the Stanford Research Institute in the late 1960's. "Insect 69" is capable of analyzing entire networks, even if the signals are interconnected; the model also allows for vehicle-actuated signals.

The model is based on Newell's proposal that traffic can be regarded
as a continuous fluid, rather than composed of discrete vehicles. This concept is similar to the continuum model suggested by May and mentioned earlier in this report. One of the interesting aspects of the model is its attempt to deal with "moving queues" by breaking up the analysis of the traffic into "pulsed" and "non-pulsed" components.

Traffic behavior at each individual intersection is simulated with input characteristics being based on output from upstream intersections. Departure headways are based on the Highway Capacity Manual and unique rates are associated with each of three possible departure directions. Geometric input required for the program includes length or number of turning lanes, and number of approach lanes. Offsets of upstream signals may be updated at a time increment specified in the model inputs. A fairly sophisticated controller simulation allows specification of initial green interval, maximum green, unit extension and clearance interval.

Consol Corporation has developed a model based on the formulas found in Wohl and Martin. It is a simple program, analyzing one intersection at a time. The model, named "Intsim", was field checked briefly, and gave fairly good results.

As noted in Hillier of England's Road Research Laboratory, suggested a simple, yet effective analysis of network-wide signal timing. His model (more commonly known as the delay/difference of offset model) states that the delay experienced by vehicles on any street is caused solely by the difference in the offsets of the signals at each end of the street. The program seeks to determine the optimal timing pattern to
minimize delay. Tests indicate that this model works as well as, if not better than, the Webster model.

Another network simulation model showing great promise is NETSIM. The model was originally called UTCS-1 and a concise description is given by Lieberman et al.\(^{(65)}\) and by Gibson and Ross\(^{(48)}\).

1.2.4 Air Pollution

There are, of course, many articles which have been written on air pollution, both in general and as related to the automobile. During the preparation of this report, much use has been made of a text by Biberno and Young\(^{(6)}\). An excellent summary and evaluation of dispersion models is given by Darling et al.\(^{(35)}\) More recent work includes papers by Claggett\(^{(13)}\) and by Chock\(^{(10)}\). A paper of considerable interest, since it describes the origin and operation of the Connecticut modeling process, is that of Hanisch et al.\(^{(21)}\)
2. THE CONNECTICUT PROCESS

2.1 General

Figure 2.1 is a flow chart of the overall process as conducted in Connecticut, beginning with the systems planning stage and continuing through the estimation of future carbon monoxide levels in the vicinity of a given intersection in some future year.

2.2 Systems Planning

In Connecticut, the transportation planning process has been conducted by the Bureau of Planning and Research of ConnDOT. That organization has provided the long-range systems analyses required to predict future volumes of traffic upon which all subsequent analyses are founded. While there is activity underway to transfer more of the responsibility to the metropolitan planning organizations (MPO's), the bulk of the work is still being done by ConnDOT. The following description of traffic volume predictions is therefore based on that agency's procedures.

2.2.1 Traffic Zones and the Network Graph

In order to simulate existing travel patterns and predict future patterns, it is necessary to graphically represent the geographic area under consideration as well as the physical transportation system. The geographic area is divided into "traffic zones" of relatively homogeneous character. This homogeneity is required in order to aggregate the socioeconomic data thought to influence trip-making behavior. The selection of zone size is very important in subsequent calculation and the "appropriate" size is dictated by the scale of the traffic patterns to be modeled. Obviously smaller zones lead to more precision, but this must be weighed against...
Figure 2.1 Prediction of future CO concentrations

Political Sentiment

Base Year Socio-Economic Factors

Predicted Socio-Economic Factors

Base Year Network Characteristics

Proposed Network

Predicted Distribution

Refined Assignment

Selection of Peak Hour and Peak 8-Hour Volumes

Refined Design

Trip Generation Equations, Calibrated Gravity Model

Refinement Possible?

Yes

No

Prediction of Q

Emissions Analysis by ConnDEP

Yes

No

Excessive? (ConnDOT)

Formal Application to ConnDEP

$Q > Std. 
\geq 10 m \text{ from Broadway}$

Yes

No

Build
the increased costs. In Connecticut, the entire State comprises the study area and it is divided into 884 zones. The largest generator of trips is a zone in the Hartford central business district, with 199,000 per day. The smallest in this sense is a zone which covers about half the town of Union. It includes the Nipmuck State Forest and generates 1400 trips per day.

The graphical representation of the transportation system is by means of a "network". Subsequent computer calculations require that this network be coded and it is therefore broken down into a series of links and nodes. As described by Kelly (54):

A link usually represents some physical distance (or time) that may be traversed on the street network. A complete description of a link would provide travel time and/or cost as functions of link occupancy, rate of flow, time and other pertinent factors. A node represents a terminus of one or more links. A node may be used to (1) join the termini of a number of links, or (2) represent a "source" (= origin) or "sink" (= destination) of traffic. The traffic originating or terminating in a network zone is assigned to one or more appropriately located nodes of the network graph.

At first glance, it would appear obvious that the appropriate labeling strategy would be to have each node (of type (1) above) coincide with an intersection and each link with a street or highway (Figure 2.2A). However, it is neither practical nor necessary to represent every intersection and street in the physical system. Thus, again depending on the scale of the phenomenon to be modeled, the network might represent the freeway system or the system of freeways and major arterials etc. In Connecticut, the system coded into the network comprises all highways and streets down to and including the local collectors.* In densely populated urban areas many local

* A commonly used classification system is shown in Figure 2.3 and Table 2.1.
Figure 2.2A  Graph of a Network
Nodes at Intersections
(a) Part of a street network  (b) Equivalent part of network graph.

Figure 2.2B  Graph of a Network
Nodes between Intersections
(a) Part of a street network.  (b) Equivalent part of a network graph.
FIGURE 2.3 Diagrammatic layout illustrating functions of street system.

[From Traffic Flow Theory and Control by Drew Copyright (c) 1968 by McGraw Hill, Inc. Used with permission of McGraw-Hill Book Company]
<table>
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<tr>
<th>Type of facility</th>
<th>Function and design features</th>
<th>Spacing</th>
<th>Widths</th>
<th>Desirable maximum</th>
<th>Speed, mph</th>
<th>Other features</th>
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<td>Freeways</td>
<td>Limited access; no grade cross-ings; no traffic stops</td>
<td>Variable; related to regional pattern</td>
<td>200-300</td>
<td>12 ft per lane; 8-10 ft for shoulders; 8-60 ft for median strip</td>
<td>2</td>
<td>70</td>
</tr>
<tr>
<td>Expressways</td>
<td>Limited access; channelized grade crossings; parking prohibited</td>
<td>Variable; generally radial or circumferential</td>
<td>150-250</td>
<td>12 ft per lane; 8-10 ft for shoulders; 8-30 ft for median strip</td>
<td>3</td>
<td>50</td>
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<td>Major roads (major arterials)</td>
<td>Usually form boundaries for neighborhoods; parking generally prohibited</td>
<td>1 1/2-2 miles</td>
<td>120-150</td>
<td>84 ft maximum for four lanes parking, and median strip</td>
<td>4</td>
<td>35-40</td>
</tr>
<tr>
<td>Secondary roads (minor arterials)</td>
<td>Main feeder streets. Signals where needed; stop signs on streets</td>
<td>3/4-1 mile</td>
<td>80</td>
<td>60 ft</td>
<td>5</td>
<td>35-40</td>
</tr>
<tr>
<td>Collector streets</td>
<td>Main interior streets. Stop signs on side streets</td>
<td>1/4-1/2 mile</td>
<td>64</td>
<td>44 ft (2-12 ft for traffic lanes; 2-10 ft for parking lanes)</td>
<td>5</td>
<td>30</td>
</tr>
<tr>
<td>Local streets</td>
<td>Local service streets. Non-conductive to through traffic</td>
<td>300-800 ft</td>
<td>50</td>
<td>36 ft where street parking permitted</td>
<td>6</td>
<td>25</td>
</tr>
</tbody>
</table>
streets are also included. The network consists of 6000 links and 4200 nodes. (Plus the nodes at zone centroids which are sources and sinks). This labeling strategy, although obvious and commonly practiced, has certain disadvantages. As observed by Helly (54), these are:

1. In a congested environment, delays occur more at intersections than in between. Since travel cost or time is associated with the links, it is more appropriate to have links pass through intersections and to have the nodes at midblock points.

2. Delays are occasioned by turns at intersections. Sometimes turns are prohibited entirely. If nodes are placed at intersections, there must be an additional superstructure of modeling methodology to cope with turn costs or prohibitions. This added complication may be avoided by again placing nodes at midblock points and by placing a separate, appropriately priced turning link for each allowed turning movement.

3. If all of zone's trip ends are injected at one node, then there may be unrealistic congestion in the vicinity of that node. The solution is to have zonal source and sink traffic connected to more than one node. If this is done, the single zone centroid node must be replaced by two nodes, one for trip originations and the other for terminations. If only one centroid node is used, the zero-time links to the several network nodes may be used improperly for nonexistent short cuts via the centroid by through traffic that does not have a trip end in the zone.

Figure 2.28 shows how the improvements suggested by Helly might be incorporated.

2.2.2 Existing Travel Patterns

Knowledge of existing travel patterns is necessary for a number of reasons. In terms of the planning process, the data are used (1) to identify existing deficiencies and adverse effects and (2) as a basis for
predicting future travel patterns. The data base of existing traffic volumes is maintained by the Engineering Data and Inventory Section. It is derived from (1) 37 automatic traffic recorders and toll stations throughout the State giving a continuous count for 24 hours a day, 365 days a year, (2) "control" counts and (3) "coverage" counts. "Classification" counts are also performed for the purpose of identifying the composition of the traffic in terms of percentage of trucks.

The planning process, however, requires additional information on travel patterns, primarily origin and destination data. Typically, these data are acquired during the initial phases of the process and are updated periodically. While a number of techniques may be used, the data are normally collected by means of home interview survey, roadside interviews and commercial vehicle survey. In Connecticut the surveys upon which current computations are based were performed in 1962-64.

2.2.3 Trip Generation

It has been found that the number of trips that will be made from or to such points as a household, store, or workplace may be related to socio-economic data such as population, income, automobiles owned, etc. The appropriate relation for the base year is established through multiple regression, which is a statistical technique based on achieving the best "fit" of the observed data to the postulated relation. The resulting "trip generation" equations are then assumed to hold for future years. Thus the future number of trips may be determined by first predicting the socio-economic variables and then inserting these into the equations to predict the number of trips that will be generated. In Connecticut, the
trip generation equations are those given in Table 2.2. To illustrate, the work trips that will be produced at some future date in a zone in a "non-transit" town will be roughly 133 plus .93 times the difference between the labor force and the number of transit users.

It may be seen that the important socio-economic determinants in Connecticut are felt to be labor force, car ownership, population and various categories of employment. According to the Federal Highway Administration, the important determinants nationwide are shown in Table 2.3.

In Connecticut, the prediction variables are generally adjusted in proportion to population predictions provided by the Office of Management and Budget.

It is very important to note that although the prediction variables are updated, the coefficients determined from the multiple linear regression analysis for the base year are assumed to hold constant into the future. In effect, this assumes that those socio-economic data found to be important determinants in the base year will be important in the future and will retain the same relative importance amongst themselves.

The determination of the number of trips made by public transit is, at present, based on the percentage determined by the original home interview survey. In cases where particularly significant transit improvements are contemplated, these percentages are hand adjusted.
Production Equations

**Transit 1 Towns**

\[ WP = 0.913 \text{ LF-T} + 258.170 \]
\[ LF = 0.344 \text{ CARS} + 216.770 \]
\[ SP = 1.258 \text{ CARS} - 137.150 \]
\[ NP = 0.470 \text{ CARS} + 0.244 \text{ SE} + 1.215 \text{ RE} + 0.129 \text{ ME} + 215.220 \]
\[ TP = 0.058 \text{ POP} + 0.159 \text{ TØ} + 274.130 \]

\[ R^2 = \]
\[ 0.75 \]
\[ 0.35 \]
\[ 0.65 \]
\[ 0.79 \]
\[ 0.55 \]

**Transit 2 Towns**

\[ WP = 1.032 \text{ LF-T} + 209.820 \]
\[ LF = 0.375 \text{ CARS} + 167.030 \]
\[ SP = 1.379 \text{ CARS} + 145.870 \]
\[ NP = 0.408 \text{ CARS} + 0.691 \text{ SE} + 1.826 \text{ RE} + 0.092 \text{ ME} + 194.010 \]
\[ TP = 0.100 \text{ POP} + 0.260 \text{ TØ} + 85.020 \]

\[ R^2 = \]
\[ 0.73 \]
\[ 0.39 \]
\[ 0.68 \]
\[ 0.58 \]
\[ 0.34 \]

**Non-Transit Towns**

\[ WP = 0.927 \text{ LF-T} + 132.870 \]
\[ LP = 0.388 \text{ CARS} + 87.720 \]
\[ SP = 1.483 \text{ CARS} - 50.72 \]
\[ NP = 0.477 \text{ CARS} + 0.229 \text{ SE} + 3.866 \text{ RE} + 0.278 \text{ ME} - 50.250 \]
\[ TP = 0.128 \text{ POP} + 0.199 \text{ TØ} + 67.980 \]

\[ R^2 = \]
\[ 0.69 \]
\[ 0.32 \]
\[ 0.69 \]
\[ 0.77 \]
\[ 0.35 \]

**Note:** For explanation see text and pages 19 and 20.

**TABLE 2.2** The Connecticut Trip Generation Equations
Attraction Equations

Transit 1 Towns

\[ WA = 0.703 \ \theta T + 0.285 \ SE + 1.010 \ RE + 0.805 \ ME + 783.840 \]

\[ R^2 = 0.83 \]

\[ LA = 0.157 \ SE + 0.072 \ POP + 338.050 \]

\[ R^2 = 0.31 \]

\[ SA = 1.970 \ RE + 0.197 \ POP + 410.520 \]

\[ R^2 = 0.61 \]

\[ NA = 0.268 \ SE + 1.246 \ RE + 0.116 \ POP + 0.137 \ ME + 368.350 \]

\[ R^2 = 0.77 \]

\[ TA = 0.068 \ POP + 0.052 \ ME + 0.129 \ TW + 196.510 \]

\[ R^2 = 0.56 \]

Transit 2 Towns

\[ WA = 0.663 \ SE + 1.984 \ RE + 0.796 \ ME + 404.530 \]

\[ R^2 = 0.84 \]

\[ LA = 0.403 \ RE + 0.122 \ POP + 82.710 \]

\[ R^2 = 0.42 \]

\[ SA = 0.729 \ SE + 3.961 \ RE + 0.236 \ POP + 145.090 \]

\[ R^2 = 0.62 \]

\[ NA = 0.762 \ SE + 2.029 \ RE + 0.123 \ POP + 0.138 \ ME + 219.010 \]

\[ R^2 = 0.61 \]

\[ TA = 0.097 \ POP + 0.258 \ TW + 92.400 \]

\[ R^2 = 0.32 \]

Non-Transit Towns

\[ WA = 0.763 \ \theta T + 0.892 \ SE + 2.076 \ RE + 0.797 \ ME + 123.160 \]

\[ R^2 = 0.83 \]

\[ LA = 0.402 \ \theta T + 0.247 \ SE + 0.125 \ POP + 49.360 \]

\[ R^2 = 0.46 \]

\[ SA = 4.889 \ RE + 0.340 \ POP + 0.358 \ ME - 118.510 \]

\[ R^2 = 0.78 \]

\[ NA = 4.070 \ RE + 0.193 \ POP + 0.291 \ ME - 57.180 \]

\[ R^2 = 0.77 \]

\[ TA = 0.122 \ POP + 0.089 \ ME + 0.166 \ TW + 73.250 \]

\[ R^2 = 0.36 \]

Note: For explanation see text and pages 19 and 20.

Table 2.2 The Connecticut Trip Generation Equations (Con’t.)
Transit 1 - Urban area with developed transit service.
Transit 2 - Suburban area with partial transit service.
Non-Transit - Rural area with no transit service

WP = Work trip productions
LP = Long trip productions
SP = Short trip productions
NP = Non home-based trip productions
TP = Truck trip productions
WA = Work trip attractions
LA = Long trip attractions
SA = Short trip attractions
NA = Non home-based trip attractions
TA = Truck trip attractions
LF-T = Labor force - transit users
CARS = Car ownership
POP = Population
ME = Manufacturing pop.
RE = Retail employment
SE = Service employment
OT = Other employment
TO = Total other employment (RE, SE, OT)

TABLE 2.2 - The Connecticut Trip Generation Equation (Con't.)
Five Trip Purposes for Auto-Truck Travel

1. Work Trips (Home to Work – Work to Home)

2. Long Trips
   Grouped Trip Purpose
   a. Related Business
   b. Education
   c. Social
   d. Recreation

3. Short Trips
   Grouped Trip Purpose
   a. Personal Business
   b. Medical-Dental
   c. Eat-Drink
   d. Civic-Religious
   e. Convenience Goods
   f. Shopping Goods
   g. Other
   h. Serve Passenger

4. Non-Home Based Trips

5. Truck Trips

TABLE 2.2 - The Connecticut Trip Generation Equations (Con't)
A. Variables found significant in zonal trip generation analysis

1. Demographic Data
   a) Total population 1 *
   b) Age, sex, race, etc. 3
   c) No. of household units 1
   d) School enrollment 2
   e) Family life cycle 3

2. Economic Data
   a) Total employment (1) ** 1
   b) Selected employment (2) 1
   c) Employment by industry (3) 3
   d) Employees by residence (4) 1
   e) Labor force (5) 3
   f) Labor force by occupation and industry (6) 3
   g) Median income 1
   h) Income stratified 3
   i) Automobile ownership 1
   j) Dwellings without autos 2
   k) Retail sales 2
   l) Average home value 3

3. Land Use Data
   a) Specified activities 3
   b) Selected categories 1

TABLE 2.3 Selected Variables Found Significant in Urban Transportation Planning [After (100)]
B. Variables found significant in dwelling unit trip generation analysis

a) Car ownership 1
b) Family size 1
c) No. of persons 5 years old, and over in household 1
d) Length of residence 3
e) Family income 2
f) No. of persons 16 years old and over 2
g) No. of persons 16 years old and over who drive 1
h) Age of head of the household 2
i) Distance from the CBD 3
j) Stage in the family life cycle (?) 1
k) Occupation of head of household 1
l) Structure type 1

TABLE 2.3 Selected Variables Found Significant in Urban Transportation Planning [After (100)] (Con't.)
* Key to weights: 1 = Essential data; 2 = Desirable data; 3 = Useful data.

(1) **Total Employment:**
All persons employed in an analysis unit or study area (without regard to place of employment); or, employment by place of employment (without regard to industry or occupation).

(2) **Selected Employment:**
An employment grouping (either total employment or partial employment) for an analysis unit or study area that classifies workers.

(3) **Employment by Industry:**
All employment in an analysis unit or study area grouped according to the SIC or other industry classification system; or, employment by place of employment and by industry.

(4) **Employees by Residence:**
All persons living in an analysis unit or study area who are employed someplace (with or without regard to industry, occupation, or class of worker categories). A person is employed if he has a job (although he may be absent due to illness, vacation, etc.).

(5) **Labor Force:**
All persons living in an analysis unit or study area who are either employed or unemployed (without regard to industry, occupation, or class of worker of the employed; or to experience of the unemployed; also without regard to place of employment of the employed).

(6) **Labor Force by Occupation:**
All persons living in an analysis unit or study area who are either employed or unemployed and grouped according to occupation of the employed or experience of the unemployed. "Experience of the unemployed" refers to the occupation at which they last worked.

(6) **Labor Force by Industry:**
All persons living in an analysis unit or study area who are either employed or unemployed and grouped according to industry of the employed or experience of the unemployed. "Experience of the unemployed" refers to the industry in which they last worked.

(7) **Stage in the Family Life Cycle:**
Based on age of the head of the household, marital status and age and number of children.

**TABLE 2.3 Selected Variables Found Significant in Urban Transportation Planning [After (100)]**
2.2.4 Trip Distribution and Traffic Assignment

The next step in the procedure is to answer the question "to where do the trips go and from where do they come?" In Connecticut the distribution of trips is by means of a synthetic "gravity" model, so termed because of its resemblance to Newton's law of gravitational attraction.

The model may be stated as:

\[ T_{ij} = \frac{A_i F_{ij} K_{ij}}{\sum_j A_j F_{ij} K_{ij}} \]

where:
- \( T_{ij} \) = the trip interchange between zones \( i \) and \( j \),
- \( A_j \) = the trip attractions at zone \( j \),
- \( P_i \) = the trip productions at zone \( i \),
- \( F_{ij} \) = the travel time factor for a trip between zones \( i \) and \( j \), and
- \( K_{ij} \) = socio-economic adjustment factor between zones \( i \) and \( j \).

