Cost-Effectiveness of Smart Traffic Signals

by

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B.A., Mathematics and Economics, Macalester College (1983)  

Submitted to the Sloan School of Management  
in Partial Fulfillment of  
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Abstract

This thesis examines the cost-effectiveness of real-time traffic-responsive ('smart') signals. The study is motivated by current interest in intelligent vehicle/highway systems and a hypothesis that a policy devised to encourage the implementation of smart signals nationwide would, as the right-turn-on-red law, yield improvements in the three key measures of performance of a traffic control system: (1) time delay, (2) fuel consumption, and (3) air pollution.

The thesis begins by summarizing the results of a survey mailed to traffic engineers across the nation. The primary aim of the survey was to identify current signal control technologies for isolated intersections and to determine a first-order approximation for the number of existing signals operated under each type of control. There were two secondary objectives: identify the most commonly used methodologies for developing signal timing plans, and identify issues of importance to the urban traffic engineer including explicitly used performance measures.

Analyzing the cost-effectiveness of smart traffic signals requires an understanding of the tradeoffs which exist among the three key performance measures within a given signal control technology. Models for a fixed-cycle signal are developed from which Pareto optimal sets for the performance measures are determined.

Two different techniques for determining the expected value of each of the performance measures under "perfect information" are used. The first technique is an existing discrete-time dynamic programming based algorithm which produces exact solutions. The second technique is a new heuristic developed in this thesis which attempts to identify optimal deviations from an initially imposed fixed-cycle schedule (F-COD).
ABSTRACT

There are two uses for the expected value of the measures under perfect information: to determine the magnitude of potential savings which could be realized by the implementation of smart traffic signals both at individual intersections and nationwide, and to evaluate the expected value of perfect information. It is found that there are substantial savings in time, fuel, and pollution which can be realized under perfect information. In addition, it is discovered that there exist substantial differences among the values of the performance measures depending upon which one is minimized.

Two specific smart signal technologies for isolated intersections are found to be cost-effective. One strategy is a variation of conventional vehicle-actuation; it employs two detectors per approach (smart V-A) and is basically a gap seeking strategy. The second strategy optimizes a specific performance measure over a short-term rolling horizon (ROPAC). The payback period for the excess installation cost of ROPAC at a "typical" intersection is conservatively estimated to be under one year, and for smart V-A to be under two years. If all isolated intersections nationwide were operated under ROPAC, a conservative estimate of the daily monetary savings in terms of time and fuel is $13.0M, and the analogous amount for smart V-A is $3.6M. Comparisons are also made between the possible benefits from smart signal control strategies and other nationally supported policies: the right-turn-on-red law, the 55 mph speed limit, the automobile emissions standards, and the corporate average fuel economy (CAFE) laws. In most cases, ROPAC compares favorably.

Thesis Supervised by:

Richard C. Larson  Professor of Electrical Engineering and Computer Science
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Chapter One

Introduction

Over fifty years ago (1937), California allowed the first right-turn-on-red (RTOR) maneuver with an authorizing sign. The primary aim of the policy was to reduce needless motorist delay at signalized intersections [Parker 1976]. Ten years later, after widespread use and successful experience, the California legislature amended the law to permit RTOR at all signalized intersections unless posted otherwise. Gradually, a few other states adopted some form of RTOR convinced it saves unnecessary delay without a significant increase in accidents. Although, as recent as 1972, only eight states had a general permissive RTOR law [Chang et al. 1977].

In 1976, there were 34 states which had a general permissive RTOR law. This widespread policy change was largely motivated by the nation's desire to conserve fuel [Chang et al. 1977]. Studies indicated that fuel savings ranging from 4 percent to 8.9 percent for all vehicles could be realized under RTOR [McGee and Warren 1976, Chang et al. 1977]. Parker [1976] estimated that a general permissive RTOR could save Virginia motorists over 3 million gallons of fuel annually. He also found that there was an average of 14 seconds saved for every delayed right turn which implies over 6500 seconds of delay saved per day at a typical RTOR intersection in Virginia.

Finally, in addition to the documented delay and fuel savings cited above, it was generally known that RTOR would also have a positive impact with regard to reducing congestion and air pollution.
Federal adoption of RTOR was primarily motivated by its fuel conservation potential. It is an example of a policy designed to increase the energy efficiency of vehicle travel per mile by improving the quality of traffic flow. Another policy which could improve the quality of traffic flow is the use of "smart" or real-time demand-responsive traffic signal control techniques. This is the subject of my thesis. It is my belief that such a policy would produce fuel savings of an order of magnitude greater than those realized under RTOR. It is also my belief that such a nationally supported policy would have substantial benefits in terms of reducing vehicular delay and vehicular emitted air pollutants.

Section 1.1 provides some background, perspective and motivation for my work. In the following section, I use an illustrative example to delineate some of the issues surrounding the development of smart signal timing algorithms and technologies. I close this chapter with a statement of my objectives and an overview of the thesis content and organization.

SECTION 1.1 Background, Perspective and Motivation

Traffic management systems are an array of human, hardware and software components designed to monitor, control and manage road users in order to achieve the highest quality of traffic flow possible. These have existed since there has been identifiable congestion (which antedates the car) and the need for them will remain as long as congestion continues to exist. An essential part of these systems are traffic signals. Approximately two-thirds of all vehicle miles of travel and even a higher fraction of vehicle hours of travel are on roadways controlled by traffic signals [Wagner 1980]. It is of no surprise that quality of the traffic signal operations is the major determinant of the quality of urban vehicular traffic flow.