The model assumes that the number of trips that will be made between any two zones is directly proportional to the sizes of the zones and inversely proportional to some power of the distance (or time) between the zones. The impedance due to travel time is accounted for by the travel time factors \( F_{ij} \). The adjustment factor \( K_{ij} \) is inserted to account for the fact that certain zonal pairs will not fit the basic formulation. For instance, the residential area immediately outside the central business district (CBD) would, without the introduction of \( K_{ij} \), supposedly interchange a large number of trips with the CBD. In the typical case where CBD employment is white-collar and the immediately surrounding population is actually employed in lower economic scale positions, the resulting distribution would be erroneous.
without the $k_{ij}$.

In order for the model to be used, it is first necessary that it be "calibrated". In general two conditions must be satisfied through the calibration procedure:

(1) The number of trips attracted to a given zone according to the model should equal the total number of attractions known to occur in that zone.

(2) The frequency of occurrence of trips taking various times should approximate that observed in the base year.

These conditions are satisfied through an iterative procedure which results in the traveltime factors $F_{ij}$. A sample of the $F_{ij}$ used in Connecticut is given in Table 2.4.

With knowledge of the appropriate socio-economic variables and having the trip generation equations and a calibrated trip distribution model, it is possible to predict, for any year, the number of trips that will interchange between any zone and all other zones. The final stage in the process is to determine the route taken by all of these trips. This "traffic assignment" is based on the route of least "cost." In general, and in Connecticut, the cost is in terms of travel time. As each of the trip interchanges between zones is loaded into the network, a tabulation is kept of the volume on each link. At this point it is possible to divert some of the traffic to an alternate route or to apply a "capacity restraint" to the volume allowed on each link. This is in recognition of the fact
<table>
<thead>
<tr>
<th>Time</th>
<th>F_{ij}</th>
<th>Time</th>
<th>F_{ij}</th>
<th>Time</th>
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**TABLE 2.4: Sample of Connecticut Travel Time Factors (Work)**
that, as the loading on a particular link approaches capacity, it becomes less attractive and hence less vehicles will use it. In Connecticut, the process at the systems planning stage utilizes an "all-or-nothing" assignment wherein vehicles are all assigned to the route originally of least cost regardless of subsequent loading. These volumes are subsequently hand adjusted at the corridor location stage to assign certain portions of the volume to alternate local routes.

The distribution and assignment phases are based on a twenty-four hour period (AADT). For design purposes and for input to the indirect source procedure, it is necessary to apply suitable multiplicative factors to determine: (1) an estimate of the variation in volume by season, by day of the week and by hour within the day; (2) a directional split; and (3) a percentage of trucks. This is done using general categories derived from the ongoing inventory program.

2.3 Permit Application

A broad description of the operation of the indirect source program in Connecticut is given in (88). The paragraphs below draw from that document and add an in-depth view of the permit application.

The Connecticut indirect source regulation, as revised through 1977, requires a permit for construction or modification of any "indirect source" whose operation will or may result directly or indirectly in aggregate total emission of air pollutants greater than 50 tons per year or in a significant reduction in air quality. An "indirect source" is defined as any highway with a design capacity of 1,000 vehicles per hour
in one direction or any airport designed to accommodate 50,000 commercial flights per year.

The indirect source review procedure for highways is shown in Figure 2.4 which is modified after (88). According to (88) "the regulation does not specify what air pollutants are to be analyzed, or how they are to be analyzed, nor does it specify the time horizon for the air quality analysis." The present procedure calls for hydrocarbons to be analyzed at the systems planning stage and referencing this analysis at the project level. As a result, the pollutant of interest in the present report is carbon monoxide.

A sample application for an indirect source is included as Appendix B of this report. Briefly, the process consists of:

1. Identifying those "existing and planned roads for the distance along which the traffic from the proposed highway or modification will contribute the equivalent of at least ten percent (10%) of the ETC + 1 (No-Build) peak hourly traffic volume on at least one directional link approaching an intersection;"

2. Examining the intersections identified in 1 (above) to determine, by means of "screening charts", if a queuing analysis is required in any link; and

3. Performing a queuing analysis for those intersections having one link requiring a queuing analysis according to the screening charts; for "all exits and entrances to the proposed highway and all points of possible congestion on
FIGURE 2.4 State of Connecticut Indirect Source Review Procedure for Highways

[After (88)]
the proposed highway (e.g., tunnels, toll booths, etc.); and all intersections requested by the DEP after reviewing the above."

The queuing analysis, to which reference is made, pertains to the peak hour and peak eight hour time periods.

The resulting predicted queue properties along with speeds and volumes for the free-flow conditions are then used by ConnDEP to determine the peak hour and peak eight hour CO concentrations as described in Section 3.
3. EMISSIONS MODELING

As discussed in the previous section, the final phase in the overall process is the calculation of average emission rates over the peak 8-hour period. The following discussion considers first the emissions modeling procedure in general and next the specific equations used and assumptions inherent in the Connecticut model. It was felt appropriate to discuss the emissions modeling phase before the discussion of queuing, even though the queuing analysis precedes the emissions analysis in the process. This was done so that the queuing analysis could later be examined in depth and so that characteristics of the queuing models that relate to either system or emissions analyses could be pointed out in the course of that examination.

3.1 General Theory

The pollutant under consideration is carbon monoxide (CO). The National Ambient Air Quality Standards (Federal Register, 32, 11355-11356, May 7, 1973) calls for a maximum 8-hour concentration of 9.6 ppm and a maximum one-hour concentration of 35.0 ppm (both at 25°C and 760 mm). The regulations call for these limits to not be exceeded more than once annually.

Since CO reacts relatively slowly, its effects are localized and thus the appropriate scale for examination is that of an individual highway element such as an intersection.

* Since the 8-hour criterion nearly always governs, it will be discussed here.
The simplest model that might be used to predict concentration can be developed as follows*. Consider an area (Figure 3.1) emitting pollutant at the rate of Q (mass emission rate/units area). If the height of an assumed mixing layer is taken to be Z, then an imaginary box of unit width may be constructed around the source area and it may be shown that

\[ X_e = \frac{QZ}{U} \tag{3.1} \]

where

- \( X_e \) = equilibrium concentration (mass per unit volume),
- \( U \) = wind speed,

and the other symbols are as indicated in Figure 3.1.

The important observation (for this very crude model) is that equilibrium concentration varies inversely with wind speed.

Of the more sophisticated approaches, the two general categories which are most often used are Gaussian models and conservation of mass models. Since the Connecticut process most closely approximates the Gaussian, that category of models is discussed here.

Consider a "puff" of material from a point source at height \( H \) as shown in Figure 3.2. It can be shown that the concentration of material at any point in space at a time \( t \) will be given by \( X \),

*Much of the material presented in this section is based on (6) or (35).*
Figure 3.1 Area-Source "Box" Diffusion Model
A. Three Dimensional Puff of Material

B. Component Distributions of Material about Axes through \((u, 0, 0)\)

FIGURE 3.2 Schematic Representation of Gaussian Puff Model [After (35)]
\[ x(x,y,z,t) = \frac{Q}{1.5} \exp \left[ -\frac{(x-\bar{u}t)^2}{2\sigma_x^2} - \frac{y^2}{2\sigma_y^2} \right] \]

\[ = \exp \left[ -\frac{(y-B)^2}{2\sigma_y^2} \right] + \exp \left[ -\frac{(z+B)^2}{2\sigma_z^2} \right] \]

(3.2)

where

- \( Q \) = mass of material emitted.
- \( x, y, \) and \( z \) are Cartesian coordinates with the positive \( x \) direction being taken downwind,
- \( t \) = time since the puff,
- \( \bar{u} \) = mean wind speed, and
- \( \sigma_x, \sigma_y, \sigma_z \) are the standard deviations of the material concentration distribution in the three coordinate directions relative to the puff center with origin \( (\bar{u}t, 0, B) \).

Integration of Equation (3.2) with respect to time and keeping \( \sigma_x \) constant as the puff passes any point yields the so-called Gaussian Plume equation (see Figure 3.3),

\[ X_2(x,y,z) = \int_{T=0}^{T} x(x,y,z,t) \ dt - \frac{Q'}{2\pi \sigma_y \sigma_z} \exp \left[ -\frac{y^2}{2\sigma_y^2} \right] \exp \left[ -\frac{(y-B)^2}{2\sigma_y^2} \right] + \exp \left[ -\frac{(z+B)^2}{2\sigma_z^2} \right] \]

(3.3)
Figure 3.3 Gaussian Diffusion from a Point Source
where

\( Q' \) = source emission rate (mass/time)

\( x \)-direction is downwind along the plume axis, and

\( \sigma_y, \sigma_z \) are standard deviations of the material concentration distribution in the y and z directions relative to the plume axis.

By the appropriate integrations along the \( y \)-axis, there results the equation for a finite line source (as might be the case for a highway)

\[
x_3 = \frac{2q}{(2\pi)^{5/2} \sigma_y \sigma_z} \int \exp \left( -\frac{1}{2} \frac{H^2}{\sigma_y^2} \right) \cdot \frac{P_2}{P_1} \left( \frac{1}{(2\pi)^{1/2}} \exp \left( -(P^2/2) \right) \right) dP
\]

where

\[
P_1 = \frac{y_1}{\sigma_y}
\]

\[
P_2 = \frac{y_2}{\sigma_y}
\]

\( q \) = mass emission rate per unit length of line

3.2 The ConnDEP Analysis

The analysis used by ConnDEP is described in Appendix II of the permit application. In order to predict the "worst case" expected carbon monoxide levels, concentrations are time-averaged for the queuing and free-flow conditions. This, the ConnDEP Intersection Model\(^{(20)}\), is

\[
CO = \frac{R}{C(COq) + \frac{C}{C}(CO_{ff})}
\]  \hspace{1cm} (3.5)
where

\[ CO = \text{Total CO (ppm)} \]
\[ R = \text{Red light time or, if available, queue time (seconds)} \]
\[ G = \text{Green light time or, if available, free flow time (seconds)} \]
\[ C = \text{Total cycle time (seconds)} \]
\[ CO_q = \text{CO from queued vehicles} \]
\[ CO_{fz} = \text{CO from free flow vehicles} \]

\[ CO = (q_q)(\phi_R)(C_q) + (0.1726)(q_q)(1-\phi_R) \cdot (V)(R)(F) \]  

(3.6)

where

CO = The Carbon Monoxide concentration in ppm.

\[ q_q = \text{The idling car CO emission rate in grams per vehicle minutes.} \]

This rate changes with the analysis year as shown in Table II.

\[ \phi_R = \text{The ratio of the red light time to total cycle time. (A dimensionless term)} \]

\[ C_q = \text{A unit idling car CO concentration which depends on the number of queued cars as shown on Table I. Units are ppm per emission rate in grams per vehicle minutes.} \]

\[ q_f = \text{The free flow CO emission factor in grams per mile. This factor also changes with the analysis year as shown in Table II.} \]

\[ V = \text{The hourly traffic volume for the approach lanes in thousands of vehicles per hour per lane.} \]

\[ E = \text{A unit free flow CO concentration. The units of this term are ppm per gram vehicles per mile per second (See Table I).} \]

\[ F = \text{The road speed correction factor which is dimensionless.} \]

Since the newer cars have characteristics different than earlier models, the method outlined in Table III must be used to
accurately calculate this variable. 

0.1726 = A conversion factor which has the units mile hours per thousand meter seconds.

Equation (3.6) is written to correspond with tabulated values of the variables \( C_q, Q_i, Q_f, E \) and \( y \). The tabulated factors associated with free flow are derived from the EPA HIWAY model and those associated with queuing are derived from the ConnDTP CO Queuing Model:

\[
Y = 1.26\bar{V} \sum_{i=1}^{N} \frac{Q(i)}{S(i)} \exp \left[ -\frac{\Delta_i^2 (10^{-6})}{110,000} \right]
\]  

(3.7)

where:

- \( Y \) = predicted CO concentration at a receptor point [ppm]
- \( \bar{V} \) = average wind speed [mph]
- \( i \) = identifies a particular idling car
- \( N \) = number of idling cars
- \( Q(i) \) = CO emission rate (grams/minute)
- \( S(i) \) = distance from tailpipe to receptor [feet]
- \( \Delta_i \) = difference between the average wind direction and the position vector of the tailpipe [degrees]

Equation (3.7) is empirical and, as described in (21), was developed using the results of field testing. The fieldwork consisted of measuring the concentration at a point downwind from a number of idling cars. The predicted versus observed CO concentrations given in Figure 3.4 are from 32 tests that fell within limits established in (21):
Figure 3.4 COURSE model prediction vs. monitored data for idling car tests

[After (21)]
1. $\bar{V}$ from 2 to 8 mph
2. perpendicular receptor distances from 10 to 30 meters, and
3. wind angle with queue from 0 to $\pi$ radians

It may be seen that, for $\Delta = 0^\circ$ (i.e., wind blowing in a line directly from the tailpipe to the receptor), Equation (3.7) is linear in wind speed. This seems to be contrary to the classic Gaussian plume model, Equation (3.3). The equation also includes no allowance for the diffusion coefficients, $\sigma_y$ and $\sigma_z$. These are based on the Pasquill turbulence category (which reflects atmospheric conditions) and the distance downwind. The Connecticut equation has apparently been derived for the "worst-case" condition.

A computer program for this equation has been written based on the geometry shown in Figure 3.5. The effect of the several variables is shown in Figures 3.6 through 3.9. In all cases the receptor location is taken at ten meters from the edge of the nearest lane of traffic, and multiple lanes are assumed to be equally loaded. The origin of the coordinate system is taken at the rear end of the last vehicle in line and the rate of emission is taken at 14.1 grams per minute. Several observations may be made. First, the maximum CO concentration, while a function of the combination of receptor location and wind angle, is essentially independent of receptor location, as long as the receptor is located at any point between the tailpipes of the first and last vehicles in line.
Location of Vehicles

Direction of Travel→

Wind Speed = 8 mph

Figure 3.6 Variation in Maximum CO Concentration with Receptor Location
Figure 3.7 Variation in Wind Angle Yielding Maximum CO Concentration with Receptor Location
Figure 3.9  Worst Case CO Concentration vs. Total Number of Queued Vehicles, N

Emission Rate = 14.1 gpm
Wind Speed = 8 mph
Second, the relationship between receptor location and the wind angle yielding the maximum CO concentration for that receptor location appears to be linear.

Third, for a given receptor location, the relationship of CO concentration to wind angle appears to be a bell-shaped curve.

Shown in Figure 3.9 is a plot of the worst-case CO concentration (based on the ComDEP model) versus the total number of queued vehicles for one and two-lane approaches. The emission rate has been taken at 14.1 grams per minute, the wind speed at 8 mph, and the receptor is located at 10 meters from the edge of the closest travel lane. As would be expected, the CO concentration increases with an increase in the number of vehicles queued. However, the increase is not linear. It may also be seen that, for a given number of queued vehicles, the concentration from two lanes is greater than that from an equal number of vehicles queued in one lane.
A. QUEUING

Although queuing theory has many applications, our considerations center on vehicular traffic and an introductory example may be taken from that field. Consider a toll station on a turnpike. Vehicles arrive, select a waiting line, wait, pay and depart. This simple example demonstrates the elements common to all queuing problems:

1. A pattern of arrivals specified by the average arrival rate and a statistical description of the variation in the arrival times;
2. A pattern of service time specified by the average service rate, a statistical description of the variation in service times, and the number of service channels;
3. A characteristic queue discipline, e.g. "first come first served."

We will adopt a shorthand notation described in (47), which takes the form of $s/b/c$. Letters denoting the statistical distributions of the arrival and service times are substituted for $a$ and $b$ respectively. A number designating the number of service channels is substituted for $c$. The letters designating the statistical distributions are $M$ (exponential), $D$ (deterministic), $G$ (general), $E_k$ (Erlang). These terms will be explained in this section.

The toll station example might be designated $M/M/3$, meaning exponential (random) arrivals, exponential service times and 3 booths in operation.

After a discussion of general traffic flow characteristics, this section discusses queuing analysis; first by classical techniques and then by simulation.
In both cases the discussion generally proceeds from the simplest case of a
priority (yield or stop-sign) intersection through fixed-time and actuated
traffic signals to systems of intersections.

4.1 Arrival and Headway Patterns

Since the time period of interest for the emissions analysis is eight
hours, consider a typical record of vehicle arrivals at a point on a highway
for a period of a day. Figure 4.1 shows the pattern of arrivals measured
by an automatic counter on an "on-ramp" to southbound I-91 at Morgan Street
in Hartford, on Tuesday, April 27, 1976.

Not surprisingly, morning and evening commuter peaks are apparent.
Although there is no way of knowing from the data as presented, we might
assume that arrivals during any given hour are "random". By this is meant
that the probability of an arrival in any small time interval is independent
of the occurrence or non-occurrence of an arrival in the previous time
interval. Thus intervals A and B (Figure 4.2), of equal length, have equal
probability of an arrival.

4.1.1 Poisson Arrivals

Under the assumption of random arrivals, it can easily be shown that

\[ P(x) = \frac{(\lambda t)^x e^{-\lambda t}}{x!} \]  

(4.1)

where

\[ P(x) = \text{probability of exactly } x \text{ arrivals during any time interval of }
\text{length } t, \]
At the beginning of the flow period, there are no arrivals. The flow is zero, and the probability of no arrivals is 1. As time progresses, the number of arrivals increases, and the flow also increases. The probability of a certain number of arrivals is given by the Poisson distribution, which is a discrete probability distribution that expresses the probability of a given number of events occurring in a fixed interval of time or space if these events occur with a known constant mean rate and independently of the time since the last event.

The cumulative distribution function gives the probability of K or fewer arrivals:

\[ F(k) = \sum_{x=0}^{k} \frac{e^{-\lambda t} \lambda^x}{x!} \]

for \( k = 0, 1, 2, \ldots \)

where \( \lambda \) is the average rate of arrival and \( t \) is the time interval.

**Figure 4.1** Variation in Volume by Hour of Day on Morgan Street Entrance Ramp to Southbound 1-91 in Hartford on April 27, 1976

Integration of equation (4.4) yields the "probability density function" for \( h(t) \):

\[ h(t) = \frac{dF}{dt} = \lambda e^{-\lambda t} \]

for \( t \geq 0 \).

**Figure 4.2** Fundamental Property of Completely Random Arrivals
\[ f(h) = \lambda e^{-\lambda h} \quad (4.5) \]

The distribution characterized by Equation (4.5) is called the "exponential" for obvious reasons. It is important for present purposes because of its description of headways and because it is often taken as representing the distribution of service times. Note that whereas Equation (4.1) was a function of the discrete variable \( x \), Equation (4.5) gives the distribution of the continuous variable \( h \) as a function of the continuous variable \( t \).

4.1.3 Pearson Type III and Erlang Distributions

A variety of more general distributions have forms similar to the basic exponential. For example, the so-called "Pearson Type III" distribution is given by:

\[ P(t) = \frac{1}{\beta \Gamma(K)} \left( \frac{t-a}{\beta} \right)^{K-1} \exp \left( -\frac{t-a}{\beta} \right) \quad (4.6) \]

where

- \( a \) is a location parameter \((a \geq 0)\),
- \( K \) is a shape parameter \((K > 0)\),
- \( \beta \) is a scale parameter \((\beta > 0)\), and
- \( \Gamma(K) \) is the gamma function of \( K \).

A better feel for the significance of Equation (4.6) may be gained by the following simplifications. First, let the location parameter, \( a \), equal zero. Thus
\[ P(t) = \frac{1}{\beta T(K)} \left[ \frac{t}{\beta} \right]^{K-1} e^{-\left( t/\beta \right)} \]  
\hspace{2cm} (4.7) 

or

\[ P(t) = \frac{K-1}{\beta^2 T(K)} e^{-\left( t/\beta \right)} \]  
\hspace{2cm} (4.8) 

Letting \( \beta = T/K \), there results the **gamma distribution**:

\[ P(t) = \left[ \frac{Kt}{T} \right]^{K-1} \frac{K}{T(K)} e^{-\left( tK/T \right)} \]  
\hspace{2cm} (4.9) 

Letting \( K = 1 \) gives

\[ P(t) = \frac{1}{T} \cdot e^{-\left( t/T \right)} \]  
\hspace{2cm} (4.10) 

which may be seen to be the exponential distribution Equation (4.5) if \( T \) is interpreted as

\[ T = \frac{1}{\lambda}, \]  
\hspace{2cm} (4.11) 

the mean of the headway distribution.

The name given to the gamma distribution where \( K \) is restricted to integer values is the **Erlang** and it is widely used in traffic theory. The parameter \( K \) may be considered a measure of randomness, with a value of one associated with random data and increasing values indicating increasing non-randomness. In fact the variance of the Erlang distribution is given by

\[ \sigma^2 = \frac{\pi^2}{K} \]  
\hspace{2cm} (4.12)
Note that $K$ may be estimated from the mean and variance of the observed data:

$$K = \frac{T^2}{S^2}$$  \hfill (4.12a)

where

$K$ = estimate of $K$,

$T$ = mean of observed intervals, and

$S^2$ = variance of observed intervals.

Thus as the data cluster about the mean, the value of $K$ increases. Lacking any better information for a particular case under consideration, Figure 4.3, from Drew (44), may be used to estimate $K$.

The effect of varying the Erlang parameter may be seen in Figure 4.4, from Wohl and Martin (110). It should be noted that the probability shown is that of waiting time $t$ for the $K$th arrival. Thus $K = 4$ gives the probability of waiting time $t$ for the 4th arrival.

4.1.4 Modifications to the Exponential Form

Additional modifications to the exponential form in order to more closely fit observed data have been suggested by a number of investigators. For example, if for the case represented by Equation (4.4), we wish the probability of a headway less than $t$, we have the complementary relationship:

$$P(h < t) = 1 - e^{-(t/T)}$$  \hfill (4.13)

where we have used the "law of total probability."
FIGURE 4.3 Approximate value of Erlang k as related to the freeway outside lane value. [From Traffic Flow Theory and Control by Drew. Copyright (c) 1968 by McGraw-Hill. Used with permission of McGraw-Hill.]
Figure 4.4 Erlang Probabilities for Fixed $\lambda$ Value and Various Integer $k$ Values
Equation (4.13) is plotted in Figure 4.5. An immediate criticism of the formulation is suggested, i.e., the fact that it allows exceedingly small headways, approaching zero. If passing is impossible and traffic is heavy, this weakness becomes more serious. Thus, the shifted exponential has been suggested to allow for a certain minimum headway \( \tau \). This is shown in Figure 4.6.

Additional models of headways have been proposed, among them are:\(^{(47)}\)

1. the lognormal distribution which considers the logarithm of the variable to be normally distributed;

2. composite headway models which consider the headways consisting of two subpopulations (one of freely flowing cars and one of cars constrained by traffic ahead); and

3. the "hyper-Erlang" distribution obtained by shifting the Erlang distribution to the right to account for the minimum headway.

Although many of these seem particularly applicable in certain cases, for the present time we will content ourselves with the exponential and Erlang distributions. For later consideration we observe [(47) p. 31].

...Newell has shown that delays are relatively insensitive to the form of the distribution of the arriving traffic. Thus, if the objective is simply the computation of delays, the simplest (i.e., the negative exponential) should be used. If, however, the objective is the determination of gaps for, say, crossing purposes, a more faithful distribution may be needed.

4.1.5 Critical Gap

In his work on developing a warrant for stop sign control, Raff\(^{(87)}\) proposed the use of "critical lag" \( L \), defined as
FIGURE 4.5 Probability of headways less than t, with T = 1 sec.  
[After (47)]

FIGURE 4.6 Shifted exponential distribution to represent the probability of headways less than t with a prohibition of headways less than T. (Average of observed headways is T)  
[After (47)]
"The critical lag \( L \) is the size lag which has the property that the number of accepted lags shorter than \( L \) is the same as the number of rejected lags longer than \( L \)."

Thus the critical lag (or acceptable gap) is a measure of the size of gap required before a driver will attempt to cross or merge with a stream of traffic. The value is sensitive to the conditions surrounding the site and the results of Raff's field observations for one intersection are shown in Figure 4.7.

The results of a study by DeLeuw, Cather and Company reported in (110) are shown in Figure 4.8. It may be seen that, in general, drivers accept a smaller gap at a yield than at a stop sign except during the peak period when the reverse seems to be true.

Table 4.1 gives observed values of the critical gap at a number of intersections as reported in (59).

4.2 Classical Techniques

With the background of statistical theory from the preceding paragraphs it is now possible to discuss queuing characteristics that might be expected under projected conditions of arrivals and servicing.

4.2.1 Stop or Yield-Sign Control

As mentioned earlier, an analysis of queuing requires specification of arrival rate and service rate. Although number of servers and queue discipline are also required in the general case, for present purposes the
FIGURE 4.7 Distribution of accepted and rejected lags at Orange and Willow Streets, New Haven, Connecticut.

[From *A Volume Warrant for Urban Stop Signs* by Morton S. Raff. Copyright 1950 (c) by the Eno Foundation. Used with permission of the Eno Foundation and the author.]
FIGURE 4.8 Gap and lag acceptance, yield-versus stop-sign control.

[From Effect of Control Devices on Traffic Operations
NCHRP Interim Report No. 11, 1964]
<table>
<thead>
<tr>
<th>Situation-Location</th>
<th>Gap (sec)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor street with major one-way (England)</td>
<td>8.0</td>
<td>1</td>
</tr>
<tr>
<td>Minor street with stop sign (New Haven)</td>
<td>6.1</td>
<td>2</td>
</tr>
<tr>
<td>Open intersection, no control (New Haven)</td>
<td>2.57</td>
<td>2</td>
</tr>
<tr>
<td>Blind intersection, no control (Harford)</td>
<td>2.82</td>
<td>2</td>
</tr>
<tr>
<td>Stop sign, 39 ft one-way street</td>
<td>4.6</td>
<td>3</td>
</tr>
<tr>
<td>Stop sign, 34 ft one-way through street</td>
<td>4.7</td>
<td>3</td>
</tr>
<tr>
<td>Stop sign, 41 ft two-way street</td>
<td>5.7</td>
<td>3</td>
</tr>
<tr>
<td>Stop sign, 63 ft two-way street</td>
<td>6.9</td>
<td>4</td>
</tr>
<tr>
<td>Yield sign</td>
<td>6.2</td>
<td>4</td>
</tr>
<tr>
<td>Stop sign</td>
<td>6.3</td>
<td>4</td>
</tr>
<tr>
<td>Stop sign (through vehicles)</td>
<td>5.8</td>
<td>5</td>
</tr>
<tr>
<td>Stop sign (left turn)</td>
<td>6.2</td>
<td>5</td>
</tr>
<tr>
<td>Stop sign (right turn)</td>
<td>5.4</td>
<td>5</td>
</tr>
<tr>
<td>Left turn through opposing</td>
<td>4.25</td>
<td>6</td>
</tr>
<tr>
<td>Left turn through opposing (moving)</td>
<td>4.4</td>
<td>7</td>
</tr>
<tr>
<td>Left turn through opposing (from stop)</td>
<td>4.6</td>
<td>7</td>
</tr>
<tr>
<td>60° intersection—stop sign (right turn)</td>
<td>5.5</td>
<td>8</td>
</tr>
<tr>
<td>60° intersection—stop sign (left turn)</td>
<td>7.0</td>
<td>8</td>
</tr>
<tr>
<td>T intersection—stop sign (right turn)</td>
<td>5.7</td>
<td>8</td>
</tr>
<tr>
<td>T intersection—stop sign (left turn)</td>
<td>7.2</td>
<td>8</td>
</tr>
<tr>
<td>Stop sign—into one-way street (right turn)</td>
<td>4.0</td>
<td>8</td>
</tr>
<tr>
<td>Stop sign—into one-way street (left turn)</td>
<td>5.6</td>
<td>8</td>
</tr>
</tbody>
</table>

Sources:

**TABLE 4.1 Observed values of critical gaps at urban intersections.** (After (39))
number of servers will always be one and the queue discipline will always be "first in first out".

The characteristic of ultimate interest for the present study of emissions is the number of vehicles idling. It is important to note that the computations in queuing theory refer to the number in the system and not the number in the queue. Considering a line of vehicles at a stop sign, the first vehicle is considered to be "in service" and the following vehicles are "in queue". The number of vehicles in the system at any one time is then equal to the number in the queue plus the first car. However, the average number in the system is not, in general, equal to the average number in the queue plus one. Rather, it is equal to the average number in the queue plus the average number being served.

In the paragraphs to follow, a rather detailed derivation* is given for the case M/M/1. Only results are given for more complicated cases. At this point we observe that the identification of the appropriate distribution of service times is not at all straightforward since it depends on the headway distribution of crossing vehicles and the "gap acceptance" function for served vehicles (in the case of stop or yield sign control) or the time required to enter an intersection (in the case of signalization).

Consider an intersection of major and minor streets controlled by a stop sign on the minor street. Assume that vehicles arrive randomly on the

---

*The derivation given closely parallels that in (47).
minor approach and depart randomly from the stop sign. The mean arrival rate will be taken to be \( \lambda \) vehicles per time unit.

Each vehicle on the minor leg must wait some amount of time at the stop line for an acceptable gap in the conflicting traffic flow. The amount of time spent at the stopline is taken as the service time and it should be noted that this does not include any time waiting to get to the head of the line. Let the mean service rate be \( \mu \) and let it follow a negative exponential distribution.

If the mean arrival rate is \( \lambda \), the probability of a vehicle arriving in a time interval \( \Delta t \) is \( \lambda \Delta t \). (\( \Delta t \) is assumed to be so small that only one vehicle may arrive, and only one vehicle may depart during it. The same vehicle may not both arrive and depart in \( \Delta t \).)

In this situation, there are only two possibilities: either one vehicle arrives during \( \Delta t \) or one doesn't. Since the sum of all the probabilities must equal unity:

\[
\lambda \Delta t + P_{na} = 1 \tag{4.14}
\]

where

\[P_{na} = \text{the probability of no arrivals during } \Delta t.\]

Thus

\[P_{na} = 1 - \lambda \Delta t \tag{4.15}\]
The probability \( P_n \) of \( n \) vehicles being in the system at a given moment must now be found.

Suppose that, at time \( t \), there are \( n \) vehicles in the system under consideration, where \( n \geq 0 \). In order for there to be \( n \) vehicles in the system at time \( t + \Delta t \) (where \( \Delta t \) is defined in the same way as above), one of the four possible combinations of events shown in Table 4.2A must occur.

An examination of Table (4.2A) will show that if one of Cases A, B, C or D occurs, none of the others can occur. Thus, they are mutually exclusive, and the probability \( P_n \) of \( n \) cars being in the system at time \( t + \Delta t \) is the sum of the probabilities of A, B, C and D. Algebraically,

\[
P_n(t + \Delta t) = P_A + P_B + P_C + P_D,
\]

(4.16)

where

- \( P_A \) = the probability of Case A occurring,
- \( P_B \) = the probability of Case B occurring,
- \( P_C \) = the probability of Case C occurring, and
- \( P_D \) = the probability of Case D occurring.

Considering Case B, by our assumption of randomness, the occurrence of \( n \) vehicles being in the system at time \( t \) has absolutely no influence on the probability of a vehicle arriving during \( \Delta t \), and no influence on the probability of a vehicle departing during \( \Delta t \). Furthermore, the arrival of a vehicle during \( \Delta t \) has nothing to do with the probability of \( n \) cars being
<table>
<thead>
<tr>
<th>Case</th>
<th>Statement</th>
<th>Probability (n=0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>A.1) There are n vehicles in the system at time t</td>
<td>( P_n(t) )</td>
</tr>
<tr>
<td></td>
<td>1.2) No vehicle arrives during ( \Delta t )</td>
<td>( 1 - \lambda \Delta t )</td>
</tr>
<tr>
<td></td>
<td>1.3) No vehicle leaves during ( \Delta t )</td>
<td>( 1 - \mu \Delta t )</td>
</tr>
<tr>
<td>B</td>
<td>B.1) There are n vehicles in the system at time t</td>
<td>( P_n(t) )</td>
</tr>
<tr>
<td></td>
<td>B.2) One vehicle arrives during ( \Delta t )</td>
<td>( \lambda \Delta t )</td>
</tr>
<tr>
<td></td>
<td>B.3) One vehicle leaves during ( \Delta t )</td>
<td>( \mu \Delta t )</td>
</tr>
<tr>
<td>C</td>
<td>C.1) There are (n + 1) vehicles in the system at time t</td>
<td>( P_{n+1}(t) )</td>
</tr>
<tr>
<td></td>
<td>C.2) No vehicle arrives during ( \Delta t )</td>
<td>( 1 - \lambda \Delta t )</td>
</tr>
<tr>
<td></td>
<td>C.3) One vehicle leaves during ( \Delta t )</td>
<td>( \mu \Delta t )</td>
</tr>
<tr>
<td>D</td>
<td>D.1) There are (n - 1) vehicles in the system at time t</td>
<td>( P_{n-1}(t) )</td>
</tr>
<tr>
<td></td>
<td>D.2) One vehicle arrives during ( \Delta t )</td>
<td>( \lambda \Delta t )</td>
</tr>
<tr>
<td></td>
<td>D.3) No vehicle leaves during ( \Delta t )</td>
<td>( 1 - \mu \Delta t )</td>
</tr>
</tbody>
</table>
in the system at time \( t \), and has no influence on the probability of a vehicle departing during \( \Delta t \). Finally, the departure of a vehicle during \( \Delta t \) has no effect on the probability of a car being in the system at time \( t \), nor does it in any way affect the probability of a vehicle arriving during \( \Delta t \). It may be concluded, therefore, that the three events in Case B are independent. A similar examination of Cases A, C, and D will yield the same conclusion.

Therefore

\[
P_A = P_n(t)[1 - \lambda \Delta t][1 - \mu \Delta t] = P_n(t)[1 - \lambda \Delta t - \mu \Delta t + \mu \lambda (\Delta t)^2],
\]

\[
P_B = P_n(t)[\lambda \Delta t][\mu \Delta t] = P_{n+1}(t)[\mu \lambda (\Delta t)^2],
\]

\[
P_C = P_{n+1}(t)[1 - \lambda \Delta t][\mu \Delta t] = P_{n+1}(t)[\mu \Delta t - \mu \lambda (\Delta t)^2],\text{ and}
\]

\[
P_D = P_{n-1}(t)[\lambda \Delta t][1 - \mu \Delta t] = P_{n-1}(t)[\lambda \Delta t - \lambda \mu (\Delta t)^2].
\]

By substitution into Equation (4.16),

\[
P_n(t + \Delta t) - P_n(t) = -P_n(t)[\mu \Delta t - \mu \lambda (\Delta t)^2] + P_{n+1}(t)[\lambda \Delta t - \mu \lambda (\Delta t)^2].
\]

Thus,

\[
\frac{P_n(t + \Delta t) - P_n(t)}{\Delta t} = -\frac{P_n(t)}{n} - \frac{P_n(t)}{n} + 2P_n(t)[\mu \lambda (\Delta t)^2] + P_{n+1}(t)[\lambda \Delta t - \mu \lambda (\Delta t)^2] + P_{n-1}(t)[\lambda \Delta t - \lambda \mu (\Delta t)].
\]

Letting \( \Delta t \to 0 \) leads to the differential equation:

\[
\frac{dP_n(t)}{dt} = -P_n(t)(\lambda + \mu) + P_{n+1}(t)\mu + P_{n-1}(t)\lambda. \tag{4.17}
\]
Table (4.2A) and Equation (4.17) are valid only when $n > 0$.

Obviously, $n$ will never be less than zero, but it will, on occasion, be equal to zero.

Special conditions exist when $n = 0$. In order for there to be no vehicles in a system at time $t + \Delta t$, one of the following combinations of events must occur.

<table>
<thead>
<tr>
<th>Case</th>
<th>Statement</th>
<th>Probability $(n=0)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>E.1) There are no vehicles in the system at time $t$</td>
<td>$P_0(t)$</td>
</tr>
<tr>
<td></td>
<td>E.2) No vehicle arrives during $\Delta t$</td>
<td>$1 - \lambda \Delta t$</td>
</tr>
<tr>
<td>F</td>
<td>F.1) There is no vehicle in the system at time $t$</td>
<td>$P_1(t)$</td>
</tr>
<tr>
<td></td>
<td>F.2) One vehicle leaves during $\Delta t$</td>
<td>$\mu \Delta t$</td>
</tr>
<tr>
<td></td>
<td>F.3) No vehicle arrives during $\Delta t$</td>
<td>$1 - \lambda \Delta t$</td>
</tr>
</tbody>
</table>

Examination of Table (4.2B) reveals that Cases E and F are mutually exclusive; therefore the addition rule may be applied, and

$$ P_0(t + \Delta t) = P_E + P_F, \quad (4.18) $$

where

- $P_E$ = the probability of Case E occurring and
- $P_F$ = the probability of Case F occurring.
Further examination of Table (4.2B) shows that the two statements of 
Case E are independent, and that the three statements of Case F are independent. 
Thus, by the multiplication rule,

\[ P_E = P_0(t)[1 - \lambda \Delta t] = P_0(t) - P_0(t)\lambda \Delta t \]

and

\[ P_F = P_1(t)[\mu \Delta t][1 - \lambda \Delta t] = P_1(t)[\mu \Delta t - \lambda \mu (\Delta t)^2] \]

By substitution into Equation (4.18)

\[ P_E(t + \Delta t) = P_0(t) - P_0(t)\lambda \Delta t + P_1(t)[\mu \Delta t - \lambda \mu (\Delta t)^2] \]

or

\[ \frac{P_0(t + \Delta t) - P_0(t)}{\Delta t} = P_0(t)\lambda + P_1(t)[\mu - \lambda \mu] \]

And, as before, by letting \( t \) approach zero,

\[ \frac{dP_0(t)}{dt} = -P_0(t)\lambda + P_1(t)\mu \quad (4.19) \]

Equations (4.17) and (4.19) may be simplified considerably if \( \lambda \) and 
\( \mu \) are assumed to remain constant over time. Under this assumption, there 
is no reason to think that \( P_n \) would ever change. If the average arrival rate 
and the average departure rate do not change with time, it seems logical 
that \( P_n \) would be the same at this instant as it was last Tuesday at 2:05 
p.m., and the same as it will be next Wednesday at 10:29 a.m.
We may therefore assume that, under these conditions, \( P_n \) is independent of time. If \( P_n \) is independent of time, then the rate of change of \( P_n \) with respect to time will be zero. Thus

\[
\frac{dP_n}{dt} = 0 \quad (4.20)
\]

and

\[
\frac{dP_0}{dt} = 0. \quad (4.21)
\]

Obviously, the assumption that \( \lambda \) and \( \mu \) do not change with time places a severe restriction on the model. \( \lambda \) and \( \mu \) do indeed change with time, as evidenced by peak hour congestion. However, in order to derive a workable formula, this assumption must be made.

By substitution of Equation (4.20) into Equation (4.17) and substitution of Equation (4.21) into Equation (4.19), we obtain the recursive relations:

\[
-P_\theta (\lambda + \mu) + P_{n+1} \mu + P_{n-1} \lambda = 0 \quad (n>0) \quad (4.22)
\]

and

\[
-\lambda P_0 + \mu P_1 = 0 \quad (n=0) \quad (4.23)
\]

Through use of Equations (4.22) and (4.23), it is possible to predict the probability \( P_n \) of a system of size \( n \), \( n \geq 0 \).

By identity,

\[
P_0 = P_0. \quad (4.24)
\]
By using Equation (4.23)

\[ -\lambda p_0 + \mu p_1 = 0 , \]
\[ \lambda p_0 = \mu p_1 \text{ and} \]
\[ p_1 = \left(\frac{\lambda}{\mu}\right)p_0 . \]  
(4.25)

By letting \( n = 1 \), and substituting the value of \( p_1 \) found in Equation (4.25), Equation (4.22) becomes

\[ 0 = -p_1 (\lambda + \mu) + \mu p_2 + \lambda p_0 , \]

from which

\[ 0 = \left[ \frac{\lambda^2 + \lambda \mu}{\mu} \right] p_0 + \mu p_2 + \frac{\lambda \mu}{\mu} p_0 , \]

\[ - p_2 = \left[ \frac{-\lambda^2 + \lambda \mu}{\mu} \right] p_0 + \frac{\lambda \mu}{\mu} p_0 , \]

and

\[ -\mu p_2 = \left[ \frac{-\lambda^2}{\mu} \right] p_0 . \]

Thus,

\[ p_2 = \left[ \frac{\lambda^2}{\mu} \right] p_0 . \]  
(4.26)

by letting \( n = 2 \), substituting the value of \( p_2 \) found in Equation (4.26), and substituting the value of \( p_1 \) found in Equation (4.25), Equation (4.22) becomes
\[-(\alpha + \mu)I_{2} + \mu P_{3} + \gamma P_{1} = 0\]

or

\[\frac{\lambda^{3}}{\mu^{3}} P_{0} - \frac{\lambda^{2}}{\mu} P_{0} + \mu P_{3} + \frac{\lambda^{2}}{\mu} P_{0} = 0\]

Thus,

\[P_{3} = \frac{\lambda^{3}}{\mu^{3}} P_{0} = \left[\frac{\lambda}{\mu}\right]^{3} P_{0}\]

(4.27)

\[P_{4}, P_{5}, P_{6}, \ldots, P_{n}\] may be found in the same way that \(P_{2}\) and \(P_{3}\) were found.