1.1.1 Brief History and Background

The first identifiable traffic management systems designed to improve traffic flow consisted solely of static signs and markings. A more dynamic form of control, manually operated semaphores, first appeared in 1868 on the streets of London. In 1914, the first electric traffic signal, invented by James Hoge, was installed in Cleveland, Ohio (a fact ignored by the city of Detroit which has enshrined its own first U.S. traffic light in the Ford Museum in Dearborn) [Gazis 1974]. This device is the
origin of today’s three-color traffic signals. Fourteen years later, the first actuated signals were installed at New Haven, East Norwalk and Baltimore [Homburger and Kell 1988]. Instead of assigning a predetermined fixed amount of green to each approach, these signals respond to a demand for the right-of-way as registered through a detector placed in the road.

An obvious purpose of signals is to alleviate congestion by resolving conflicts between traffic flows. On the other hand, improperly timed traffic signals may exacerbate congestion. Basically, there are two distinguishable improvement methods applicable to traffic signals, either in isolation or in a coordinated systems, which are able to enhance traffic flow quality: better signal timing plans using the existing hardware and software, or new, more real-time demand-responsive timing techniques which may require an upgrade of the existing software and hardware.

1.1.2 Perspective and Motivation

A new form of traffic control system is emerging which will use, in addition to traffic signals, an assortment of innovative tools. During the past few years, transportation officials have envinced growing interest in intelligent vehicle/highway systems (IVHS) as the next generation of traffic management systems [USDOT 1989a and 1989b, Mobility2000 1989, IEEE 1991]. The main impetus for developing IVHS technologies has been the strain exerted on existing roadways by overwhelming congestion. The 100 million commuters in the United States spend two billion hours a year in traffic jams which results in an estimated $73 billion loss in productivity. These figures are projected to increase five-fold by the year 2005 [TR News 1989]. Traditionally, the United States’ response to unacceptable levels of congestion has been to build more streets and urban highways. This option is no longer feasible. In the past two decades, expenditures on roads by all levels of government has not kept pace with increasing road transportation demand. Between 1970 and 1988, total vehicle miles travelled in the U.S. increased 78 percent compared to an increase of 13.6 percent in inflation-adjusted expenditures [Wallis 1990]. Even when highway administrators can meet projected costs for a new urban highway project, in many areas they cannot overcome political resistance to more highway building by affected communities [Koltnow 1989, Sheldrick 1990]. IVHS offer a new solution to the congestion problem which makes use of existing streets and highways by improving traffic flow.
The main applications of IVHS can be grouped into four categories: advanced traffic management systems (ATMS), advanced driver information systems (ADIS), heavy vehicle and commercial operations (HVO), and automated vehicle control (AVC) systems. ATMS involve computer controlled traffic signals that adapt to changes in traffic flows in real-time, freeway access control systems, and systems which combine freeways and adjacent arterial streets into a centrally managed road network. ADIS are located in an individual's vehicle and are interconnected through a communication link with traffic management centers. These systems provide in-vehicle navigational guidance. Advanced versions would be capable of transmitting information on vehicle location, destination, and speed to a traffic management center [Mobility2000 1989]. Heavy vehicle and commercial operations systems encompass weigh-in-motion, automatic vehicle identification, automatic vehicle location, automatic cargo identification, and automatic driver identification technologies. Lastly, AVC systems range from in-vehicle collision avoidance systems to vehicles capable of travelling from origin to destination without any driver intervention in any traffic environment.

Improvements in signal timing plans, given the current technology, have been documented as offering the most immediate payoffs for traffic congestion with the smallest investment cost. It has been estimated by engineers within the Federal Highway Administration (FHWA) that of the approximately 240,000 urban signalized intersections in the United States, about 148,000 need upgrades in physical equipment and signal timing optimization while an additional 30,000 require timing optimization only [Wallis 1990]. The FHWA estimates that if all traffic signals throughout the nation were optimally timed, approximately 2 million gallons of gasoline per day could be saved as a result of smoother traffic flow (less delay and stops). If the control hardware at these 240,000 signals was modernized in addition to optimally timing the signals, a total of 5 million gallons of gasoline per day could be saved [Euler 1983]. Signal timing optimization is cost-effective. It has been estimated that 21 to 29 gallons of fuel could be saved for every dollar spent (evaluated in current fuel prices) on signal timing optimization [Wagner 1980].

The encouraging results of past signal retiming programs (Section 8.6.1) which use established timing techniques provide strong motivation to investigate the cost-effectiveness of implementing newly proposed algorithms for real-time traffic-responsive signals nationwide achievable through upgrades in the signal control software. Thus, this is the area within ATMS on which I choose to focus. Relative to
other IVHS technologies, ATMS requires the least initial investment and promises the most immediate benefits. I further narrow my focus to traffic-responsive strategies designed for isolated intersections.

In addition to concern about mushrooming urban congestion, the public is also becoming concerned about fuel consumption and air pollution issues. Fuel used for transportation accounts for more than 60 percent of all petroleum used in the U.S. and accounts for 40 percent of all hydrocarbon emissions and two-thirds of the carbon monoxide emissions [Wright 1990], which makes it the largest user of petroleum in the country and a major source of urban smog and "greenhouse" gasses [USDOT 1990]. Within the current timing schemes, tradeoffs exist between more efficient fuel use, less air pollutants emitted and decreased delay. It is not possible, through signal retiming, to optimize all three measures under a given technology, but improvements in all three can be achieved through the use of smarter technologies.