In general,

\[P_{n} = \left[\frac{\lambda}{\mu}\right]^{n} P_{0}\]

(4.28)

Thus, if \(P_{0}\) can be found, \(P_{n}\) for any \(n > 0\) can be found. \(P_{0}\) may be obtained from the following procedure. Since

\[P_{0} + P_{1} + P_{2} + P_{3} + \ldots + P_{n} = 1\]

or

\[\sum_{n=0}^{\infty} P_{n} = 1\]

(4.29)

substituting for Equation (4.28) yields

\[\sum_{n=0}^{\infty} (\lambda/\mu)^{n} P_{0} = 1\]

Because \(P_{0}\) does not change with a change in \(n\), it may be taken out of the summation notation, leaving
\[ P_0 \sum_{n=0}^{\infty} \left( \frac{\lambda}{\mu} \right)^n = 1. \]  

(4.30)

Using the well-known expression for the sum of an infinite series,

\[ \sum_{n=0}^{\infty} \left( \frac{\lambda}{\mu} \right)^n = \frac{1}{1 - \left( \frac{\lambda}{\mu} \right)}, \]

Equation (4.30) gives

\[ P_0 = 1 - \rho, \]  

(4.31)

where the traffic intensity, \( \rho \), is given by

\[ \rho = \frac{\lambda}{\mu}. \]

We may note in passing that, since \( P_0 \) is the probability that the system is empty and \( 1 - P_0 \) is the probability that it is occupied, the traffic intensity, \( \rho \), expresses the fraction of time that the system is busy.

By definition, the expected (average) number of vehicles in the system, \( \bar{n} \), is given by

\[ \bar{n} = \sum_{n=0}^{\infty} nP_n. \]

Substituting from Equations (4.29) and (4.31)

\[ \bar{n} = P_0 \sum_{n=0}^{\infty} n\rho^n, \]

or

\[ \bar{n} = (1 - \rho)(\rho + 2\rho^2 + 3\rho^3 + \ldots). \]
or
\[ \bar{n} = (1 - \rho) \left( \frac{\rho}{(1 - \rho)^2} \right) \]

or
\[ \bar{n} = \frac{\rho}{(1 - \rho)} \cdot \quad (\rho < 1) \] (4.32)

Thus, Equation (4.32) gives the average number of vehicles in the system as a function of traffic intensity and is valid for the case of Poisson arrivals and a negative exponential distribution of service times.

For Poisson arrivals and a general distribution of service times, Cox and Smith (31) give
\[ \bar{n} = \rho + \frac{\rho^2(1 + C_b^2)}{2(1 - \rho)} \] (4.33)

where \( C_b \), the fractional coefficient of variation of service times is given by
\[ C_b^2 = \frac{\mu_b^2}{\sigma_b^2} \] (4.34)

and \( \sigma_b \) is the standard deviation of the service time distribution.

For the particular case of the Erlang distribution and recalling Equation (4.12):
\[ \sigma_b^2 = \frac{1}{K\lambda^2} \] (4.35)

and thus
\[ C_b^2 = \frac{1}{K} \].
Therefore, for Poisson arrivals and Erlang service times, the average number of vehicles in the system is given by

$$\bar{n} = \rho + \frac{\rho^2(1 + (1/\lambda))}{2(1 - \rho)} \quad (4.36)$$

In the use of the classical queuing equations, it should be kept in mind that the parameter of interest, the average system length, is very sensitive to small fluctuations in traffic intensity $\rho$ for values of $\rho$ above about 0.85. This is vividly illustrated in Figure 4.9. An example will illustrate the implications of this.

Suppose that arrivals and service times are random so that Equation (4.32) holds. Assume that

$$\lambda = 500 \text{ vph} = .139 \text{ vps}$$

and that the average service time per vehicle $\bar{s}$ is 6 seconds.

Thus

$$\mu = \lambda/6 = .167 \text{ vps}$$

$$\rho = \lambda/\mu = .139/.167 = .832$$

so that

$$\bar{n} = \rho/(1 - \rho) = .832/(1 - .832) = 4.95$$

But suppose the average service time per vehicle was 6.5 instead of 6 (8% error) and the average rate of arrivals was 520 instead of 500 (4% error). Both of these seem to be well within the range of the errors in estimating. Then
FIGURE 4.9 Average number and variance of the number in system as a function of traffic intensity.

[After (47)]
\[ \lambda = .144 \text{ vps} \]
\[ \mu = .154 \text{ vps} \]

therefore, \[ \rho = 0.940 \]

and \[ \bar{n} = .940/(1 - .940) = 15.7 \]

Thus, the selection of input parameters is very important for high values of traffic intensity.

It is likely, however, that such high values of \( \rho \) would indicate signalization. The Manual on Uniform Traffic Control Devices (101) contains a set of eight warrants for signalization. The first of these is based on the minimum vehicular volume that would exist on each of any eight hours of an average day. The critical volumes are shown in Table 4.3. Note that, according to this warrant, the highest volumes that would be tolerated under stop sign control would be 600 vph on the major street and 150 vph per lane on the minor street. For this case, if we used the simple approach above, we would have

\[ \lambda = 150 \text{ vph} = .042 \text{ vps} \]

A reasonable value of average service time might be \( \bar{s} = 6 \) seconds.

Therefore \[ \mu = .167 \text{ vps} \]

and \[ \rho = \lambda/\mu = .252 \]

so that \[ \bar{n} = .252/(1 - .252) = .34 \]
Suppose now that gross errors were made in predicting the input volume and service time so that the actual values should have been

\[ \lambda = 200 \text{ vph} \times 0.056 \text{ vps} \ (32\% \text{ error}) \]

and

\[ e = 10 \text{ seconds} \ (67\% \text{ error}). \]

Thus

\[ p = 0.56 \]

so that

\[ n = 0.56/(1 - 0.56) = 1.3 \]

which is still far below that which might reasonably be expected to cause a violation.

In recognition of the above consideration, the proposed worksheets and charts provide cautionary comment when \( \rho \) exceeds 0.85.

---

**Table 4.3**

Minimum Vehicular Volumes for Warrant 1

<table>
<thead>
<tr>
<th>Number of lanes for moving traffic on each approach</th>
<th>Vehicles per hour (total of both approaches)</th>
<th>Vehicles per hour on higher-volume minor-street approaches (one direction only)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major street</td>
<td>Minor street</td>
<td></td>
</tr>
<tr>
<td>1---------</td>
<td>1---------</td>
<td>500</td>
</tr>
<tr>
<td>2 or more--</td>
<td>1---------</td>
<td>600</td>
</tr>
<tr>
<td>2 or more--</td>
<td>2 or more--</td>
<td>600</td>
</tr>
<tr>
<td>1---------</td>
<td>2 or more--</td>
<td>500</td>
</tr>
</tbody>
</table>

The equations developed thus far assume, of course, that the service rate \( \mu \) is known or can be estimated. For a non-signalized intersection, the time
in service corresponds to that time the lead vehicle takes to find and use an acceptable gap in the crossing traffic. What follows is a brief summary of a derivation by Drew (40). The derivation was developed by Drew for freeway merges but is applicable here. It assumes Erlang headways and that any driver will accept any gap greater than or equal to the critical gap, $\tau$, and that $\tau$ is constant with time.

The probability $P(n)$ of any driver having to wait for $n$ intervals, each less than the critical gap $\tau$ is

$$P(n) = p^n(1 - p) \quad n = 1, 2, 3 \ldots$$  \hspace{1cm} (4.37)

where

$$p = P(r < \tau) = \int_{\tau}^\infty p(t) \, dt$$  \hspace{1cm} (4.38)

and $p(t)$ is the distribution of gaps in the major stream.

The expected number of intervals through which a driver has to wait will be

$$E(n) = \frac{p}{1 - p} = \frac{\int_{\tau}^\infty p(t) \, dt}{\int_{\tau}^\infty p(t) \, dt}.$$  \hspace{1cm} (4.39)

Now the sum of the time intervals of size less than the critical gap $\tau$ will be given by

$$\lambda \int_{0}^{\tau} t \cdot p(t) \, dt$$

so that the average length of interval less than $\tau$ is
Since the average time required for the vehicle waiting to cross will be the product of the expected number of intervals less than $\tau$ and the average length of interval less than $\tau$, we have, from Equations (4.39) and (4.40)
\[
\bar{s} = \frac{\lambda^T t^* p(t) \, dt}{\lambda^T p(t) \, dt}
\]  
(4.41)

For the case of Erlang headways in the main flow, and recalling Equation (4.9),
\[
p(t) = \frac{(Kt)^K e^{-\lambda t}}{(k - 1)!}
\]

we have
\[
\bar{s} = \frac{\lambda^T t \left[ \frac{(K-1) e^{-\lambda t}}{(k-1)} \right] \, dt}{\lambda^T \left[ \frac{(K-1) e^{-\lambda t}}{(k-1)} \right] \, dt}
\]  
(4.42)

For the simple case of a negative exponential distribution of headways, $K = 1$, and Equation (4.42) can be evaluated to give
\[
\bar{s} = \frac{1}{\lambda} \left( e^{\lambda \tau} - 1 - \lambda \tau \right)
\]  
(4.43)

For $K = 2$ there results
\[
\bar{s} = \frac{2 \lambda \tau - \lambda - 2(\lambda + 2(\lambda \tau)^2)}{\lambda(1 - 2\lambda \tau)}
\]  
(4.44)

and so on.
Drew has summarized these solutions in the form of a series of charts given in Figure 4.10. Each chart corresponds to a value of the Erlang parameter, is entered with the conflicting volume and the average delay to a crossing vehicle is read depending on the critical gap \( t \).

Since \( \bar{t} \) is the average time in service, the mean service rate, \( \mu \), will be given by

\[
\mu = \left( \frac{1}{\bar{t}} \right)
\]

(4.45)

and the previously developed equations may be used. If anything but the value 1 is used for the Erlang parameter, then an exponential distribution of service times is not appropriate and the more general Equation (4.33) should be used for the estimate of \( \bar{t} \).

Finally, we note that the foregoing assumes that only one vehicle may cross per gap.

One fairly simple attempt to handle the yield sign problem was made by Oliver and Bisbee (83) in 1961. They hypothesized that the queue length on the side street was a function of the flow rate on the major street.

Using the usual assumptions that a critical gap of \( t \) is needed for entry onto the major street, and that the side-street arrivals follow a Poisson distribution, and using the additional assumptions that

1) only one queued vehicle enters the major street during an acceptable gap,
Figure 4.10: Merging delay in terms of the freeway flow q, critical gap T, and Erlang constant a.

2) The major street's flow is not affected by the queue length on the minor street, and
3) Entries during acceptable gaps occur at the very beginning of that gap.

Oliver and Biase hypothesized that the average length on the side street is

\[ n = \frac{(q_A/q_B)^2 (1 - q_B e^{-t_1})}{e^{-t_1} - (q_A/q_B) e^{-t_2}} \]  

(4.46)

where \( q_A \) = flow rate on the side street
and \( q_E \) = flow rate on the major street.

Despite the limiting assumption that there is only one entry per acceptable gap, the authors maintain that the equation is reasonably good, especially when flow on the major street is heavy. Figure 4.11 is a plot of Equation (4.46) for a value of \( \tau = 5 \) seconds.

Weiss and Maradudin (109), in their consideration of yield sign delay, allowed for the fact that the acceptable gap for a moving vehicle is likely to be less than that of a stopped vehicle. Thus, they proposed a mean delay \( E(t) \) given by

\[ E(t) = \frac{q_2}{q} \left( 1 - e^{-q \tau_2} \right) + e^{-q \tau_1} (\tau_2 - \tau_1) - \tau_2 \]  

(4.47)

where \( \tau_1 = \) acceptable gap for a moving vehicle
\( \tau_2 = \) acceptable gap for a stopped vehicle
and \( \tau_1 \leq \tau_2 \)
Figure 4.11: Relationship of the Average Minor Stream Queue Length and the Major Stream Flow Rate

[After (15)]
Note that Equation (4.47) reduces to Equation (4.43) as a special case when \( \tau_1 = \tau_2 \) and also that Equation (4.43) is the more conservative.

While it might appear that Equations (4.43) through (4.46) could be used for the case of a two-way stop sign, the problem is complicated by the difference in acceptable gap and, probably more importantly, the move-up time required for a vehicle to come into service after the preceding vehicle has left. In most states, the following vehicle is required to come to a full stop whether or not there is an acceptable gap present. In fact, it becomes apparent that the physical characteristics of the specific intersection are highly important. Sight distance, grades, turning radii and (in the case of a merge) length and design of acceleration lane all greatly influence the mean service time.

Thus this case is fairly complicated and site-specific. Also, any intersection carrying volumes sufficient to result in a significant queue would, in most circumstances, be signalized. Therefore, even though there have been several elegant solutions proposed\(^{(45)}\), their use under the present circumstances is not justified and the simple approach of using the equation in the form of Equation (4.43) and adding three seconds to account for reaction and acceleration time is recommended.

4.2.2 Fixed - Time Traffic Signal

As was noted earlier, the simple Poisson process is an adequate description of arrivals under light flows. Under heavy traffic however,
deviations from the simple process are apparent and a compound Poisson process seems more appropriate. This deviation from the simple process may be quantified by considering the "index of dispersion" $I$ which may be thought of as

$$I = \frac{\sigma^2 \cdot N(t)}{At},$$

(4.48)

where $\sigma^2$ is the variance and $N(t)$ is the expected number of arrivals during the period $t$.

Recalling Equation (4.2) we note that $I$ for a simple Poisson process is 1. In measurements of actual traffic under conditions of heavy flow, however, Miller (74) has found values of $I$ approaching 2.

In consideration of the departure process, the general approach is to work with the "effective" red period $R$ which includes the actual red time and any lost time due to signal change-over. If the total cycle length is $C$ the effective green time $G$ becomes

$$G = C - R.$$

(4.49)

During the green period, for a single lane, the departures may be assumed to be independent and randomly distributed. However, the assumption of constant departure headways for straight-through vehicles of $s = 2.1$ seconds (56) is typically applied to this period. If right turns are given greater times and if left turns are prohibited, as observed by (73) "one could assume that the departure times have a distribution containing two jumps, one for each of the
classes of motorists." The delays to vehicles turning left may be considered separately from those in the main stream if there is a separate left-turn lane (not necessarily a separate phase), otherwise they may not since one vehicle turning left could block all the straight-through and right-turning vehicles for an entire phase.

The delay at a traffic signal is a complicated phenomenon. In classical queuing theory, individuals are given service in a certain prescribed manner (the simplest being first in first out), service periods are independent, identically distributed random variables and no individual may depart until service is completed. The distributions of the number of vehicles in the system and the busy period are easily obtainable only if the distributions of arrivals and service periods have an exponential tail. These conditions do not, in general, obtain at a traffic signal for the following reasons:

1. No departures at all are possible during the red periods.
2. With the exception of a single-lane approach, vehicles do not necessarily depart in order of arrival.
3. If there is no separate left-turn lane, service times are not independently distributed.
4. Under conditions of heavy flow, arrivals are not independent.

Much of the work on traffic signals has concentrated on the total delay to all vehicles during a cycle (as opposed to individual delay) since this is a measure of the total operational efficiency and, for the reasons cited above, is generally easy to obtain.
In order to gain a feel for the phenomenon, consider Figure 4.12 which depicts the number of vehicles in the system \( n(t) \) as a function of time. The total delay \( d \) during any interval from \( t \) to \( (t + \delta t) \) will be given by

\[
d = n(t) \delta t + o(\delta t) \tag{4.50}
\]

where \( o(\delta t) \) denotes a quantity of the order \( \delta t \). Therefore the total delay \( D \) during a cycle \( C \) is given by the area under the curve,

\[
D = \int_0^C n(t) \, dt \tag{4.51}
\]

As a rough approximation then, the expected individual delay \( \bar{d} \) would be given by \( D \) divided by the expected number of arrivals

\[
\bar{d} = \frac{1}{\lambda C} \int_0^C n(t) \, dt \tag{4.52}
\]

Finally, the average number in the system, \( \bar{n} \), would be given by

\[
\bar{n} = \frac{D}{C}
\]

or

\[
\bar{n} = \frac{\lambda C}{C}
\]

so that

\[
\bar{n} = \bar{d} \lambda \tag{4.53}
\]

While the process is obviously probabilistic, some appreciation for its general nature can be achieved through a continuum approach. Thus May (69) assumes uniform arrivals at a rate \( \lambda \), being serviced at a uniform rate \( s \), and derives expressions for various queuing characteristics.
Figure 4.12  The Variation in Queue Length at a Traffic Signal
Referring to Figure 4.13, the time \( t_0 \) after the start of green that the queue is dissipated is given by

\[
t_0 = \frac{y r}{1 - y},
\]

(4.54)

where \( r \) = the effective red time,

\( y = q/s, \) and

the proportion of the cycle during which a queue exists \( P_q \) is given by

\[
P_q = \frac{(r + t_0)}{c},
\]

(4.55)

where \( c \) is the cycle time.

The proportion of vehicles stopped \( P_s \) will be given by

\[
P_s = \frac{q(r + t_0)}{q(r + g)} = \frac{t_0}{yc},
\]

(4.56)

where \( g \) is the green time.

The maximum number of vehicles in queue \( Q_m \) and the average number of vehicles in \( \bar{Q} \), are given by

\[
Q_m = qr
\]

(4.57)

and

\[
\bar{Q} = \left( (r + t_0) / c \right) (q r / 2)
\]

(4.58)

respectively.

The total vehicle-time of delay,

\[
D = \frac{q r^2}{2(1 - y)}
\]

(4.59)

may be divided by the number of vehicles to give the average individual delay.
FIGURE 4.13 Representation of queuing at a signalized intersection.

[AFTER 17]
\[ d = \frac{r^2}{2c(1 - \gamma)} \]  

(4.60)

Finally, the maximum individual delay is

\[ d_m = r. \]

It should be noted that for this case of uniform arrivals and departures, if the number of departures during a cycle \( Sc \) is less than the number of arrivals \( Qc \), the queue grows and the preceding equations do not hold.

Returning to the probabilistic aspects of the problem and referring to Figure 4.12, we assume no departures are possible during the red period \((0,R)\) and that there is no restriction on departures during the green period \((R,C)\). Following (73), we may derive an expression for the expected total wait per cycle, \( E[D] \) and, from this, the mean delay to an individual vehicle.

Letting \( A(t) \) be the number of vehicles which join the queue from the beginning of the red period until \( t \), i.e. \((0,t)\) and assuming simple Poisson arrivals with \( E[A(t)] = t \) and constant departure headways \( s \), put

\[ D = D_1 + D_2 \]  

(4.61)

where

\[ D_1 = \int_0^R [n(o) + A(t)] \, dt \]  

(4.62)

and

\[ D_2 = \int_R^C n(t) \, dt \]  

(4.63)
Considering the expected (average) values in Equation (4.62),

\[ E[D_1] = E[n(o)] + \frac{1}{2} \lambda R^2 \]  

(A.62)

the determination of the expected delay \( E[D_2] \) is significantly more complicated. Sufficient to say that it can be shown that

\[ E[D_2] = \frac{2E[n^2(o)] - n(c)}{2(1 - \lambda s)^2} + \frac{2E[n^2(R) - n^2(c)]}{2(1 - \lambda s)} \]  

(A.65)

Assuming the queue is in statistical equilibrium,

\[ \lambda c < (c - R) s^{-1} \]  

(4.66)

or

\[ \lambda s < 1 - R/c \]  

(4.67)

Also, for the equilibrium condition,

\[ E[n(o)] = E[n(c)] \]  

(4.68)

\[ E[n^2(o)] = E[n^2(c)] \]  

(4.69)

and

\[ n(R) = n(o) + A(R) \]  

(4.70)

Therefore

\[ E[n(R) - n(c)] = E[A(R)] = \lambda R \]  

(4.71)

and

\[ E[n^2(R) - n^2(c)] = 2E[A(R)] E[n(o)] + E[A^2(R)] \]

\[ = 2\lambda RE[n(o)] + \lambda^2 R^2 + AR \]  

(4.72)
Inserting Equation (4.71) and 4.73 into Equation (4.65) gives \( E[D_2] \), which, using Equation (4.64) and (4.61) leads to the desired expression for the total wait per cycle.

\[
E[D] = \frac{\lambda R}{2(1-\lambda s)} \left( R + \frac{\lambda}{2} E[n(o)] + s(1 + \frac{1}{1-\lambda s}) \right)
\]  

(4.73)

For the more general arrival process postulated by Darroch (36), Equation (4.73) becomes

\[
E[D] = \frac{\lambda R}{2(1-\lambda s)} \left( R + \frac{\lambda}{2} E[n(o)] + s(1 + \frac{1}{1-\lambda s}) \right)
\]  

(4.74)

where \( I \) is the index of dispersion to which earlier reference was made.

For binomial arrivals, Equation (4.74) specializes to the formula obtained by Beckmann et al. (5),

\[
E[D] = \frac{\lambda R}{2(1-\lambda s)} \left( R + \frac{\lambda}{2} E[n(o)] + 2s \right)
\]  

(4.75)

If right-turning vehicles are included, the assumption of constant departure headways is no longer valid. Assuming random departures with mean value \( m \), coefficient of variation \( v \), and simple Poisson arrivals, McNeil and Weiss (73) use the results of Daley and Jacobs (39) to obtain

\[
E[D] = \frac{\lambda R}{2(1-\lambda s)} \left( R + \frac{\lambda}{2} \left( 1 + \frac{(1-\lambda)(1-\lambda v^2)}{2} \right) E[n(o)] + s(1 + \frac{\lambda^2 m v^2}{2-\lambda s}) \right)
\]  

(4.76)

The mean delay for an individual vehicle may be obtained from any of Equations (4.73) through (4.76) by dividing by \( \lambda c \), the average number of vehicles to arrive during a cycle, thus,
\[ \bar{d} = \frac{E[D]}{\lambda c} \]  

(4.77)

The difficulty, however, lies in determining the expected number in the system at the beginning of the red period. During light traffic, it is unlikely that any vehicles will remain in the queue at the end of the green period and, hence, \( E[D(o)] \) may be neglected. A series expansion for \( E[D(o)] \) for simple Poisson arrivals was obtained by Kleinrock \(^{62} \) and according to Miller, \(^{74} \) as long as the normalized traffic intensity, \( \lambda s / (1 - h/c) \), is smaller than 0.5, \( E[D(o)] \) is not significantly different from zero. Table 4.4, based on \(^{73} \) gives values for the bounds of the mean delay per unit cycle time for \( R = 1/2c \). The symbols in the table have the following meanings:

- \( l \) = lower bound by neglecting overflow,
- \( E \) = exact value from numerical techniques,
- \( U_1 \) = upper bound according to McNeil, and
- \( U_2 \) = upper bound according to Miller.

According to Miller's field studies, his equation and Webster's are comparable when \( I \) is close to 1, and his equation gives much better values when \( I \) greatly exceeds one.

One additional formulation deserves mention since it is the basis for the screening charts which are part of the permit application. The expression was developed by Little \(^{66} \) assuming that:

(a) Arriving traffic is Poisson and in a single lane,
(b) Traffic is held up for a time \( T \), and then released,
(c) Vehicles starting up leave a constant time, \( h \), between them.
Table 4.4

VALUES OF BOUNDS FOR $E[R]/(\lambda c^2)$

<table>
<thead>
<tr>
<th>C/S</th>
<th>$\lambda$</th>
<th>0.165</th>
<th>0.33</th>
<th>0.495</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>L</td>
<td>0.1826</td>
<td>0.2331</td>
<td>0.3213</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>0.1828</td>
<td>0.2492</td>
<td>0.5093</td>
</tr>
<tr>
<td></td>
<td>$U_1$</td>
<td>0.1828</td>
<td>0.2505</td>
<td>0.5104</td>
</tr>
<tr>
<td></td>
<td>$U_2$</td>
<td>0.2274</td>
<td>0.3428</td>
<td>0.5271</td>
</tr>
<tr>
<td>40</td>
<td>L</td>
<td>0.1661</td>
<td>0.2099</td>
<td>0.2845</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>0.1662</td>
<td>0.2136</td>
<td>2.636</td>
</tr>
<tr>
<td></td>
<td>$U_1$</td>
<td>0.1662</td>
<td>0.2138</td>
<td>2.640</td>
</tr>
<tr>
<td></td>
<td>$U_2$</td>
<td>0.1885</td>
<td>0.2646</td>
<td>2.760</td>
</tr>
<tr>
<td>80</td>
<td>L</td>
<td>0.1579</td>
<td>0.1983</td>
<td>0.2660</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>0.1579</td>
<td>0.1986</td>
<td>1.416</td>
</tr>
<tr>
<td></td>
<td>$U_1$</td>
<td>0.1579</td>
<td>0.1986</td>
<td>1.418</td>
</tr>
<tr>
<td></td>
<td>$U_2$</td>
<td>0.1691</td>
<td>0.2256</td>
<td>1.504</td>
</tr>
<tr>
<td>80</td>
<td>L</td>
<td>0.1497</td>
<td>0.1866</td>
<td>0.2475</td>
</tr>
<tr>
<td></td>
<td>$U_2$</td>
<td>0.1497</td>
<td>0.1866</td>
<td>0.2475</td>
</tr>
</tbody>
</table>
(d) Normal road speed is lost instantaneously on joining the queue and regained instantaneously on starting up.

(a) There are no vehicles turning left or right.

The resulting equation for the average queue length is

$$\bar{n} = \frac{qR}{1 - qh}$$

(4.84)

where

- $q =$ flow rate (vps),
- $R =$ actual red time (sec), and
- $h =$ constant departure headway (sec)

For multiple lanes, Little considered that arriving vehicles will either (a) join the shortest queue at the traffic signal, or (b) form separate and independent streams of traffic. Equation (4.84) may be used for case (b) if $q$ is taken as the total flow divided by the number of lanes. According to Cleveland and Cappello (15), case (b) "appears to yield a better approximation of the average queue length for most applications when the arrival of traffic is Poisson distributed."

Probably the most widely used delay equation for fixed time signals is the Webster model (108). Developed as part of Webster's study of optimal signal settings, the equation is

$$\bar{d} = \frac{c(1 - x)^2}{2(1 - x)q} + \frac{x^2}{2q(1 - x)} = 0.65 \left( \frac{q}{x} \right)^{1/3} (2q + 1),$$

(4.78)
where \( \bar{d} \) = average delay per vehicle on the particular arm of the intersection,
\( c \) = cycle length in seconds,
\( \lambda \) = the ratio of the effective green time to the total cycle,
\( x \) = the degree of saturation = \( q/\lambda s \),
\( q \) = ave. arrival rate (in passenger car units per second), (PCU),
and \( s \) = saturation flow.

Much of the research on fixed-time signals is devoted to the study and analysis of this model and it has apparently held up well under the close scrutiny.

Weber developed the equation by simulating traffic flow with Poisson arrivals and fitting a curve to the results obtained. When compared to the "true" values in Table 4.4, Equation (4.78) is extremely accurate for high traffic intensities but underestimates somewhat for low intensities.

Recalling equation (4.60) and noting that \( \lambda \) in the Weber formulation is \( G/c \) and
\[ x = q/\lambda s, \]
the correspondence with the symbols used in the continuum model is
\[ r = c(1 - \lambda), \]
and
\[ y = \lambda x. \]
Therefore the first term in the Weber equation is simply that of continuous flow. Moreover, it may be shown (2) that the second term may be thought of as representing a queue of constant service \( 1/\lambda s \) between the signal and the arriving flow. Since the third term is a correction accounting for 5 to 15
percent of the total mean delay, Allsop\(^2\) suggested that the average delay may be taken as

\[
\overline{d} = 0.9 \left[ \frac{q(1-k)^2}{2(1-k)} + \frac{q^2}{2q(1-k)} \right] \tag{4.79}
\]

As part of his study, Webster noted that the average queue length at the beginning of the green period is given approximately by

\[
\bar{n} = \frac{(qr/2 + q\bar{d})}{q} \text{ or } qr \tag{4.80}
\]

whichever is larger. It may be seen that \(\bar{q}\) is the average queue length over the cycle and \(qr\) is the queue that develops during the red period. In fact, equation (4.80) gives very nearly the maximum queue during a cycle and the average maximum over time.

It should be noted in passing that as the degree of saturation approaches 1, i.e. \(x + 1\), the average delay predicted by Equation (4.79) becomes infinite, the queuing process is no longer stationary and the equation does not hold.

Equation (4.79) is quite sensitive to \(\lambda\), which in turn is sensitive to the effective green time, \(g\), which Webster (page 3) defines as follows:

The green and amber periods together \((k + a)\) may be replaced by an "effective" green \((g)\) and a "lost" time \(l\), such that the product of the effective green and the saturation flow is equal to the correct number of vehicles (say, \(b\)) discharged from the queue on the average in a saturated green period (i.e. a green period during which the queue never clears). Thus, \(k + a = g + 1\) and \(b = ga\), where \(s\) is the saturation flow.

Webster mentions that \(l\) varies from 1/2 second to 7 seconds in extreme cases. It "depends on gradients, type of traffic, etc." Lacking any more
specific information, he suggests 2 seconds per phase. This seems low when compared with Greenshields' (50) value of 3.7 seconds for start-up delay. It seems very low when compared to the lost time used by WoHl and Martin (110) who include an additional time for the last vehicle to clear the intersection. Therefore, it is suggested that the effective green time per phase $G_{eff}$ be taken as

$$G_{eff} = G + A - K_1 - K_2,$$

where $G$ = green time (seconds),

$A$ = amber time (seconds),

$K_1$ = start-up time = 3.7 seconds, and

$K_2$ = time for last vehicle to clear the intersection.

The time for the last vehicle to clear the intersection will be given by

$$K_2 = \frac{W + 17}{v},$$

where

$W$ = intersection width (feet), and

$v$ = speed of approaching traffic (fps).

Here the length of a typical passenger car has been taken as 17 ft.

The sensitivity of Equation (4.79) may be seen in Figure 4.14 where the equation is plotted for a cycle time of 70 seconds, a green plus amber time
Figure 4.14 Plot of the Webster Equation
of 34 seconds, an approach flow rate of 616 uph and a saturation flow of $1/2.1 = .475$ uph.

Miller\(^{76}\) proposed a model which was based on the premise that arrivals were independently distributed during successive phases. He made no other assumptions regarding arrivals. The delay, when $x > 1/2$, is given by

$$d = \frac{1 - \lambda}{2(1 - \lambda x)} \left[ c(1 - \lambda) + \frac{(2x - 1)I}{q(1 - x)} + \frac{1 + \lambda - 1}{s} \right] \quad (4.83)$$

where $s = $ saturation flow on the approach and

$I = $ variance/mean, for all PCU arriving during one cycle.

When $x < 1/2$, the middle term in the brackets is 0.

4.2.3 Traffic-Actuated Signals.

A modern traffic-actuated signal is exceedingly complex, having the capability, if properly adjusted, to respond efficiently to a wide range of traffic patterns. The simplest use of this type signal is at the intersection of two one-way streets. A paper by Darroch et al\(^{37}\) assumes Poisson arrivals and variable departure times and loss times. A general discussion of this case under the additional simplifying assumptions of fixed departure and lost times is given in\(^{73}\). Here the queue on approach $i$ is assumed to be discharged until a headway of $\beta_i$ is detected and there is assumed to be no upper limit. The system is given as

$$\beta_i = \frac{1}{\lambda} \log \left( \frac{1}{1 - \frac{1}{s \lambda}} \right) \quad i = 1, 2 \quad (4.85)$$

The question of optimization of the traffic-actuated signal is also considered by Courage and Papasanou\(^{29}\). In that study, the authors observed that the second term of the Webster equation, accounting for the random
nature of arrivals, could be modified based on maximum cycle lengths rather than optimal or average. It was concluded, based on simulation, that the difference in the predicted delay between the model proposed in (29) and the original Webster model is a maximum at 75 percent saturation but drops to zero at 88 percent. This comparison is shown in Figure 4.15. The reason for the zero difference at high volume-to-capacity ratios is, of course, due to the fact that at heavy loadings the traffic-actuated signal will behave as a fixed-time signal.

4.2.4 A Note on Passenger Car Equivalents

In general, unless otherwise specified, all of the equations and analyses presented in this report are in terms of passenger car equivalents (pce). In order to convert mixed traffic consisting of commercial vehicles and turning traffic in pce we will adopt the rather standard procedure given in Wohl and Martin (110) which assigns as follows:

- Non-left-turning passenger cars = 1 pce
- Buses and trucks = 1.5 pce
- Left-turning vehicles = 1.6 pce
- Right-turning vehicles = 1.4 pce

The factors given above are very approximate. In particular, the influence of buses or trucks and left-turning vehicles depends greatly on their position in line. For good geometries and little pedestrian interference the factor for right-turning vehicles is generally taken as 1.

4.3 Simulation
Figure 4.15 Relative and Absolute Benefits of Vehicle-Actuated Signal Control over Pretimed Signal Control to V/C Ratio

[After (29)]
4.3.1 General

It should be apparent, from the discussion of the previous section, that the solution to the problem under examination is very difficult unless a number of simplifying assumptions are introduced. A technique that has become increasingly popular in the last decade is that of simulation.

Through simulation, a real-world process is modeled and its operation studied over a period of time. With the availability of high-speed digital computers, it is possible to replace many hours of field observations with a short period of simulation. Since the input parameters may be varied at will, the technique readily allows for the conducting of experiments and represents a powerful design aid. Although the entire procedure may be deterministic, it finds its greatest application when certain of the inputs are probabilistic. The classic illustration of the technique is the study of the "drunkard's walk" which is used to introduce the general concept of "Monte Carlo" techniques. Briefly, the problem is as follows.

Suppose it is desired to study the perambulations of a drunk given that each of his steps has an equal probability of going in any direction. One possible (though inefficient) method would be by observing a large number of inebriates. The more efficient technique would be to simulate the behavior of a single drunk, record the results and repeat for as many drunkas as desired. The probabilistic aspects of the problem enter when the direction of each step is determined. While in the case cited, it is assumed that each direction is equally probable thus implying a rectangular probability density function (p.d.f.), the nature of the p.d.f. is arbitrary. With the p.d.f. given, "stochastic" (random) sampling is used to complete the technique. In this
simple case, a table of two-digit random numbers is entered for each step. If the first digit is odd the step is one unit in the positive X-direction, if even or zero, in the negative direction. The same interpretation only for the Y-direction is given to the second digit. Thus after the \( n \)th phase, the drunk has wandered a distance \( d_n \) given by

\[
d_n^2 = x_n^2 + y_n^2,
\]

where \((x_n, y_n)\) represents the position of the drunk at the end of the \( n \)th phase.

The procedure is repeated to follow the path of a second drunk and so on. After a number of simulations, a probability distribution of the distance walked may be constructed and analyzed for the mean, standard deviation and any other statistical parameters desired.

The technique can be developed to any level of complexity. In the case of traffic, the following simple example from (45) should be helpful.

**Simulation Example: Simple Four-Way Intersection**

It is now possible to progress to the simulation of a more complex situation; namely the four-way intersection shown in Figure 4.16, as described by Worrall. Several types of event are generated randomly. The intersection consists of two one-way streets, each with a single lane. The east-west street has priority over the north-south street. Low flows are assumed, and the arrival headways are assumed to follow the negative exponential distribution. The value \( T \) in the equation is taken as 3,600/\( V \), where \( V \) is the arriving volume in vehicles/hr. Each approach is assigned (as input data) probabilities that vehicles will turn or continue straight ahead. A gap acceptance distribution is provided, in tabular form, and the same distribution is used for all gaps. (See Table 9.2.)

Figure 4.16 illustrates the decisions by north-south (N-S) vehicles in accepting or rejecting gaps between east-west
Figure 4.16: Simulated Intersection

[After (47)]
(E-W) vehicles. The simulation model may be described with
the aid of the flow diagram of Figure 4.17.

The input data consist of volume levels on the two
approaches and a gap acceptance table. The input operation
is indicated by the first box at the top of Figure 4.17.
This includes the initial settings of all counters, etc.
The simulation loop starts with the second box, "Generate
Next N-S Arrival Gap."
This arrival gap is generated by
first generating a random fraction and then substituting
this fraction in Eq. 9.34. The actual arrival time of the
new arrival is obtained by adding the generated time head-
way to the arrival time of the previous N-S vehicle.

At this point it is appropriate to note whether the
undelayed arrival time of the new N-S vehicle exceeds the
maximum duration of the experiment. If the limit has been
reached, the program jumps to the calculation and printing
of output results; if not, the program continues.

Next the effective arrival time of the new N-S
vehicle is computed. This is the time that the vehicle
will arrive at the stopline. This calculation starts by
determining whether the previous N-S vehicle has crossed
the intersection; if it has not, the new vehicle joins
the queue; if it has, the new effective arrival time is
equal to the new actual arrival time.

Queues may be handled in several ways.

**TABLE 9.2 Gap Acceptance Distribution**
for Example

<table>
<thead>
<tr>
<th>Gap (sec)</th>
<th>Percent Acceptance</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>0</td>
</tr>
<tr>
<td>2-3</td>
<td>9</td>
</tr>
<tr>
<td>3-4</td>
<td>18</td>
</tr>
<tr>
<td>4-5</td>
<td>22</td>
</tr>
<tr>
<td>5-6</td>
<td>40</td>
</tr>
<tr>
<td>6-7</td>
<td>59</td>
</tr>
<tr>
<td>7-8</td>
<td>61</td>
</tr>
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<td>9-10</td>
<td>90</td>
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<td>10-11</td>
<td>92</td>
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<td>11-12</td>
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<td>12-13</td>
<td>94</td>
</tr>
<tr>
<td>13-14</td>
<td>95</td>
</tr>
<tr>
<td>14-15</td>
<td>96</td>
</tr>
<tr>
<td>15-16</td>
<td>97</td>
</tr>
<tr>
<td>16-17</td>
<td>98</td>
</tr>
<tr>
<td>17-18</td>
<td>99</td>
</tr>
<tr>
<td>&gt; 18</td>
<td>100</td>
</tr>
</tbody>
</table>

* Shifted exponential. (See 45)
Figure 4.17: Flow Chart for Intersection Simulation

[After (47)]
Figure 4.17: (continued)
One effective way is to establish within the computer an array three columns wide and of sufficient length to accommodate the longest queue expected. In addition, a queue counter indicates the number of vehicles currently in the queue. As each new vehicle joins the queue, the queue counter is first increased, then the row of the array corresponding to the count is selected. Into the three columns of this row are placed the actual arrival time, the effective arrival time (if it can currently be computed), and the departure time (if it can currently be computed). As the first vehicle in the queue moves out of the queue and across the intersection, the first row in the array is removed, each of the remaining entries is moved up one row, the place vacated by the last vehicle is set to zero and the queue counter is decreased. At the same time any further computations of departure and actual arrival times are performed.

The gap in the E-W stream acceptable to the first-in-queue N-S vehicle is determined. A random fraction is generated; this fraction is then compared with the percentage values in Table 9.2 (expressed as fractions). The gap corresponding to the random fraction is then designated as the minimum gap that will be accepted by the N-S vehicle.

Now begins an extensive series of tests concerning gaps in the E-W stream. First, test whether the effective arrival time of the (first-in-queue) N-S vehicle is later (greater) than the arrival time of the last E-W vehicle generated. If it is, it is necessary to generate a new E-W vehicle arrival gap and arrival time. The method used is similar to that used for N-S vehicles. If the new E-W arrival time exceeds the time limit for the experiment, the run is terminated (after calculating and printing the output results); otherwise, the E-W traffic count is increased, and the available gap in the E-W traffic is examined for acceptance by the first-in-queue N-S car. The available gap is the arrival time of the last E-W car minus the effective arrival time of the N-S car. After this gap is computed it is compared with the previously computed minimum acceptable gap.

If the gap is not acceptable, the effective arrival time of the first-in-queue N-S vehicle is reset to the arrival time of the last E-W vehicle, and a new E-W vehicle is generated. If the arrival time of this new E-W vehicle does not exceed the time duration of the experiment, the E-W traffic count is increased by one, and the acceptability of the gap in front of this new E-W car is tested. If the available gap (discussed in the previous paragraph) is found acceptable, it is accepted, and the departure time of the N-S vehicle entering
the intersection is computed as its effective arrival time plus the appropriate starting delay. Its delay in queue is computed by subtracting its arrival time from its departure time. This delay is added to the cumulative record of delay. After correcting the queue for the departure of one vehicle, the N-S count is increased and the simulation loop starts again by the generation of a new N-S vehicle.

Two important points are illustrated by this example. First, in addition to random generation of arrivals on two approaches there is random generation of gap acceptance. Second, the "clock" is not advanced by a uniform periodic interval; instead the examination moves from one important time to another important time. These differences in the methods of "review" or "scanning" are known, respectively, as "periodic scan" and "event scan."

Note that it is up to the analyst to decide what use is to be made of the data and the format for its tabulation. Thus, in the example given, we might record the average length of queue during the time a queue existed or during the entire period of interest. To illustrate further, not only could we record these obviously important parameters, but we could also record the time it took to move from the fourth to the first place in line and so on. Another very important aspect of the technique is its ability to handle systems of intersections. This is done by having the arrivals at one intersection dependent upon the departures for an upstream intersection and the traffic behavior in between.

The great flexibility inherent in simulation would be of obvious benefit in the present CO analysis for several other reasons. Perhaps the most important of these is that since the choice of the performance measure is at the discretion of the analyst, he can make that choice based on the most critical and meaningful data required in the emissions analysis. Thus, while with the current CO model, the input required is average queue length and
percent queue time, more refined analyses would probably take into account acceleration and deceleration and these would be readily obtained.

Finally, it should be noted that simulation is not only used to study a specific intersection or network, but rather it is used as a substitute for field observation from which phenomena can be more clearly understood and equations developed and verified. Indeed, it was in this manner that Webster used simulation in developing his model and Courage and Papapanou checked it for traffic-actuated signals.

4.3.2 Specific Models

As mentioned in the literature review, there have been several simulation models submitted in connection with the Connecticut indirect source program. Most of these seem adequate for present purposes. A detailed summary of the models and critical parameters is given in this section. First, however, a number of models of either historic interest or which appear to represent the state of the art are presented.