From this brief discussion, three motivational factors underlying my work emerge. The first is the transportation professional and academic communities' interest in IVHS. Of all the applications within IVHS, advanced traffic management systems offer the most immediate payoffs and a major component of ATMS are traffic signals. The second factor arises from the historical trend of timing signals to minimize delay. As fuel conservation and pollution reduction become more significant national issues, it is necessary to explicitly include all three measures when developing signal timing plans. Lastly, establishing signal control algorithms which are more real-time traffic-responsive could produce savings in fuel consumption, air pollution, and time of an order of magnitude greater than the nationally legislated right-turn-on-red policy.

SECTION 1.2 Introduction to Some Specific Issues

During my final years as a graduate student at MIT, I have been living on Massachusetts Avenue (Mass Ave) near Harvard Square. On days it rains or snows, I take a bus to and from school. This bus travels on Mass Ave between MIT and my apartment as part of its route from Dudley Square in Boston and Harvard Square in Cambridge. In the evenings, while I wait for the bus in front of 77 Mass Ave, I have time to contemplate. Over the years there have been two problems that have particularly intrigued me and to which I have given much thought: the mythical Poisson busline and the pedestrian crossing problem. These problems are two of my
personal favorites and are extremely familiar to me as a result of my many semesters as a 6.041\(^1\) teaching assistant. The spot in front of 77 Mass Ave reminds me of these problems and could well be their inspirational source. Fortunately, I am able to use one of the problems as an introduction to some issues examined in my thesis.

As I wait for the bus, I observe the operation of the pedestrian crosswalk located at 77 Mass Ave. This crosswalk is similar to an example presented in 6.041 which I later saw in my Urban OR class\(^2\). The physical layout of the crossing is diagramed in Figure 1.1. The example I use in this section is a variation of the problem found in Urban Operations Research [Larson and Odoni 1981].

![Diagram of the pedestrian crosswalk at 77 Massachusetts Avenue.](image)

One goal of my thesis is to evaluate feasible traffic control technologies for a signalized intersection. This includes control policies that are currently in use on our streets, and adaptive traffic-responsive control strategies that are not yet in use. In the evaluation of the different policies, there are three types of performance measures of particular interest to me: (1) the time delay experienced by the occupant(s) of a vehicle, (2) the amount of fuel consumed, and (3) the amounts of the various

---

\(^1\) The name of this course is Probabilistic Systems Analysis and is taught by Professor Alvin Drake through the Electrical Engineering and Computer Science Department.

\(^2\) The official name of the class is Logistical and Transportation Planning Methods.
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pollutants emitted. Generally, it is not possible to optimize the three performance measures simultaneously. This underscores the need to explicitly understand the tradeoffs between the three measures when evaluating or developing any control strategy. In the remainder of this section, I use the pedestrian crossing problem as a means to introduce some of the relevant issues surrounding the evaluation of traffic signal control policies.

1.2.1 Description of the Illustrative Example

The pedestrian crosswalk at 77 Mass Avenue (Figure 1.1) is used by thousands of MIT students, staff, and faculty each day. The signal is operated on a fixed-cycle. For the pedestrian traffic, a cycle consists of a WALK message, a flashing DON'T WALK message and a steady DON'T WALK message in the indicated sequence. The cycle for vehicular traffic consists of a green, yellow and red indication in the usual order. A fixed-cycle operating mode implies that each message (signal color) lasts for a predetermined duration which does not vary from cycle to cycle.

A version of the pedestrian crossing problem is as follows. Determine which decision rule for operation of the light minimizes the expected waiting time for a randomly arriving pedestrian. Three different decision rules, including the rule currently in use, are analyzed:

- **Rule A:** The length of a steady DON'T WALK signal is \(T_A\) minutes each cycle (current rule).

- **Rule B:** Initiate the WALK message whenever the total number of waiting pedestrians equals \(N_B\).

- **Rule C:** Initiate the WALK message whenever the first pedestrian to arrive during the steady DON'T WALK has waited \(T_C\) minutes.

For the analysis of each rule, the following simplifying assumptions are made. Pedestrians approach the crosswalk independently from the east and west in a Poisson manner with average arrival rates (in arrivals/minute) \(\lambda_e\) and \(\lambda_w\), respectively. If a pedestrian arrives at the crosswalk during a WALK or a flashing DON'T WALK message, she crosses. The period of time which includes the WALK and flashing DON'T WALK messages, referred to as a dump, is constant under each rule and equal to \(D\) minutes. If a pedestrian arrives during a steady DON'T WALK signal, she waits until the next dump at which time all waiting pedestrians cross.
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Note that each decision rule only affects the length of the steady DON’T WALK display. To evaluate the rules, I hold the expected cycle lengths constant and compare the mean pedestrian waiting time for each system.

1.2.2 Expressions for Common Quantities

To evaluate the three decision rules from the perspective of a pedestrian, and later from the perspective of a vehicle, it is necessary to determine, under each rule, the probability density function (pdf) for the length of the steady DON’T WALK signal and the expected time a randomly arriving pedestrian must wait before crossing.