In broad terms, traffic simulation models may be classified as single road, single intersection and network. Several of the models which have been proposed in the indirect source program are of the intersection type. For the present, our attention centers on the network models. These can represent surface streets only or they may include freeway networks. Moreover they may be "macroscopic", in which individual vehicles are not identified; "platoon" models in which a platoon or group of vehicles is identified or "microscopic" in which the individual vehicle is identified. In the
microscopic type, a record of the position, speed and acceleration at any time for every vehicle is kept in memory. Behavior on streets in the microscopic models is approximated by car-following theory. It is also possible to handle lane-changing, buses and trucks. A relatively complete modeling of intersection behavior should include allowance for pedestrian interference, turning radii, collision avoidance and, of course, traffic signal operations.

What follows is a summary of the key features of a number of models reviewed by Gibson and Ross (48). In all cases the following assumptions apply unless otherwise stated:

The models are fully microscopic, that is, the model has a specific location, speed, and acceleration for each individual vehicle.

All variables in the system (vehicle locations, speeds, accelerations, and signal indications) are updated once each time step.

The simulation starts collecting data only after some initialization period.

Vehicle routes are determined from origin-destination tables (O-D) or the probability of going through or turning left/right at the end of each link (turning movements).

a. The TRANS model, although it has seen wide usage and has been somewhat validated, is now considered outmoded. The model updates at a period which cannot be less than 2 seconds, it does not account for unsignalized minor streets and all signalization is assumed to be fixed-time.

b. There are a number of other models which, according to (48) are now considered obsolete. These include:

1. Trautman and Davis
2. Stark/NBS
3. PAK-POY
4. Vehicle Traffic Simulator (VTS)  
5. Horthy  
6. VNET  
7. Sakai and Nagao  
8. VETERS  

C. A model by the Aerospace Corporation called VPT (Vehicle Performance in Traffic) links two other models, FREeway and VPRT. It allows for inclusion of a wide variety of probabilistic variables, including Poisson distribution of automobiles, trucks and buses, driver characteristics including frustration, and accidents. It is less flexible in its consideration of the details of signal timing than are some of the other models.  

D. The SCOT (Simulation of Corridor Traffic) model is UTCS-1 (discussed below) with an additional model of freeway traffic included. On the freeway portion, the model is not microscopic since vehicles are grouped into platoons.  

E. The SIGNET model is completely microscopic, but does not provide for lane changing or stop and yield sign control. It provides extreme flexibility in representing geometrics and traffic controllers and, since it is written in FORTRAN IV, it may be easily modified to provide statistics in addition to total vehicle miles, total delay, average delay, delay standard deviation and average speed. The model has not been validated other than that the output appears to be reasonable and consistent with the travel time studies.  

F. Although the name might imply otherwise, Micro-Assignment
is not a microscopic model in terms of simulation models since it ignores details such as offsets. It was originally designed as a traffic assignment method to be used in the transportation planning process. The feature of Micro-Assignment that sets it apart from other models is its treatment of turning vehicles. A node is placed in the center of each block, and turning vehicles are placed on different links than through vehicles. No validation of the model has been reported.

g. Although TRANSYT is a signal optimization program, it has a traffic simulation component. This simulation component is completely deterministic.

Traffic is assumed to enter the network at a uniform rate, and flow remains uniform until the first red signal. As a queue of vehicles is discharged from the signal, the vehicles form a platoon, which changes in shape as it proceeds down the next link. The shape of the platoon continues to change until the next red signal, where, of course, the platoon again forms a queue. By allowing the platoon to change shape as it moves through the network, TRANSYT's treatment of an intersection is dependent on the operation of all upstream intersections.

The output of TRANSYT is less detailed than most other models. However, it does yield good signal timings.

h. SIGOP II is another signal optimization model which, like TRANSYT, involves a simulation model, and treats vehicles in terms of platoons. It is more sophisticated than TRANSYT in
that it can deal with multiple-phase signals. Again, the output
is not as detailed as the output of most other models. SIGOP II
is currently being field-tested by FHWA.

1. CORSIG is a corridor optimization program, which is useful only
for expressways paralleled by two-way arterials. Basically, it
uses TRANSYT for analysis of the arterial, and FREQ5C for analysis
of the freeway. No validation of the model has been reported.

j. NETSIM is one of the most popular of the currently available
simulation models. Formerly called UTCS-l, it is completely
microscopic and, although it does not handle factors such as
parking restrictions very well, it is otherwise very realistic.
It requires very detailed inputs, but yields very detailed outputs,
both for individual links and for the network as a whole. The
model has been validated by field testing in Washington, D. C.
The program is coded in FORTRAN and requires the following
inputs:

1. For each network link, number of moving lanes, length,
capacity of left-turn pocket, desired free-flow speed,
mean queue discharge rate, turning movements at downstream
node, identification of receiving links, pedestrian
volume, and lane channelization

2. At each intersection complete specification of signal (or
sign) control, including sequence and duration of each
phase and identification of signal facing each approach

3. Traffic demand specified as flow rate (vph), percentage of
trucks emitted onto the network along input (entry) links
and from internal source nodes, and rate of extraction of
vehicles at sink nodes
4. Duration of simulation subintervals and specification of output options
5. As an option, specification of bus systems (routes, stations, mean headways, and mean dwell times) and frequency and duration of events, i.e., vehicles or conditions that block moving lanes of traffic

Output includes the following data which, as well as being coded for each network link, are also aggregated for the entire network.

1. Link identification, by origin and destination node
2. Estimate of total vehicle-miles of travel
3. Count of total vehicles discharged
4. Total vehicles-moving-time (at free-flow speed) in vehicle-minutes
5. Total delay time computed at the difference between total travel time and ideal travel time based on target speed for link, in vehicle-minutes
6. Ratio of moving time at desired speed to total travel time
7. Total travel time in vehicle-minutes
8. Average travel time per vehicle in seconds
9. Average travel time per vehicle-mile in seconds per mile
10. Average delay time per vehicle in seconds
11. Average delay time per vehicle-mile in seconds per mile
12. Average traffic speed in mph
13. Average occupancy (population) in number of vehicles
14. Percentage of vehicles stopping at least once, expressed as a decimal
15. Average saturation percentage, expressed as the average over time of the portion of the link that is occupied by vehicles divided by its total storage capacity

16. Total number of cycle failures, defined as the number of times queue fails to clear from the discharge end of the link during a green period

17. Ratio of number of vehicles stopping more than once in a link to the total number of vehicles processed

k. One model submitted in connection with the indirect source program is INSECT 69, developed by the Stanford Research Institute as a submodel to the Dynamic Highway Transportation Model. While INSECT 69 is deterministic, it is also quite comprehensive in its allowance for intersection geometry and signal strategy. Downstream input rates reflect the operation of upstream signals and the diffusion of a platoon of vehicles moving between the signals. Departure headways for each possible movement at an approach (i.e. straight, right or left) are derived from capacities given in the Highway Capacity Manual (55) and modified according to downstream congestion and interactions with other vehicles. Left turns are assigned a headway of

$$H = \frac{1}{q} (e^{Tq} - 1 - Tq) + 1.42 ,$$  \hfill (4.85)

where $q$ is the flow rate of the opposing traffic and $T$ is the initial gap (taken as 5 seconds). Equation (4.85) may be seen to be Equation (4.43) with 1.42 seconds added for
the time for the driver of a following vehicle to react to the movement of a vehicle in front.

Vehicles are assumed to accelerate up to a target velocity downstream, or for right-turning vehicles, up to a maximum turning speed determined by the radius of the turn. Finally, even though the model may be applied to non-signalized intersections, it is sufficiently sophisticated to handle fully-activated signals and coordinated signal systems. Controller settings that may be introduced are initial interval, maximum green, unit extension and clearance interval.

1. A probabilistic simulation model submitted by Wilbur Smith Associates is based on the work of May and Pratt (71) and May and Gyamfi. It is designed to handle an isolated intersection controlled by a fixed-time signal. It assumes a composite exponential arrival distribution although it may be modified to handle any input distribution. Turning movements are also handled stochastically, with a directional distribution function required as input. Other input includes the signal cycle length and green time for the approach under consideration.

One hour of traffic is simulated, with vehicles being processed one at a time through the intersection. The delay of each vehicle is dependent upon the signal phase and upon the preceding vehicle. Minimum output headway is required as input and apparently no recognition is given to intersection geometry other than that implicit in output headway. It appears as though no allowance is made for time lost in
getting the queue in motion.

A set of summary statistics is produced as follows:

1. average number of arriving vehicles per hour;
2. percent vehicles that did not wait;
3. average waiting time per vehicle;
4. average queue length as observed each second, and
5. average queue length observed at the end of each cycle.

A model submitted by the Comsis Corporation is called INTSIM.

Although it is not entirely clear from the presentation, the technique seems to be a hybrid and not a true simulation.

For non-signalized intersections, it consists of determining an average service time and then applying Equation (4.32) to determine queue length. A negative exponential distribution of headways for conflicting traffic is assumed. Although a program for handling a signalized intersection has apparently been used, available documentation did not permit its review for this report.

4.4 Significant Parameters

Based on the analyses presented in this report, the following are recommended as reasonable distributions and parameter values to be used in connection with the indirect source program.

Arrival distribution. This does not appear to be a particularly crucial determinant of queue length. As a result, the assumption of a simple negative exponential distribution is probably adequate. Composite
exponential or Erlang might also be used. In cases where an upstream control is likely to introduce non-randomness, either another appropriate distribution should be used or a network simulation technique should be employed.

Service time distribution. Again a negative exponential distribution would be adequate for many studies. Under heavier flows the Erlangian would probably be more suitable. Figure 4.3 may be used as a guide to the proper Erlang parameter. In certain simulation models for signalized intersections, a constant service time distribution is appropriately used.

Critical Gap. As discussed in Section 4.1.5, specification of a critical gap or lag is essentially impossible since there is no one value, but a distribution of values. On the other hand, gap acceptance is only one element in the total service time. For this reason, in applying classical analytic techniques it is probably sufficient to assume conservative values of 6.5 and 5 seconds for the stop and yield-sign cases respectively. With more detailed simulation models it should be possible to introduce a distribution function for acceptance.

Lost Time. Unless more specific data are available for the site under consideration, the queue discharge characteristics presented by Greenshield (30) should be used. These are 3.7 seconds lost time and 2.1 second headways.

Traffic Intensity. Since the length of queue builds rapidly as \( \rho \) approaches one, traffic intensities greater than .85 should be examined with caution.
5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary of Models

Table 5.1 lists the various queuing and air pollution models which have been studied. Given in the table are the equation (or, in the case of simulation models, the name), the restrictions on its use and other comments. For a more complete description, the reader is referred to the body of this report.

5.2 A Note on the Average Number and Percent Queue Time

Several questions of interpretation of "average number of vehicles in the system" are discussed in this subsection. This was touched upon briefly in Subsection 4.2.2. Figure 4.18 shows the variation in the number of vehicles in an hypothetical system over a time period $t$. Thus, between time 0 and 1 there are 8 vehicles in the system; between 1 and 2 there are 6, etc.

Let

$t_Q = \text{time a queue exists (i.e., there are vehicles in the system)},$

$t = \text{total time during the period of interest},$

$n = \text{average number of vehicles in the system over } t,$

$n_Q = \text{average number of vehicles in the system when a queue exists},$

$u = \text{an increment of } t$

By definition,

$$n = \frac{\sum n_k}{\text{number of arrivals}} = \frac{\sum n_k}{t/u} \quad (5.1)$$
<table>
<thead>
<tr>
<th>Model</th>
<th>Equation Number</th>
<th>Applicability</th>
<th>Restrictions</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n = \frac{\rho}{1 - \rho}$</td>
<td>4.32</td>
<td>1. single lane, unsignalized approach, can be applied to multiple lane approach if arrivals are assumed to divide equally among all lanes</td>
<td>1. Poisson arrivals 2. exponential service times 3. $\rho &lt; 1$</td>
<td>1. can be derived from basic theorems of probability</td>
</tr>
<tr>
<td>$n = \rho + \frac{\rho^2}{2} \left( 1 + \frac{2}{k} \right)$</td>
<td>4.33</td>
<td>1. single lane, unsignalized approach</td>
<td>1. Poisson arrivals general distribution of service times 3. $\rho &lt; 1$</td>
<td></td>
</tr>
<tr>
<td>$n = \rho + \frac{\rho^2}{2} \left( 1 + \frac{1}{k} \right)$</td>
<td>4.36</td>
<td>1. single lane, unsignalized approach</td>
<td>1. Poisson arrivals 2. Erlang service times 3. $\rho &lt; 1$</td>
<td></td>
</tr>
<tr>
<td>$s = \int_0^t \frac{\lambda k}{\lambda k} e^{-\lambda t} , dt$</td>
<td>4.42</td>
<td>1. single lane, yield or stop sign controlled</td>
<td>1. Poisson arrivals on minor leg 2. Erlang headways in major flow 3. only one vehicle processed at time</td>
<td></td>
</tr>
<tr>
<td>$s = \frac{1}{k} \left[ \left( 1 - q_B \right) e^{-\lambda t} - \left( 1 - q_A \right) e^{-\lambda t} \right]$</td>
<td>4.46</td>
<td>1. single lane merge (yield sign controlled)</td>
<td>1. Poisson arrivals on minor leg 2. only one vehicle enters per gap</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 5.1 Summary of Models**
<table>
<thead>
<tr>
<th>Model</th>
<th>Equation Number</th>
<th>Applicability</th>
<th>Restrictions</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>( -\frac{q_{1}}{s} = \frac{q_{2}}{q} \left( 1-e^{-\frac{q_{1}t_{1}}{1-e^{-\frac{q_{1}t_{1}}{2}}} - \frac{t_{1}}{t_{2}} - \frac{t_{1}}{t_{2}} \right) )</td>
<td>4.47</td>
<td>1. single lane merge (yield sign controlled)</td>
<td>1. ( t_{1} \leq t_{2} )</td>
<td></td>
</tr>
<tr>
<td>( \bar{n} = \left( \frac{r + c}{c} \right) \frac{1}{2} \ \frac{\bar{q}}{\bar{q}} )</td>
<td>4.58</td>
<td>1. single lane, signalized approach</td>
<td>1. ( qC &lt; SC ) 2. uniform arrivals and departures</td>
<td></td>
</tr>
<tr>
<td>( n = \frac{q_{k}}{1 - q_{h}} )</td>
<td>4.84</td>
<td>1. single lane approach</td>
<td>1. no turning vehicles 2. Poisson</td>
<td></td>
</tr>
<tr>
<td>( \bar{n} = \frac{1}{(k-1)!(k\nu - \lambda)^{2}} p(0) + \frac{\lambda}{\mu} )</td>
<td>(110), page 368</td>
<td>1. multiple lane approach such as toll booth</td>
<td>1. all arriving vehicles placed in single queue 1. situation rarely found in practice</td>
<td></td>
</tr>
<tr>
<td>( \text{where} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( p(0) = \sum_{n=0}^{\lambda-1} \frac{\lambda}{n!} \frac{n!}{(\nu)^{n}} + \frac{1}{\lambda} \sum_{n=0}^{\lambda-1} \frac{\lambda}{(\nu)^{n}} \frac{1}{k!} \frac{k\nu}{k\nu - \lambda} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \bar{J} = \frac{c(1-\lambda)^{2}}{2(k-\lambda \lambda)} + \frac{x^{2}}{2q(1-x)} )</td>
<td>4.78</td>
<td>1. fixed-time signal</td>
<td>1. Poisson arrivals 1. Webster equation</td>
<td></td>
</tr>
<tr>
<td>( -0.65(a/q^{2})^{1/3} \lambda^{(2+5\lambda)} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 5.1 (Continued)**
<table>
<thead>
<tr>
<th>Model</th>
<th>Equation Number</th>
<th>Applicability</th>
<th>Restrictions</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \bar{a} = (q_{r}/2 + q_{d}) \text{ or } q_{r} )</td>
<td>4.80</td>
<td>1. fixed-time signal</td>
<td>1. average as beginning of green period</td>
<td></td>
</tr>
<tr>
<td>( \bar{d} = 0.9 \frac{n_{(1-i)^{2}}}{2(1-\lambda x)} + \frac{x^2}{2q_{d}(1-x)} )</td>
<td>4.79</td>
<td>1. fixed-time signal</td>
<td>1. reasonable approximation of Shearer equation</td>
<td></td>
</tr>
<tr>
<td>( 30=q_{p} + 0.1726 Q_{f} (1-t_{p}) \text{VEF} )</td>
<td>3.6</td>
<td>1. any roadway</td>
<td>1. none</td>
<td></td>
</tr>
<tr>
<td>( \psi = 1.26 \sqrt{\frac{N}{L \text{m}}} \left[ \frac{Q_{t}}{S_{t}} \exp \left[ \frac{-s_{t}^2}{110,000} \right] \right] )</td>
<td>3.7</td>
<td>1. any intersection</td>
<td>1. receptor must be located 10 m from road edge</td>
<td></td>
</tr>
<tr>
<td>SETSIM</td>
<td></td>
<td>1. networks</td>
<td>1. none</td>
<td>1. detailed output; requires detailed input</td>
</tr>
</tbody>
</table>

*TABLE 5.1 (Continued)*
<table>
<thead>
<tr>
<th>Model</th>
<th>Equation Number</th>
<th>Applicability</th>
<th>Restrictions</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRANSYT</td>
<td>1.</td>
<td>networks</td>
<td>1. uniform arrival rate assumed at signal farness upstream</td>
<td>1. macroscopic 2. deterministic 3. yields good signal timings</td>
</tr>
<tr>
<td>SIGVP II</td>
<td>1.</td>
<td>networks</td>
<td>1. none</td>
<td>1. macroscopic 2. groups vehicles into platoons</td>
</tr>
<tr>
<td>INSECT 69</td>
<td>1.</td>
<td>non-signalized intersections through signaled networks</td>
<td>1. none</td>
<td>1. deterministic</td>
</tr>
<tr>
<td>Albur Smith Model</td>
<td></td>
<td>isolated intersection with fixed-time signalization</td>
<td>1. none</td>
<td>1. assumes composite exponential arrival distribution, but can be modified to deal with any input distribution</td>
</tr>
<tr>
<td>INTSIM</td>
<td>1.</td>
<td>signalized or non-signalized intersections</td>
<td>1. none</td>
<td>1. apparently hybrid; apparently not true simulation</td>
</tr>
</tbody>
</table>

**TABLE 5.1 (Continued)**
Figure 4.18 Variation in Queue Length with Time
and
\[ \bar{n}_Q = \sum_n \frac{n}{\bar{c}_Q} f_n, \]  

therefore
\[ \frac{\bar{n}}{\bar{n}_Q} = \frac{t_0}{t}. \]  

But, as discussed in Subsection 4.2.1, the fraction of time the system is busy is given by \( \rho \), therefore
\[ \frac{t_0}{t} = \rho. \]  

or
\[ \bar{n}_Q = \frac{\bar{n}}{\rho}. \]  

Recalling Equation (3.5) for the weighted average CO concentration,
\[ CO = CO_{C} \frac{\bar{c}_C}{C} + CO_{ff} \frac{\bar{c}_C}{C} \]  

we note that
\[ \frac{\bar{c}_C}{C} = \rho, \]  

and
\[ \frac{\bar{c}_C}{C} = 1 - \rho. \]  

If
(a) CO was a linear function of \( n \), say \( Q(n) \) and

(b) arrivals and service times were random over the entire period of interest, then Equation (3.5) would become
\[ CO = \bar{Q} \rho + C E_{ff} (1-\rho) \]  

and, by inserting for Equation (5.5)
or
\[ \omega = \frac{\bar{S}}{\bar{n}_2} \rho + CE_{ff}(1-\rho) \quad (5.9) \]

so that it would make no difference whether \( \bar{n} \) or \( \bar{n}_Q \) were used as long as the appropriate adjustment was made in Equation (3.5).

However, neither of the conditions (a) nor (b) is satisfied over the 8-hour period of interest. In order to demonstrate the effect of non-randomness, consider a minor street controlled by a stop sign and having an average rate of arrivals \( \lambda_1 \). The major street has an arrival rate of \( \lambda_2 \). Thus, considering the queue on the minor street the value of service time will be given by, e.g. Equation (4.42)
\[ \bar{s} = \frac{1}{\lambda_2} \left( e^{\lambda_2 \tau} - 1 - \lambda_2 \tau \right) \quad (4.42) \]
and thus
\[ \mu = \lambda_2 \left( e^{\lambda_2 \tau} - 1 - \lambda_2 \tau \right)^{-1} \quad (5.11) \]
so that, even if \( \lambda_1 \) and \( \lambda_2 \) change in the same pattern during the course of the 8-hours, i.e.
\[ \frac{\lambda_1}{\lambda_2} = \text{constant} \quad (5.12) \]
the ratio
\[ \rho = \frac{\lambda_1}{\mu} = \frac{\lambda_1}{\lambda_2} \left( e^{\lambda_2 \tau} - 1 - \lambda_2 \tau \right) \quad (5.13) \]
does not stay constant. Hence the average queue length will vary in a
rather complicated fashion.

The fact that $CO$ is not a linear function of $n$ was demonstrated in Section (3.4).

Therefore, we recommend the following procedure if $p > 0.80$.

a. Divide the 8-hour period of interest into several (about 4) periods of relatively constant arrival rate,

b. Within each of the periods defined in a, calculate the average number of vehicles in the system $\bar{n}$ by Equations such as (4.33),

c. Using $\bar{n}$, $R/C = p$, and $G/C = 1 - p$, calculate CO by Equation (3.5) for each period,

d. Weight the CO concentrations from c according to the length of each period and the resulting average CO will be the eight-hour average.

5.3 Recommendations

This study has shown that the present process of predicting queue properties for the purpose of predicting carbon monoxide concentrations for comparison with the standards set by the Clean Air Act is, in general, adequate. This adequacy arises from the relative accuracy in the queue modeling as compared to volume predictions and, more importantly, emissions modeling. While it was beyond the scope of the present study to exhaustively critique these other phases, a sufficient background was established to place the queuing analysis in a proper perspective.

The prediction of traffic volumes seems to be a reasonable process, consistent with accepted practice for the time frame under consideration.
Probably the most serious shortcoming lies in the identification of the peak one-hour volume during the year. The figure sought is a difficult one to predict and the methods presently being used are reasonable but not conservative. When it is realized that all of the other phases in the analysis are conservative, the present approach is appropriate. In any event, the lack of conservatism is particularly unimportant since the one-hour standard is rarely the one of concern.

The emission and dispersion analysis is based on a technique developed in response to the need for answers to a complicated problem at a time when the state-of-the-art left much to be desired. CommDEP recognized the profound importance of interrupted flow at an early date and, while the model developed reflects this importance by examining the idling mode, there are several suggestions that may be offered. First, the model neglects acceleration and deceleration. The resulting approach leads to unnatural and unnecessary restrictions on the queuing analysis. As will be discussed later, simulation has the ability to examine the phenomenon on the microscopic basis of the individual vehicle. In this way, emissions and dispersion are automatically keyed to whether the vehicle is in free-flow, accelerating, decelerating or idling. It is recognized, of course, that simulation is impractical for every intersection to be examined. For this reason, it is suggested that the current approach be retained as a screening device with simulation being used in cases where a potential violation is suggested.

The present idling car model, if it is to be retained, should be subjected to more extensive evaluation. A particular peculiarity of the model
is its prediction of increased concentration with increased wind speed. This seems to be contrary to predictions of other widely-used models. The finding that maximum CO concentration is independent of receptor location is also interesting and seem contrary to intuition. It is our understanding that "worst-case" meteorological conditions have been assumed. Without a more extensive statistical evaluation of the variability of these meteorological conditions it is impossible to be definitive, but it would appear highly improbable that the worst case meteorological conditions would coincide with the worst-case queuing for a period of eight hours.

As an aside, establishing a point at ten meters from the edge of the pavement as the receptor location seems artificial. In fact, there are instances where concentrations within the mixing-cell would be of more interest and many more instances when the nearest expected human habitat is at a much greater distance than ten meters, if indeed, it is in the vicinity of the intersection at all. Finally, since the idling-car model is based on measurements taken in an open parking-lot and there are no allowances for corrections, it totally ignores the "canyon effect" which is of great importance in an urban environment.

With regard to the main focus of this study, queuing analysis, the following recommendations are offered. First, the models listed in Table 5.1 may be used recognizing the associated caveats. As mentioned in Subsection 5.2, it seems reasonable to examine the eight-hour average queue based on the average queue existing over the entire period with the appropriate modification to Equation 3.5. In cases where the traffic intensity exceeds 0.80, it is suggested that the eight-hour period be broken into 4 periods during each of which the average queue and resulting pollution
are calculated.

For yield-sign, stop-sign and signalized intersections, it is recommended that Appendix A be used. For certain situations, if a reasonable estimate of service times is available, Equations (4.32) through (4.36) may be used (provided traffic intensity is less than 0.85). Figure A-1 is a worksheet to be used with Figure A-2 for the yield-sign or stop-sign conditions. Figure A-3 is the worksheet completed for three examples.

Figures A-7 and A-8 are used for the solution of the Webster equation. It should be noted that the Webster equation was derived for fixed-time signals and according to Courage and Papamou (29) the relative delay savings of an actuated signal over a fixed-time signal approaches 40% for a volume/capacity ratio of about 0.6. This conclusion, however, is based on the simulation conducted in (29) and until it is verified in J&K Project 78-3 we suggest that the more conservative Webster results be used. Additional reasons for this are that the delay savings disappear at high V/C ratios and that the savings assume optimal signal settings.

An upcoming Joint Highway Research Project (78-3) will be concerned with implementing the computer simulation program NETSIM. As a part of that project it is anticipated that more direct and realistic predictions of CO concentrations will be forthcoming. The program will have the capability of handling multiple intersections. In addition, it will allow for testing and refinement of the analytic queuing models discussed herein. It is hoped that this will result in a streamlining of the process so that appropriate consideration may be given to problem areas while minimizing effort expanded on non-problem situations.
References


27. PACER. (1973).


93. Smith, Wilbur. Queuing Model.


112. Yagm, S. CORS - A Model for Predicting Flows and Queues in a Road Corridor. Transportation Research Record 4533, (1975), pp. 77-87.

APPENDIX A

WORKSHEETS
This appendix is intended to provide a simple means of applying the techniques recommended in JHR 77-2, Queuing Models for Air Pollution. The user is referred to that report for a detailed development of the equations and statement of assumptions.

The following cases are covered by the charts in this appendix:

1. **MERGE, YIELD OR STOP-SIGN.** Figure A-1 is a worksheet to be used with Figure A-2 for the merge, yield or stop-sign conditions. Figure A-3 is the worksheet completed for three examples. Note that the third example is for a stop-sign condition and the average delay is that read from A-2 with 3 seconds added to account for the compulsory stop. The result of the calculation on line 10 is $\bar{n}$, the average number in the system.

2. **FIXED TIME TRAFFIC SIGNAL.** Worksheets "A" and "B" (Figures A-5 and A-7) along with Figure A-8 and Tables A-1 through A-5 are to be used for fixed-time traffic signals. An example of a two-phase signal from (110) is shown in Figures A-4 and A-6. Specific instructions are given below.

**Worksheet "A"**

a. Enter intersection dimensions (lane widths and setback)

b. Enter approach volumes by lane. The parenthetical values correspond to trucks and commercial vehicles. Values without parentheses correspond to passenger cars. Each lane is identified by a circled number. Thus, in the example, the northbound approach consists of one 14 ft. wide lane; the total approach volume consists of 495 vehicles of which 25 are trucks; 53 vehicles turn left and 22 turn right.
Worksheet "B"

The appropriate input data should be entered as follows:

Row 1. Autos. The total number of autos in each lane (vph)
Row 2. Trucks. The total number of trucks in each lane (vph)
Row 3. Left turns. This is the total number of vehicles turning left from the lane under consideration plus any vehicles turning left in front of the lane under consideration. (vph)
Row 4. Right turns. The total number of vehicles turning right from the lane under consideration. (vph)
Row 5. Autos straight. The total number of autos going straight ahead (vph)
Row 9. Passenger car equivalents. The total of rows 5 through 8.
Row 11. Critical lane volume. This is the larger value for each phase in row 8. (vph)
Row 13. Crossing width. The distance from the stopline to the far curb line. (ft.)
Row 14. Approach speed (mph)
Row 15. Amber. The amber period from Table A-1 based on approach speed from row 14 and crossing width from row 13.
Row 16. The maximum amber determined for each phase
Row 17. Any all-red interval
Row 18. Lost time, \(k_2\). This is taken from Table A-2 based on approach speed and intersection width.
Row 23. Optimum cycle length, \(C_0\). This is taken from Figure A-8 using \(K\) from row 20 and \(Y\) from row 22.
Row 30. $a_{ij}$. This is interpolated from Table A-3 using $x_{ij}$ from row 29 and $\lambda_2$ from row 28.

Row 31. $b_{ij}$. This is interpolated from Table A-4 using $x_{ij}$ from row 29.

Row 33. $T_{ij}$. This is interpolated from Table A-5 using $x_{ij}$ from row 29, $\lambda_2$ from row 28 and $M_{ij}$ from row 32.

Row 44. Average queue. This is the average queue (number in the system) at the beginning of the green period.

Thus, it is the average maximum queue for each cycle.
To Be Used with FIGURE A-2.

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Intersection</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(2) Leg</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(3) Time Period 1 hr.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 hr.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(4) $V = \text{Approach Vol. (vph)}$</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>(5) $\lambda = (4) \div 3600 \ (\text{vph})$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(6) $q = \text{Vol. on Expressway (vph)}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(or main street)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(7) $\tau = \text{Critical gap}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(8) $k$ Figure A-2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(9) $\bar{n}$ Figure A-2 (plus 3 for stop-sign)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(10) $\rho = (5) \cdot (9)$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(11) $\bar{n} = \rho + \rho^2 \cdot (1 + 3/k) \div (1 - \beta)$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Note: If $\rho > .85$, a more complete analysis should be performed since this may be approaching transient case.

Figure A - 1 MERGE OR STOP SIGN CONDITION
Figure A-2. Merging or stop-sign delay
Modified after (40)
To Be Used with FIGURE A-2.

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Intersection</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>(2) Leg</td>
<td>(merge)</td>
<td>(merge)</td>
<td>(stop-sign)</td>
</tr>
<tr>
<td>(3) Time Period</td>
<td>1 hr.</td>
<td>2 hr.</td>
<td>3 hr.</td>
</tr>
<tr>
<td>(4) ( V ) = Approach Vol. (vph)</td>
<td>400</td>
<td>300</td>
<td>350</td>
</tr>
<tr>
<td>(5) ( \lambda ) = (4) ( \div ) 3600 (vps)</td>
<td>0.12</td>
<td>0.083</td>
<td>0.097</td>
</tr>
<tr>
<td>(6) ( q ) = Vol. on Expressway (vph)</td>
<td>900</td>
<td>1100</td>
<td>900</td>
</tr>
<tr>
<td>(7) ( \tau ) = Critical gap</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>(8) k Figure A-2</td>
<td>2</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>(9) ( \bar{q} ) Figure A-2 (plus 3 row stop-sign)</td>
<td>6</td>
<td>11</td>
<td>6+3+9</td>
</tr>
<tr>
<td>(10) ( p ) = (3) ( \cdot ) (9)</td>
<td>0.72</td>
<td>0.91</td>
<td>0.87</td>
</tr>
<tr>
<td>(11) ( \bar{n} ) = ( p + \frac{p^2}{2} \frac{1 + 1/k}{1 - p} )</td>
<td>2</td>
<td>7</td>
<td>5</td>
</tr>
</tbody>
</table>

*Note: If \( p > 0.85 \), a more complete analysis should be performed since this may be approaching transient case.*

Figure A-3 MERGE OR STOP SIGN CONDITION (EXAMPLE)
Figure A-4. Worksheet "A" for Webster Equation (Example)
Figure A-5. Worksheet "A" for Webster Equation
<table>
<thead>
<tr>
<th>Phase, i</th>
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<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approach m</td>
<td>Southbound</td>
<td>Northbound</td>
<td>Westbound</td>
<td>Eastbound</td>
</tr>
<tr>
<td>1. Lane 1</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>1. Autos</td>
<td>340</td>
<td>470</td>
<td>350</td>
<td>553</td>
</tr>
<tr>
<td>2. Trucks</td>
<td>20</td>
<td>25</td>
<td>21</td>
<td>41</td>
</tr>
<tr>
<td>3. Total left lurns</td>
<td>92</td>
<td>98</td>
<td>88</td>
<td>70</td>
</tr>
<tr>
<td>4. Total right turns</td>
<td>15</td>
<td>23</td>
<td>13</td>
<td>25</td>
</tr>
<tr>
<td>5. Autos straight</td>
<td>300</td>
<td>400</td>
<td>390</td>
<td>543</td>
</tr>
<tr>
<td>6. (2) + 1.5</td>
<td>30</td>
<td>38</td>
<td>32</td>
<td>62</td>
</tr>
<tr>
<td>7. (3) + 1.5</td>
<td>147</td>
<td>147</td>
<td>78</td>
<td>80</td>
</tr>
<tr>
<td>8. (4) + 3.5</td>
<td>21</td>
<td>31</td>
<td>73</td>
<td>10</td>
</tr>
<tr>
<td>9. (5 + (6) + (7) + (8)</td>
<td>498</td>
<td>616</td>
<td>655</td>
<td>665</td>
</tr>
<tr>
<td>10. (9) + 3600</td>
<td>138</td>
<td>177</td>
<td>118</td>
<td>190</td>
</tr>
<tr>
<td>11. Critical lane vol. (vph)</td>
<td>676</td>
<td>683</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12. L (11)</td>
<td>1291</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13. Crossing width (ft.)</td>
<td>60</td>
<td>60</td>
<td></td>
<td>40</td>
</tr>
<tr>
<td>15. A, b = Table A-1</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>16. A, b = Phase Max</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>17. All v =</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18. k2 = 0.12</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19. (17) + (18) + (37)</td>
<td>6.3</td>
<td>6.3</td>
<td>5.3</td>
<td></td>
</tr>
<tr>
<td>20. k = Y (19)</td>
<td>11.6</td>
<td>11.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>21. (21)</td>
<td>0.357</td>
<td>0.357</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22. Y = X (21)</td>
<td>0.754</td>
<td>0.754</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23. Ca (Table A-8)</td>
<td>22</td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24. (23)</td>
<td>22</td>
<td>22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25. (25) + (32)</td>
<td>0.47</td>
<td>0.47</td>
<td></td>
<td></td>
</tr>
<tr>
<td>26. (30) + (24)</td>
<td>37.8</td>
<td>37.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27. (26) + (29)</td>
<td>44.1</td>
<td>44.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28. k =</td>
<td>0.46</td>
<td>0.46</td>
<td></td>
<td></td>
</tr>
<tr>
<td>29. x = 2.1 + (10) + (28)</td>
<td>0.77</td>
<td>0.77</td>
<td>0.77</td>
<td>0.77</td>
</tr>
<tr>
<td>30. Aij (Table A-3)</td>
<td>0.246</td>
<td>0.272</td>
<td>0.266</td>
<td>0.263</td>
</tr>
<tr>
<td>31. Bi (Table A-4)</td>
<td>0.849</td>
<td>3.23</td>
<td>1.29</td>
<td>2.91</td>
</tr>
<tr>
<td>32. M = (31) + (23)</td>
<td>12.7</td>
<td>15.7</td>
<td>15.5</td>
<td>15.5</td>
</tr>
<tr>
<td>33. Fe (Table A-5)</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>34. (27) + (30)</td>
<td>22.6</td>
<td>22.6</td>
<td>22.6</td>
<td>22.6</td>
</tr>
<tr>
<td>35. (31) + (10)</td>
<td>6.3</td>
<td>6.3</td>
<td>6.3</td>
<td>6.3</td>
</tr>
<tr>
<td>36. (34) + (35)</td>
<td>28.9</td>
<td>28.9</td>
<td>28.9</td>
<td>28.9</td>
</tr>
<tr>
<td>37. (100 - 35) + 100</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
</tr>
<tr>
<td>38. (36) + (37)</td>
<td>26.9</td>
<td>26.9</td>
<td>26.9</td>
<td>26.9</td>
</tr>
<tr>
<td>39. (25) + (27) + (38)</td>
<td>63</td>
<td>63</td>
<td>63</td>
<td>63</td>
</tr>
<tr>
<td>40. (40) + (11)</td>
<td>6.61</td>
<td>6.61</td>
<td>6.61</td>
<td>6.61</td>
</tr>
<tr>
<td>41. (40) + (11)</td>
<td>3.31</td>
<td>3.31</td>
<td>3.31</td>
<td>3.31</td>
</tr>
<tr>
<td>42. (38) + (10)</td>
<td>3.63</td>
<td>3.63</td>
<td>3.63</td>
<td>3.63</td>
</tr>
<tr>
<td>43. (61) + (43)</td>
<td>2.94</td>
<td>2.94</td>
<td>2.94</td>
<td>2.94</td>
</tr>
<tr>
<td>44. max of (40) + (43)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

**FIGURE A-6** Worksheet "B" for Webster Equation for average queue at beginning of green period. (Example)
<table>
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<tr>
<th>Phase</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane 1</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>1. Autos</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Trucks</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Total left turns</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Total right turns</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. Autos straight</td>
<td></td>
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</tr>
<tr>
<td>6. [2] + 1.6</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>7. [3] + 1.6</td>
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<td></td>
</tr>
<tr>
<td>8. (6) + 1.6</td>
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<td></td>
</tr>
<tr>
<td>9. (3) + (6) + (7) + 8</td>
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<td></td>
</tr>
<tr>
<td>10. (8) + 3600</td>
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</tr>
<tr>
<td>11. Critical lane vol. (vph)</td>
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</tr>
<tr>
<td>13. Crossing width (ft.)</td>
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</tr>
<tr>
<td>14. Approach speed (mph)</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>15. Amber [TABLE A-1]</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>16. Amber - Phase Max</td>
<td></td>
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</tr>
<tr>
<td>17. All red</td>
<td></td>
<td></td>
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<tr>
<td>18. k0 [TABLE A-2]</td>
<td></td>
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</tr>
<tr>
<td>19. (17) + (18) + 3.7</td>
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<tr>
<td>20. K = Z [19]</td>
<td></td>
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</tr>
<tr>
<td>21. (11) + 0.00058</td>
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<td></td>
</tr>
<tr>
<td>22. Y = Z [21]</td>
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<tr>
<td>23. Cn [FIGURE A-8]</td>
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</tr>
<tr>
<td>24. (22) + (23)</td>
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<td></td>
</tr>
<tr>
<td>25. (25) + 24</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>26. (28) + (19)</td>
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<td></td>
</tr>
<tr>
<td>27. (26) + 24</td>
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</tr>
<tr>
<td>28. k2 = 2.1 + [26] + [28]</td>
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<tr>
<td>29. M = [25] + [23]</td>
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</tr>
<tr>
<td>30. A1 [TABLE A-3]</td>
<td></td>
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</tr>
<tr>
<td>32. M = [10] - [23]</td>
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<tr>
<td>33. P1 [TABLE A-5]</td>
<td></td>
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</tr>
<tr>
<td>34. (23) + (30)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35. (31) + (10)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>36. (31) + 35</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>37. (100 - 33) + 100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>38. (36) + 31</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>39. (23) + (27) + (17)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40. (10) + 39</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>41. (40) 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42. (38) + (10)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>43. [41] + [42]</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>44. max of [40] [43]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**FIGURE A-7** Worksheet "B" for Webster Equation for average queue at beginning of green period.
Figure A-8  Optimal cycle length, $C_0$
<table>
<thead>
<tr>
<th>Intersection width, ft.</th>
<th>Approach speed, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td>20</td>
<td>4</td>
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<tr>
<td>30</td>
<td>5</td>
</tr>
<tr>
<td>40</td>
<td>6</td>
</tr>
<tr>
<td>50</td>
<td>6</td>
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<tr>
<td>60</td>
<td>7</td>
</tr>
<tr>
<td>70</td>
<td>8</td>
</tr>
<tr>
<td>80</td>
<td>8</td>
</tr>
</tbody>
</table>

Table A-1 Yellow period (in seconds) for various approach speeds and intersection widths [After (110)]

<table>
<thead>
<tr>
<th>Crossing width (ft.)</th>
<th>Approach Speed, mph</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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Table A-2 Lost time $k_2$ (sec.)

(Vehicle length = 17')
### Table A-3 Tabular Values of A for Use with Worksheet “A”

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### Table A-4 Tabular Values of B for Use with Worksheet “B”

[After (208)]
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Table A-5: Tabular values of $F$ for use with Worksheet "B". [After (208)]
APPENDIX B

PERMIT APPLICATION
HIGHWAY PROJECTS ONLY

STATE OF CONNECTICUT
DEPARTMENT OF ENVIRONMENTAL PROTECTION
AIR COMPLIANCE UNIT

APPLICATION FOR AN INDIRECT SOURCE CONSTRUCTION PERMIT

SEND COMPLETED APPLICATION TO
Department of Environmental Protection
Air Compliance Indirect Source Group
State Office Building
165 Capitol Avenue
Hartford, Connecticut 06115

The application is divided into two parts:

PART I. Includes owner, prime contact, and agent information; a description of the proposed indirect source; a description and identification of the impacted transportation network; identification and justification of traffic modeling; and other information.

PART II. Includes the queuing analysis and additional information.