Denote the pdf’s for the length of the steady DON’T WALK display as \( f_{tA}(t), f_{tB}(t), \) and \( f_{tC}(t) \) for rules A, B, and C, respectively. It suffices to consider only the pooled Poisson process of arrivals with parameter \( \lambda = \lambda_e + \lambda_w \) to determine each pdf because each decision rule either utilizes no pedestrian arrival information or only uses information regarding the total number of pedestrian arrivals. The derivations for the pdf’s, which are given in (1.1)-(1.3) below, are in Appendix A.1 and are also found in Urban Operations Research [Larson and Odoni 1981].

\[
\begin{align*}
  f_{tA}(t) &= \mu_0(t - T_A) \quad (1.1) \\
  f_{tB}(t) &= \frac{\lambda^N t^{(N-1)} e^{-\lambda t}}{(N - 1)!} \quad t \geq 0 \quad (1.2) \\
  f_{tC}(t) &= \lambda e^{-\lambda(t - T_C)} \quad t \geq T_C \quad (1.3)
\end{align*}
\]

My results for the expected time a randomly arriving pedestrian must wait before crossing differ slightly from those computed by Larson and Odoni because they assume \( D = 0 \), i.e. the dump is instantaneous. Let \( \bar{w}_A, \bar{w}_B, \) and \( \bar{w}_C \) be the mean waiting times of a randomly arriving pedestrian under rules A, B, and C, respectively. Expressions for these quantities are derived in Appendix A.1 and are summarized by (1.4)-(1.6) below:

\[
\begin{align*}
  \bar{w}_A &= \frac{T_A^2}{2(T_A + D)} \quad , (1.4) \\
  \bar{w}_B &= \frac{N_B(N_B - 1)}{2\lambda(N_B + \lambda D)} \quad , (1.5)
\end{align*}
\]

and

\[
\bar{w}_C = \frac{T_C}{2}\left(2 + \frac{\lambda T_C}{1 + \lambda(D + T_C)}\right) \quad . (1.6)
\]
Using the quantities presented in this subsection, it is possible to determine a preference ordering of the three signal operating modes from a pedestrian’s perspective. This is done next. Later, when evaluating the decision rules for various vehicular performances measures, I again make use of these quantities.

1.2.3 Pedestrian Rank-Ordered Preferences

When determining a rank-ordering for the the three rules it seems natural to use the mean wait per randomly arriving pedestrian as a pedestrian evaluation yardstick. To evaluate the alternative decision rules, I hold the mean cycle time constant and determine which system yields the smallest mean pedestrian wait. Because a cycle consists of a dump, which is the same deterministic length for each rule, and the steady DON'T WALK display, holding the mean cycle time constant is equivalent to holding the mean length of the DON'T WALK message constant for each rule.

Define \( c_i \) to be the length of a cycle under rule \( i \), \( i = A, B, \) or \( C \). Note that \( c_i = D + t_i \) \((i = A, B, \) or \( C \)) follows immediately from the definition of a cycle and the definition of \( t_i \) \((i = A, B, \) or \( C \)) given in the previous subsection. Furthermore, \( c_i = D + \bar{t}_i \), \( i = A, B, \) or \( C \). For convenience, Table 1.1 summarizes the expressions for the expected values of \( t_i \), \( c_i \) and \( w_i \), \( i = A, B, C \).

<table>
<thead>
<tr>
<th>Rule</th>
<th>( f_i(t) )</th>
<th>( t_i )</th>
<th>( \bar{w}_i )</th>
<th>( \bar{c}_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>( \mu_d(t - T_A) )</td>
<td>( T_A )</td>
<td>( \frac{T_A^2}{2(T_A + D)} )</td>
<td>( T_A + D )</td>
</tr>
<tr>
<td>B</td>
<td>( \frac{\lambda^{N_B-1}e^{-\lambda t}}{(N_B-1)!} ) ( \geq 0 )</td>
<td>( \frac{N_B}{\lambda} )</td>
<td>( \frac{N_B(N_B-1)}{2\lambda(N_B + \lambda D)} )</td>
<td>( \frac{N_B}{\lambda} + D )</td>
</tr>
<tr>
<td>C</td>
<td>( \lambda e^{-\lambda(t - T_C)} ) ( \geq T_C )</td>
<td>( \frac{1}{\lambda} + T_C )</td>
<td>( \frac{T_C}{2} \left( \frac{2 + \lambda T_C}{1 + \lambda(D+T_C)} \right) )</td>
<td>( \frac{1}{\lambda} + T_C + D )</td>
</tr>
</tbody>
</table>

Table 1.1 Summary of the relevant expected values for each of the operational rules for the pedestrian crossing.

The mean pedestrian waiting times, \( \bar{w}_i \), can be expressed in terms of \( \lambda \), \( t_i \), \( D \), and \( \bar{c}_i \) \((i = A, B, \) and \( C \)):

\[
\begin{align*}
\bar{w}_A &= \frac{T_A}{2} \left( 1 - \frac{D}{\bar{c}_A} \right) \\
\bar{w}_B &= \frac{T_B}{2} \left( 1 - \frac{D}{\bar{c}_B} - \frac{1}{\lambda \bar{c}_B} \right) \\
\bar{w}_C &= \frac{T_C}{2} \left( 1 - \frac{D}{\bar{c}_C} \right) - \frac{1}{2 \bar{c}_C \lambda^2} 
\end{align*}
\]

(1.7)
For each rule, the mean cycle time is held constant at $\bar{c}$ and the rules are rank-ordered. The rule which produces the smallest pedestrian mean waiting time is the most preferred rule.

**Claim 1.1:** If $\bar{c}_i = \bar{c}$ for $i = A, B, C$, then in terms of the smallest mean pedestrian waiting times, rule B is most preferred, rule A is least preferred and rule C falls somewhere in between.

**Proof:** If the mean cycle times are held constant at $\bar{c}$, this implies that $\bar{t} = \bar{t}_A = \bar{t}_B = \bar{t}_C$. The equations for the mean pedestrian waiting times, (1.7), can be expressed as

$$
\begin{align*}
\bar{w}_A &= \frac{\bar{t}}{2} \left( 1 - \frac{D}{\bar{c}} \right) \\
\bar{w}_B &= \frac{\bar{t}}{2} \left( 1 - \frac{D}{\bar{c}} - \frac{1}{\lambda \bar{c}} \right) \\
\bar{w}_C &= \frac{\bar{t}}{2} \left( 1 - \frac{D}{\bar{c}} - \frac{1}{2 \bar{c} \lambda^2} \right)
\end{align*}
$$

(1.8)

Because $\bar{t}, D, \bar{c},$ and $\lambda$ are all positive, it is clear from equations (1.8) that rule A produces the longest mean pedestrian waiting time implying that it is the least preferred of the rules. Now it remains to be shown that $\bar{w}_C > \bar{w}_B$:

$$
\bar{w}_C = \frac{\bar{t}}{2} \left( 1 - \frac{D}{\bar{c}} \right) - \frac{1}{2 \bar{c} \lambda^2} > \frac{\bar{t}}{2} \left( 1 - \frac{D}{\bar{c}} \right) - \frac{\bar{t}}{2 \bar{c} \lambda} = \bar{w}_B
$$

$$
\frac{1}{2 \bar{c} \lambda^2} < \frac{\bar{t}}{2 \bar{c} \lambda}
$$

$$
\frac{1}{\lambda} < \bar{t}
$$

But under rule C, $\bar{t} = \frac{1}{\lambda} + T_C$ which implies,

$$
\frac{1}{\lambda} < \frac{1}{\lambda} + T_C
$$

$$
0 < T_C
$$

How much difference in the average mean pedestrian wait is there between the three rules? Figure 1.2 is a graph of the mean wait per pedestrian (in minutes) versus $\lambda$, the average arrival pedestrian arrival rate (in arrivals per minute). The lines correspond to the equations in (1.8) and use actual $D$ and $\bar{t}$ values for the crosswalk at 77 Mass Ave. The current value for $D$ is 1/4 minutes and the value for $\bar{t}$ is 7/12 minutes. On the graph, I have indicated two arrival rates of interest. The dark solid vertical line around $\lambda = 54.4$ indicates the observed average pedestrian arrival rate within $\pm 10$ minutes of a mid-day class change. The dashed vertical line around $\lambda = 12.8$ corresponds to the average pedestrian arrival rate observed during a morning
non-class change time. This crosswalk experiences different arrival rates at different times of the day which is also a common phenomenon for vehicular signalized intersections.

![Graph showing Mean Pedestrian Waiting Time Versus Lambda](image)

Figure 1.2 Mean pedestrian waiting time versus $\lambda$ for the pedestrian crossing problem.

As can be seen from the graph, there is not a significant difference between rules A and C when $\lambda$ is greater than 30 arrivals per minute. This information is important when deciding which technology to install. Currently, the technology in place corresponds to rule A. If the pedestrian arrival rate is high, there seems little to be gained by switching to the technology needed for rule C. On the other hand, if the arrival rate is low, rule C seems to offer more savings indicating the choice requires further examination, i.e. how long will it take before the cost of the initial investment is repaid in terms of reduced pedestrian delay? From the graph, it is seen that rule B has demonstrable savings in mean pedestrian wait over both rule A and rule C. However,

---

3 The data were collected by myself and Yun Choi, an UROP (Undergraduate Research Opportunities Program) student who worked for me.
from examining equations (1.8), as \( \lambda \) increases to infinity, the average wait for a randomly arriving pedestrian converges asymptotically to \( \frac{L}{2} \) under each rule. Though rule B does outperform the other rules for the plotted values of \( \lambda \), there exists the problem, which is not uncommon for vehicular signal control, that the technology does not currently exist for implementation of rule B. Rule B requires a mechanism that can accurately determine the number of queued pedestrians and it is not obvious how this can be accomplished. Given this, it seems that rule C may be the better choice due to technological feasibility.

In the next subsection, the above analysis is extended to vehicular performance measures including the delay per randomly arriving vehicle, the total amount of fuel consumed per cycle and the total amount of pollutants emitted per cycle.

1.2.4 Vehicle Rank-Ordered Preferences

Now, assume that the three operating rules of the previous subsection are used to control the vehicular traffic stream. Instead of only examining one performance measure as was done for the pedestrian traffic, I examine all three measures of interest to me: (1) delay experienced by the vehicles occupants, (2) the amount of fuel consumed, and (3) the amount of pollutants emitted. It is possible to produce a rank-ordering of the rules for each of the key measures.

The general problem is as follows. Vehicles arrive at the signal from the north and the south in a Poisson manner with average arrival rates (in arrivals per minute) \( \lambda_N \) and \( \lambda_S \), respectively. Assume the north and south arrival processes are independent. If a vehicle arrives during a green or yellow signal, it clears the intersection. Collectively, a green-yellow sequence is referred to as a dump. Dumps are assumed to be of zero length and all queued vehicles are able to cross. If a vehicle arrives during a red signal, it must wait until the next dump before it is allowed to clear the intersection. The same three rules used for the pedestrian crossing example are analyzed:

**Rule A:** The length of a red signal is \( T_A \) minutes each cycle. (Again, this is the current rule for 77 Mass Ave).