PART I must be reviewed and approved by the DEP prior to the submission of PART II. The DEP will not consider an application for an Indirect Source Construction Permit officially "received" until all required information has been filed. If a particular part of a requirement is not applicable and therefore cannot be fulfilled, indicate so by writing "N/A". The Construction Permit will be issued subject to operating the facility in compliance with the design and operation described in this application. Any significant alterations to the design or operational features of a proposed indirect source which are made during the permit review period must be reported to the DEP. Submit the application in quadruplicate.

Application is hereby made for an Indirect Source Construction Permit pursuant to Section 19-568-120 of the Regulations for the Abatement of Air Pollution of the Regulations of Connecticut State Agencies.
PART I OF THE APPLICATION

A. OWNER AND PRIME CONTACT INFORMATION

1. NAME OF OWNER: _____________________________________________

2. ADDRESS: NO. ______ STREET _________________________________
              CITY ______________________ STATE ____ ZIP

3. NAME OF PRIME CONTACT: ____________________________________
   (Designate an individual to work with the DEP who has the authority to
   speak for and make commitments on behalf of the applicant.)

4. TITLE: ______________________ PHONE: ( ) __________

5. FIRM: _____________________________________________________

6. ADDRESS: NO. ______ STREET _________________________________
              CITY ______________________ STATE ____ ZIP

B. AGENT INFORMATION

(Complete this section only if the applicant is someone other
than the owner; e.g., consultant, attorney, developer, etc.)

1. NAME: _____________________________________________________

2. TITLE: ______________________ PHONE: ( ) __________

3. FIRM: _____________________________________________________

4. ADDRESS: NO. ______ STREET _________________________________
              CITY ______________________ STATE ____ ZIP

C. INDIRECT SOURCE DESCRIPTION

1. TYPE - Specify whether the proposed project is:
   a. [ ] new highway; or
   b. [ ] a modification to an existing highway.

2. IDENTIFICATION:
   a. Name of Highway: _________________________________________
   b. Route Number: ____________________________________________
   c. Project Number: __________________________________________
3. LOCATIONAL DESCRIPTION:
   a. Dominant Direction: ________________________________
      (e.g., east-west, northeast-southwest, etc.)
   b. FROM:  
      i. CITY ________________________________
      ii. INTERSECTION ______________________
   c. TO: 
      i. CITY ________________________________
      ii. INTERSECTION ______________________

4. SIZE:
   a. DDHF: ________________________________
   b. Number of Lanes (One Direction):
      i. Maximum: ________________________________
      ii. Minimum: ________________________________
   c. Length (miles): ___________________________

5. ESTIMATED DATE OF COMPLETION: ___________________________

B. DESCRIPTION OF TRAFFIC DISTRIBUTION METHODOLOGY

Submit a description of the methodologies used to determine:

1. The origin of trips to the proposed highway (e.g., gravity model, etc.); and
2. The assignment of these trips to specific roadways (e.g., minimum travel time, etc.).

(Items 1 and 2 above should be labeled Attachments D-1 and D-2, respectively.)

B. IDENTIFICATION OF THE TRANSPORTATION NETWORK

1. Submit a Transportation Network Map(s), drawn to scale, of the transportation network which will be impacted by the proposed highway. (Label attachment E-1 or, if more than one map is submitted, Attachments E-1a, E-1b, E-1c,...) Include on the map(s) the following:
   a. All existing and planned roads for the distance along which the traffic from the proposed highway or modification will contribute the equivalent of at least ten percent (10%) of the AADT-1 (No-Build) peak hourly traffic volume on at least one directional link approaching an intersection;
   b. All existing and planned roadways and intersections which will intersect or otherwise interface with the proposed highway; and
   c. The proposed highway project.
2. A Site Map(s), drawn to scale, is required. (Label Attachment E-2 or, if more than one map is submitted, Attachment E-2a, E-2b, E-2c, etc... Include on the map(s) the following:
   a. The proposed highway;
   b. The existing use of any properties abutting the proposed highway; and
   c. Any exit from or entrance to the proposed highway from any properties required under item b above.

   NOTE: Items b and c may be omitted if the proposed highway is a limited access highway, although the exits and entrances should still be indicated.

3. On the Transportation Network Map(s) and Site Map(s), assign each required intersection or other point of possible connection and each necessary dummy intersection (i.e., intersections shown on the map which do not have any approach directional link described in Section E-1a, but are necessary to provide approach link designations for significantly impacted intersections and to ensure that the traffic network was defined correctly) a unique consecutive number beginning with "1". Indicate the street names and route numbers, if any; also, after completing Table 1, indicate the actual resultant peak hourly traffic percent increase (see E-1a) on each approach directional link. An example of a section of a transportation network follows:

**EXAMPLE E-3:**

![Diagram of a transportation network example]

4. Provide a list of the alterations to the existing transportation network (e.g., road widening, changes in signalization, etc.) that will be completed within one year after the opening of the proposed highway. (Label Attachment E-4.)

5. Provide a list of each roadway directional link which is shown on the Transportation Network Map(s) and Site Map(s) in the format shown in the example below. (Label Attachment E-5.) *Indicated by an asterisk* any directional link where the traffic flow is obstructed away from the intersection.

**EXAMPLE E-5:** (The list should be prepared in the following format which is based on Example E-3 above.)

<table>
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<th>INTERSECTION NUMBER</th>
<th>INTERSECTION LOCATION</th>
<th>ASSOCIATED DIRECTIONAL LINKS</th>
</tr>
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<td>1</td>
<td>Elm &amp; Route 10</td>
<td>2.01, 1.01</td>
</tr>
<tr>
<td>2</td>
<td>Elm &amp; Maple Stree</td>
<td>1.92, 3.02</td>
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<td>3</td>
<td>Maple Street &amp; Rte. 10</td>
<td>1.03, 2.03, 4.03</td>
</tr>
<tr>
<td>4**</td>
<td>(Dummy Intersection)</td>
<td>N/A</td>
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</table>
NOTE:

(i) 1.02 represents the directional link along Elm Street going from Intersection 1 to Intersection 2.
(ii) 2.01 represents the directional link along Elm Street going from Intersection 2 to Intersection 1.

6. Describe the availability and type of public transportation existing or projected to serve the proposed highway. (Label Attachment E-6.)

F. IDENTIFICATION OF AREAS TO BE MODELED IN PART II

1. Complete Table II, attached, for all intersections described in Section E-1a.

2. The applicant should indicate all points of possible congestion (e.g., intersections, toll booths, etc.) to be modeled using the following guidelines:

   a. All intersections described in Section E-1a for which at least one approach link is required as indicated by the screening charts in Table II attached;
   b. All exits and entrances to the proposed highway and all points of possible congestion on the proposed highway (e.g., tunnels, toll booths, etc.); and
   c. All intersections requested by the DEP after reviewing the above.

3. List all those intersections selected using the above guidelines in the same format as shown in Example E-5. (Label Attachment F-3.) It should be noted that if any approach link to an intersection requires modeling, every approach link to that intersection requires modeling in PART II of this application.

G. IDENTIFICATION OF TIME PERIODS TO BE MODELED IN PART II

1. Complete Table II: using the following guidelines:

   a. For all points of possible congestion on the proposed highway select the peak hour and peak eight hour time periods associated with the highest total traffic volumes traveling through such areas.
   b. For all intersections described in Section F-2a above, and exits and entrances, select the peak hour and the peak eight hour time periods associated with the highest total traffic volumes traveling through the intersection; provided that at least one approach link will carry traffic contributed by the highway equivalent to at least ten percent (10%) of the ETC+1 (No-Build) peak hourly traffic volumes or such links during the chosen time periods.
H. TRAFFIC QUEUING MODEL

Identify which "Traffic Queuing Model" will be used in the analysis required in PART II, Section A, of the application. If this model is not one of the previously accepted methods, a description, documentation and verification of the model must be submitted. (Label Attachment X.)

I. ADDITIONAL INFORMATION

Additional data may be required concerning one or more of the following:

1. Transportation alternatives;
2. Air quality monitoring; and
3. Traffic monitoring

The Commissioner retains the discretion to identify and require analyses of elements of the transportation network which he has reason to believe may cause or contribute to a violation of any National Ambient Air Quality Standard. In addition, upon request and a satisfactory showing by the applicant, the Commissioner may modify any item in the application which he determines is not necessary to fulfill the requirements of Section 19-508-100 of the Administrative Regulations for the Abatement of Air Pollution.

I certify that all statements made in connection with PART I of this application and in the attachments thereto are true and complete. (The signator is subject to provisions of the Connecticut General Statutes regarding false and misleading statements.)

__________________________________________
SIGNED

__________________________________________
TITLE

__________________________________________
DATE

FOR OFFICE USE ONLY

AQCR_____ COUNTY _____ TOWN __________ URID _____ STATE __________
SIC _____ ZONE X____ Y____ DATE REC'D. ____/____/____
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¹L.N.: Intersection Number
²L.N.: Approach Link Number
³PHT (No-Build): ETC + 1 (No-Build) Peak Hourly Traffic
⁴PHT (Build): ETC + 1 (Build) Peak Hourly Traffic
⁵PHT % Increase: \( \frac{\text{PHT (Build)} - \text{PHT (No-Build)}}{\text{PHT (No-Build)}} \times 100 \)
TABLE III
IDENTIFICATION OF PEAK HOUR AND PEAK EIGHT HOUR TIME PERIODS

<table>
<thead>
<tr>
<th>INTERSECTION NUMBER</th>
<th>ONE HOUR TIME PERIOD (HOUR OF THE DAY)</th>
<th>EIGHT HOUR TIME PERIOD (HOURS OF THE DAY)</th>
</tr>
</thead>
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</table>
PART II OF THE APPLICATION

A. CUTTING ANALYSIS - Table IV

1. A cutting analysis is required for the PMT and PHMT periods for the existing and all required projected traffic situations; i.e., build and no-build for estimated time of completion (ETC) + one (1) year, ETC + five (5) years, and ETC + ten (10) years. (Label Attachment A.)

2. Sketch each intersection in the space provided in Table IV. Indicate link directional flow using arrows. Submit for each approach link the required information for:

   (i) the presently existing transportation network; i.e., existing;
   (ii) the transportation network as projected for one, five, and ten years after the proposed highway begins operating; i.e., ETC+1(Build), ETC+5(Build), and ETC+10 (Build); and
   (iii) assuming the proposed highway is not built, the transportation network as projected for one, five, and ten years after the source was to be in operation; i.e., ETC+1(No-Build), ETC+5(No-Build), and ETC+10(No-Build).

2. Calculate the average number of idling vehicles per lane (i.e., queue length) which will be waiting at the traffic control on each approach link. In the case of signalized intersections, the queue length should be the average during the queuing time.

B. ADDITIONAL DATA MAY BE REQUIRED CONCERNING ANY OF THE FOLLOWING:

1. Transportation alternatives;
2. Air quality monitoring;
3. Traffic monitoring.

The Commissioner will notify you in writing of any such additional requirement. If such additional information is required, an application will not be considered "received" until all required information is on file with the DEP.

I certify that all statements made in connection with PART II of this application and in the attachments thereto are true and complete. (The signator is subject to provisions of the Connecticut General Statutes regarding false and misleading statements.)

[Signature]

[Title]

[Date]
GLOSSARY

ADT: Average daily traffic, reported for both the approaching (ARR) and departing (DEP) directional links.

# of Approach Lanes:
For the directional link being analyzed, the number of lanes approaching the intersection. In the above sample diagram, directional link 6.03 has two (2) straight-thru lanes and one (1) right-turn approach lane.

S: Straight thru lane
L: Left turn only lane
R: Right turn only lane

Approach Volume: The traffic volume on the approaching lane(s) per time period.

Control: Type of traffic control at the intersection (e.g., signal, stop sign, etc.).

Day: Specify the day on which the analysis is being performed by indicating holiday, weekend or weekday.

# of Depart Lanes:
The number of lanes countercurrent to the directional link being analyzed. For directional link 6.03 the departing link is designated 3.04 and has two (2) lanes.

Depart Volume: The traffic volume on the departing directional link per time period.

Directional Link Number: The directional link number classifies the direction in which the traffic is moving on a road segment (e.g., traffic moving from intersection two (2) toward intersection three (3), in the figure above, would be classified as traveling on directional link 2.03).

D/C: The demand-capacity ratio for the approaching directional link.

Intersection Number: The potential point of congestion being analyzed assigned a unique consecutive number by the applicant according to the instructions for the Transportation Network Map.
<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>PNH Existing:</td>
<td>Peak hour traffic which presently exists on the roadway.</td>
</tr>
<tr>
<td>PNH ETC + 1 (Build):</td>
<td>Peak hour traffic projected for the roadway one (1) year after the estimated time of completion (ETC+1) of the proposed highway, assuming the proposed highway is built.</td>
</tr>
<tr>
<td>PNH ETC + 5 (Build):</td>
<td>Peak hour traffic projected for the roadway five (5) years after the estimated time of completion (ETC+5) of the proposed highway, assuming the proposed highway is built.</td>
</tr>
<tr>
<td>PNH ETC + 10 (Build):</td>
<td>Peak hour traffic projected for the roadway ten (10) years after the estimated time of completion (ETC+10) of the proposed highway, assuming the proposed highway is built.</td>
</tr>
<tr>
<td>PNH ETC + 1 (No-Build):</td>
<td>Peak hour traffic projected for the roadway one (1) year after the estimated time of completion (ETC+1) of the proposed highway, assuming the proposed highway will not be built.</td>
</tr>
<tr>
<td>PNH ETC + 5 (No-Build):</td>
<td>Peak hour traffic projected for the roadway five (5) years after the estimated time of completion (ETC+5) of the proposed highway, assuming the proposed highway will not be built.</td>
</tr>
<tr>
<td>PNH ETC + 10 (No-Build):</td>
<td>Peak hour traffic projected for the roadway ten (10) years after the estimated time of completion (ETC+10) of the proposed highway, assuming the proposed highway will not be built.</td>
</tr>
<tr>
<td>PNH Existing:</td>
<td>Peak eight hour traffic which presently exists on the roadway.</td>
</tr>
<tr>
<td>PNH ETC + 1 (Build):</td>
<td>Peak eight hour traffic projected for the roadway one (1) year after the estimated time of completion (ETC+1) of the proposed highway, assuming the proposed highway is built.</td>
</tr>
<tr>
<td>PNH ETC + 5 (Build):</td>
<td>Peak eight hour traffic projected for the highway five (5) years after the estimated time of completion (ETC+5) of the proposed highway, assuming the proposed highway is built.</td>
</tr>
<tr>
<td>PNH ETC + 10 (Build):</td>
<td>Peak eight hour traffic projected for the roadway ten (10) years after the estimated time of completion (ETC+10) of the proposed highway, assuming the proposed highway is built.</td>
</tr>
<tr>
<td>PNH ETC + 1 (No-Build):</td>
<td>Peak eight hour traffic projected for the highway one (1) year after the estimated time of completion (ETC+1) of the proposed highway, assuming the proposed highway will not be built.</td>
</tr>
</tbody>
</table>
PEHT ETC + 5
(No-Build): Peak eight hour traffic projected for the highway five (5) years after the estimated time of completion (ETC'5) of the proposed highway, assuming the proposed highway will not be built.

PEHT ETC + 10
(No-Build): Peak eight hour traffic projected for the highway ten (10) years after the estimated time of completion (ETC'10) of the proposed highway, assuming the proposed highway will not be built.

S: The total vehicle-seconds of delay per lane experienced by cars idling at a point of congestion, divided by the total time (in seconds) during which the idling occurs.

WQ Time: The percent of the time period that queuing exists on the lane(s).

Red/Cycle: The red light time divided by the total cycle time.

Red(red time): The red light time (sec.)

Road Speed: The posted free flow road speed (MPH).

Time Period: Identifies the one or the eight hour period, and the year being analyzed.
EXAMPLE PROBLEM:

GIVEN:

One-Lane Approach - 450 vehicle/hour over an 8 hour period

From Signal Plan TOTAL CYCLE = 60 seconds
Approach RED TIME = 22 seconds

ANALYSIS

Using Chart No. 1 (One-Lane Approach)

Total Flow = 450 vehicle/hour
B=22+3 = 25 seconds
Therefore Queue Factor = 4.7

Use the Upper Graph on Chart No. 1
Queue Factor = 4.7
Red Time/Cycle = 0.42

Since the corresponding intersection of these two values is on the "DO NOT" side of the upper curve; an in-depth queuing analysis is not necessary.
Preliminary Screening of Approaches With Exclusive Turn Channels

The following is the recommended procedure for determining whether or not an approach with exclusive turn channeling requires an in-depth queuing analysis.

Step 1.
Determine the queue factor for the turn channel(s) using the lower graph of the appropriate screening chart; use the actual turning traffic volume and red light time.

Step 2.
Determine the queue factor for the straight through lane(s) using the upper graph of the appropriate screening chart; use the actual traffic volume and red light time.

Step 3.
Select the larger of the two queue factors from Steps 1 and 2 above. Using the graph on the chart corresponding to the total number of lanes in the approach, determine whether or not an in-depth queuing analysis is required. Use the greatest red light time to total cycle time ratio.

Note
If the total number of straight through and exclusive turn lanes is four or more, the approach must be subjected to an in-depth queuing analysis.

Example Problem

<table>
<thead>
<tr>
<th>GIVEN</th>
<th>LEFT TURN LANE(S)</th>
<th>STRAIGHT LANE(S)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Lanes</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Traffic Volume, veh/hr</td>
<td>100</td>
<td>700</td>
</tr>
<tr>
<td>Total Cycle Time, Seconds</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>Red Light Time, Seconds</td>
<td>32</td>
<td>42</td>
</tr>
<tr>
<td>Red to Cycle Ratio</td>
<td>0.53</td>
<td>0.7</td>
</tr>
</tbody>
</table>

1 Average hourly volume over an eight (8) hour period.
Step 1.

*Left Turn Lane*
Total flow = 100 veh/hr, \( R = 32 + 5 = 37 \) seconds
From the lower graph in Chart 1, the queue factor equals 1.0.

Step 2.

*Straight Through Lane*
Total flow = 700 veh/hr, \( R = 42 + 5 = 47 \) seconds
From the lower graph in Chart 2, the queue factor equals 5.8.

Step 3.

a. The total number of lanes is 3.

b. The larger queue factor is 5.8

c. The greatest red time to total cycle time ratio is 0.7.

From the upper graph in Chart 3, an eight (8) hour period in-depth queuing analysis is required for this approach.
APPENDIX C

SYMBOLS
SYMBOLS

A = amber time
A_j = trip attractions at zone j
C = cycle time
C_q = factor used in Eqn. 3.6
CO = carbon monoxide concentration
CO_{ff} = free flow carbon monoxide concentration
CO_{q} = idling carbon monoxide concentration
D = total delay
D = denotes deterministic
E = unit free flow CO concentration
E( ) = denotes expected value of ( )
E_k = denotes Erlang distribution
F = road speed correction factor
F_{ij} = travel time factor for a trip between zones i and j
F(k) = probability of k or fewer arrivals
G = denotes general distribution
H = height of emitter
H = headway
I = an index
I = index of dispersion
K = shape parameter
K = Erlang parameter
K_1, K_2 = lost times
K_{ij} = socio-economic adjustment factor between zones i and j
L = critical lag
L = lower bound
M = denote exponential distribution
N = number of idling cars
P_B = proportion of vehicles stopped
P(x) = probability of exactly x arrivals
P_n(t) = probability of n vehicles in the system at time t
P_i = trip productions at zone i
Q = mass emission rate per unit area, Equation (3.1)
Q = mass emission rate, Equation (3.7)
Q = total mass of emitted material
Q' = mass emission rate
Q_f = free flow emission factor
Q_d = idling CO emission rate
R = red time
S = geometric parameter, Figure 3.1 and Equation (3.7)
S^2 = variance of interval distribution
τ = 1/λ
T_{ij} = trip interchange between zones i and j
U = wind speed
U = upper bound
V = hourly traffic volume
V = average wind speed
W = intersection width
X_c = equilibrium concentration
Z = geometric parameter, Figure 3.1
C = cycle time
d = delay
\( h \) = headway
\( n \) = number of vehicles in system
\( p(t) \) = probability distribution
\( q \) = flow rate
\( q \) = mass emission rate per unit length of line
\( r \) = effective red time
\( s \) = average service time
\( s \) = departure headway
\( t \) = time
\( u \) = an increment, Figure 4.14
\( \bar{u} \) = mean wind speed
\( v \) = speed of traffic
\( x \) = degree of saturation
\( y = q/s \)
\( \xi \) = location parameter
\( \delta \) = scale parameter
\( \delta \) = headway
\( \lambda \) = arrival rate
\( \lambda = g/c \) (Webster Equation)
\( \mu \) = service rate
\( \rho \) = traffic intensity
\( \sigma \) = standard deviation
\( \sigma, \gamma \) = standard deviation of material concentration
\( \tau \) = minimum headway
\( \tau \) = critical gap
\( \phi_H \) = red time/cycle time
\( x \) = mass concentration

\( \chi \) = predicted CO concentration

\( \gamma \) = gamma function

\( \Delta \) = geometric parameter, Equation (3.7)

\( \Delta \) = an increment