**Rule B:** A dump occurs whenever the total number of queued vehicles equals \( N_B \).

**Rule C:** A dump occurs whenever the first vehicle to arrive after the previous dump has waited \( T_C \) minutes.
Some of the simplifying assumptions make the example unrealistic. The two most objectionable assumptions are dumps of zero duration and that all queued vehicles are able to clear the intersection during the green interval. As for the pedestrian crossing problem, it is possible to analyze the problem with a positive dump length but for simplification purposes it will not be done here. If the vehicular traffic flow is light to moderate, there is a high probability that all queued vehicles are able to clear the intersection during the dump and so this does not affect the analysis. If all queued vehicles are not guaranteed to be able to clear the intersection, the problem becomes more complicated but can be solved. Because the main purpose of this example is illustrative, the simplified problem description above is adequate.

In this subsection, the following notation is used \( i = A, B, C \):

\[ t_i = \text{time (in minutes) between consecutive dumps, or equivalently, the length of a red signal}, \]

\[ w_i = \text{time (in minutes) a randomly arriving vehicle must wait before crossing the intersection}, \]

\[ s_i = \text{number of vehicles that must stop per red signal}. \]

**Delay as a Performance Measure**

As before, the delay criterion is the expected delay per randomly arriving vehicle. The rule which produces the minimum expected vehicle delay is the most preferred. To compare the performance level of each rule, \( \bar{t}_i \) \( i = A, B, C \) is held constant.

**Claim 1.2:** If \( \bar{t}_i = \bar{s} \) for \( i = A, B, C \) then in terms of the smallest mean vehicle waiting times, rule B is most preferred, rule A is least preferred and rule C falls somewhere in between.

**Proof:** Set \( D = 0 \) in Claim 1.1.

**Fuel Consumption as a Performance Measure**

Assume that the amount of fuel consumed per red signal can be expressed as a function of the wait per randomly arriving vehicle, \( w \), and the number of vehicles that stopped per red signal, \( s \). A possible form for the fuel consumption function could be

\[ f(w,s) = \alpha w^2 s + \beta s \]
where \( \alpha \) = conversion coefficient in gallons per (vehicle-minutes)\(^2\) and \( \beta \) = conversion coefficient in gallons per vehicle-stops. The coefficients \( \alpha \) and \( \beta \) depend upon the composition of the traffic, approach, grades, etc.

Using the function \( f(w,s) \), it is possible to determine a preference ordering among the three alternative systems. The system which produces the smallest expected amount of fuel consumption per red light is the most preferred. To determine the ranking, I compute \( E[f(w,s)] = \alpha E(w^2)E(s) + \beta E(s) \) for each system. This equation is valid because the amount of time a randomly arriving vehicle must wait is independent of the number of vehicles that must stop. Define \( \bar{f}_i = \alpha E(w_i^2)E(s_i) + \beta E(s_i) \) for \( i = A, B, C \). Once each \( \bar{f}_i \) is computed, I hold \( \bar{t}_i \) constant, \( i = A, B, C \), and determine which system produces the smallest \( \bar{f}_i \). In order to calculate \( \bar{f}_i \), I need \( E(w_i^2) \) and \( E(s_i) \). These are derived in Section A.2 and are used to produce (1.9)-(1.11) below.

\[
\begin{align*}
\bar{f}_A &= \lambda T_A \left( \frac{\alpha T_A^2}{3} + \beta \right) \\
\bar{f}_B &= N_B \left[ \frac{\alpha (N_B^2 - 1)}{3 \lambda^2} + \beta \right] \\
\bar{f}_C &= \frac{\alpha T_C^2 (3 + \lambda T_C)}{3} + \beta (1 + \lambda T_C)
\end{align*}
\]

To do the comparison, I again hold \( \bar{t}_i \) constant across the three rules and then determine which rule yields the smallest mean amount of fuel consumption. After some algebraic manipulations of equations (1.1)-(1.3) and (1.9)-(1.11), the equations given in (1.12) express \( \bar{f}_i \) in terms of \( \lambda, \bar{t}_i, \alpha, \) and \( \beta \) for \( i = A, B, C \).

\[
\begin{align*}
\bar{f}_A &= \lambda \bar{t}_A \left( \frac{\alpha \bar{t}_A^2}{3} + \beta \right) \\
\bar{f}_B &= \lambda \bar{t}_B \left( \frac{\alpha \bar{t}_B^2}{3} + \beta - \frac{\alpha}{3 \lambda^2} \right) \\
\bar{f}_C &= \lambda \bar{t}_C \left( \frac{\alpha \bar{t}_C^2}{3} + \beta - \frac{\alpha}{\lambda^2} + \frac{2 \alpha}{3 \lambda^2} \right)
\end{align*}
\]

Claim 1.3: If \( \bar{t}_i = \bar{t} \) for \( i = A, B, C \) then in terms of the smallest mean amount of fuel consumed per red signal, rule C is most preferred, rule A is least preferred and rule B falls somewhere in between.
Proof: The equations (1.12) can be expressed as
\[
\begin{align*}
\tilde{f}_A &= \lambda\left(\frac{\alpha^2}{3} + \beta\right) \\
\tilde{f}_B &= \lambda\left(\frac{\alpha^2}{3} + \beta - \frac{\alpha}{3\lambda^2}\right) \\
\tilde{f}_C &= \lambda\left(\frac{\alpha^2}{3} + \beta - \frac{\alpha}{\lambda^2} + \frac{2\alpha}{3\lambda^2}\right)
\end{align*}
\]  
(1.13)

Because \(\tilde{t}, \lambda, \alpha,\) and \(\beta\) are all positive, it is clear from equations (1.13) that rule A produces the largest mean amount of fuel consumed per red signal implying that it is the least preferred of the rules. Now it remains to be shown that \(\tilde{f}_B > \tilde{f}_C\):
\[
\tilde{f}_B = \lambda\left(\frac{\alpha^2}{3} + \beta - \frac{\alpha}{3\lambda^2}\right) > \lambda\left(\frac{\alpha^2}{3} + \beta - \frac{\alpha}{\lambda^2}\right) + \frac{2\alpha}{3\lambda^2} = \tilde{f}_C
\]

\[
\begin{align*}
\frac{\alpha}{3\lambda^2} &> \frac{\alpha}{\lambda^2} + \frac{2\alpha}{3\lambda^2} \\
\frac{2\alpha}{\lambda} &< 2\tilde{t} \\
\frac{1}{\lambda} &< \tilde{t}
\end{align*}
\]

But under rule C, \(\tilde{t} = \frac{1}{\lambda} + T_C\) which implies,
\[
\begin{align*}
\frac{1}{\lambda} &< \frac{1}{\lambda} + T_C \\
0 &< T_C
\end{align*}
\]

\(\square\)

**Pollutant Emissions as a Performance Measure**

An air pollution function in terms of \(s\) and \(w\) could be \(p(s,w) = \eta* w^2 + s + \theta*s^2\) where \(\eta = \) conversion coefficient in grams per (vehicle-minutes)^2 and \(\theta = \) conversion factor is grams per (stops)^2. The coefficients \(\eta\) and \(\theta\) depend upon the pollutant being modelled (e.g. carbon monoxide, hydrocarbons, oxides of nitrogen), the composition of traffic, approaches, grades, etc. The needed quantities are derived in Section A.3 and are presented below in equations (1.14)-(1.16).
\[
\begin{align*}
\bar{p}_A &= \lambda T_A \left[ T_A \left( \frac{\eta T_A}{3} + \lambda \theta \right) + \theta \right] \\
\bar{p}_B &= N_B \left[ N_B \left( \frac{\eta N_B}{3\lambda^2} + \theta \right) - \frac{\eta}{3\lambda^2} \right] \\
\bar{p}_C &= \frac{\eta T_C^2 (3 + \lambda T_C)}{3} + \theta \lambda^2 T_C^2 + 3\theta \lambda T_C + \theta
\end{align*}
\]  
(1.14)  
(1.15)  
(1.16)
The next step is to express $\bar{\rho}_i$, $i = A, B, C$, in terms of $\bar{t}_i$, $\eta$, $\theta$, and $\lambda$. Using equations (1.1)-(1.3) and (1.14)-(1.16) and performing some algebraic simplifications, the $\bar{\rho}_i$ ($i=A, B, C$) can be rewritten as

$$
\begin{align*}
\bar{\rho}_A &= \lambda \bar{t}_A \left[ \frac{\eta \bar{t}_A}{3} + \lambda \theta \right] + \theta,
\bar{\rho}_B &= \lambda \bar{t}_B \left[ \frac{\eta \bar{t}_B}{3} + \lambda \theta \right] - \frac{\eta}{3\lambda^2},
\bar{\rho}_C &= \lambda \bar{t}_C \left[ \frac{\eta \bar{t}_C}{3} + \lambda \theta \right] - \frac{\eta}{\lambda^2} + \theta - \frac{2\eta}{3\lambda^2}.
\end{align*}
\right)
(1.17)
$$

Comparisons are made using equations (1.17) and holding $\bar{t}_i$ constant ($i=A, B, C$).

**Claim 1.4:** If $\bar{t}_i = \bar{t}$ for $i = A, B, C$ then in terms of the smallest mean amount of pollutant emissions per red signal, rule A is always least preferred. If $3\theta \lambda^2 > 2\eta$ then rule B is most preferred and rule C falls somewhere in between. Otherwise, rule C is most preferred and rule B falls somewhere in between.

**Proof:** Equations (1.17) can be rewritten as

$$
\begin{align*}
\bar{\rho}_A &= \lambda \bar{t}_A \left[ \frac{\eta \bar{t}_A}{3} + \lambda \theta \right] + \theta,
\bar{\rho}_B &= \lambda \bar{t}_B \left[ \frac{\eta \bar{t}_B}{3} + \lambda \theta \right] - \frac{\eta}{3\lambda^2},
\bar{\rho}_C &= \lambda \bar{t}_C \left[ \frac{\eta \bar{t}_C}{3} + \lambda \theta \right] - \frac{\eta}{\lambda^2} + \theta - \frac{2\eta}{3\lambda^2}.
\end{align*}
\right)
(1.18)
$$

Because $\bar{t}_i$, $\lambda$, $\eta$, and $\theta$ are all positive, it is obvious from equations (1.18) that rule A produces the largest value for the expected amount of pollutants emitted per red cycle. This implies that rule A is the least preferred rule.

Now assume $3\theta \lambda^2 > 2\eta$. It must be shown that $\bar{\rho}_B > \bar{\rho}_C$:

$$
\bar{\rho}_B = \lambda \bar{t}_B \left[ \frac{\eta \bar{t}_B}{3} + \lambda \theta + \frac{\eta}{3\lambda^2} \right] < \lambda \bar{t}_B \left[ \frac{\eta \bar{t}_B}{3} + \lambda \theta - \frac{\eta}{\lambda^2} + \theta + \frac{2\eta}{3\lambda^2} \right] - \frac{\eta}{\lambda^2} + \theta + \frac{2\eta}{3\lambda^2} = \bar{\rho}_C
$$

$$
- \frac{\eta}{3\lambda} < - \frac{\eta}{\lambda} + \theta \lambda \bar{t} - \theta + \frac{2\eta}{3\lambda^2}
$$

$$
\bar{t} (3 \theta \lambda^2 - 2\eta) > \frac{3\theta \lambda^2 \bar{t} - 2\eta}{\lambda}
$$

because $3\theta \lambda^2 > 2\eta$. 

Under rule C, \( \tilde{T} = \frac{1}{\lambda} + T_c \) which implies,
\[
\frac{1}{\lambda} < \frac{1}{\lambda} + T_c
\]
\[
0 < T_c
\]
Similarly, if \( 3\theta \lambda^2 \leq 2\eta \), it can be shown that \( \bar{\rho}_b \leq \bar{\rho}_c \).

To summarize, in terms of the delay performance measure, rule B is preferred to rule C which is preferred to rule A. In terms of the fuel consumption measure, rule C is preferred to rule B which is preferred to rule A. Lastly, in terms of the air pollution measure, rule B is preferred to rule C only if \( 3\theta \lambda^2 > 2\eta \) and both rule B and rule C are always preferred to rule A. If \( 3\theta \lambda^2 \leq 2\eta \), then rule C is preferred to rule B.

1.2.5 Discussion of the Results and Issues of Interest

It has been demonstrated above that one decision rule does not optimize the three vehicular performance measures simultaneously. However, it is shown that rule A performs the worst with regard to all three measures. Thus, costs aside, rule A can be eliminated from consideration. Rule B performs best for the delay measure, rule C performs best for the fuel consumption measure and the rule which performs best with respect to air pollution depends on the coefficients of the function. How should one choose between rules B and C?

Though my example is simplistic, an analogous situation arises in practice for signalized vehicular intersections. It is possible to obtain improvements in all performance measures with a more real-time traffic responsive technology (both rules B and C) but among the more responsive techniques, there may not be one which performs best with regard to all the measures of interest. The real situation is even worse because for a given signal control technology, there are tradeoffs among the performance measures. Choosing timing parameters for the signal control which minimizes delay usually results in a timing plan which does not minimize fuel consumption.

How should one select a signal technology when the performance measures conflict and once a control technique has been selected, which performance measure should be optimized? An easy answer does not exist. To make a good choice among alternative technologies and timing parameters within a given technology, it seems important to understand the tradeoffs among the key measures. A study conducted in Gainesville [Courage and Parapar 1975] demonstrates it is possible to lower fuel consumption by
44 gallons per hour for a specific signal network if the signals are timed for minimum fuel consumption as a primary objective. Unfortunately, the more fuel efficient timing causes an increase of approximately 42 vehicle-hours per hour in delay. If a compromise is made between the two plans, it is possible to realize a fuel savings of 35 gallons per hour at the expense of only 16 vehicle-hours per hour of increased delay. This illustrates that knowledge of the relationship between delay and fuel consumption is more valuable than knowledge of the minimum values for either measure.

Signal timing procedures of the past and present usually focus on minimizing vehicle delay. A few have included fuel consumption as a secondary objective. Very few have considered the effect of all three measures: delay, fuel consumption, and air pollution. Because of increasing concern about energy conservation and environmental issues, it seems important to explicitly consider fuel consumption and air pollution in addition to vehicular delay when choosing signal control technologies and when developing signal timing plans. As shown by Courage and Parapar, there are considerable savings to be obtained from developing an understanding of the tradeoffs among the three measures. In Chapter Five, I model these tradeoffs for a fixed-cycle signal by determining the Pareto optimal set of the delay, fuel consumption and air pollution functions.

In my example and the ensuing discussion, costs and savings have been ignored. Though an operating rule may be deemed best with respect to some performance measure, this does not guarantee it is necessarily cost-effective. Cost-effectiveness adds another dimension to the selection of a control system. Consider the pedestrian crossing example. To develop and install the necessary technology for rule B would be extremely expensive. It would probably require years of operation to recoup the development and installation costs. Because future savings are valued less than immediate costs, it is conceivable that this decision rule is probably not cost-effective. Even rule C may not be cost-effective. From Figure 1.1, the mean pedestrian delay is essentially the same for rules A and C. Only at lower arrival rates does rule C outperform rule A but at these rates, few people benefit. The savings and the cost of rule C require further scrutiny to determine whether it is cost-effective.

Costs and benefits of various signal timing procedures are explored later. Using documented costs and savings, and the results of a national survey, rough estimates are developed for overall benefits realizable from a nationwide signal upgradation program.
CHAPTER ONE. Introduction

The next section describes the major objectives of my thesis and also includes a brief outline of the organizational structure of the later material.

SECTION 1.3 Statement of Objectives and Thesis Organization

1.3.1 Statement of Objectives

Recent events of the world have caused a renewed awareness of the importance of energy conservation and environmentally sound policies. One event was the twentieth anniversary of the first Earth Day. This event focussed attention on the global impact pollution is having on our world. It is no longer possible to ignore environmental issues without devastating future effects. Problems such as the depletion of the ozone layer and the accumulation of greenhouse gasses need to be addressed and resolved. There seems to be a nationwide awakening of an environmental consciousness that was dormant during the Reagan years. The Bush administration has provided some support of environmental policies (but not enough to earn President Bush his self given environmental president label). In the United States and throughout the western industrial nations there is much evidence of a growing grassroots “green” movement.

The other recent event is the Iraqi invasion of Kuwait and the subsequent U.S. involvement in a Persian Gulf war. This event evoked memories, for us old enough to remember, of the long gas lines and high gas prices resulting from the two oil crises of the 1970s. As the memories of long gas lines have receded into the past, the U.S. government and the U.S. public have relaxed their conservation policies. Energy-saving technologies and alternative fuel research and development projects have been reduced. Given the political instability of the oil producing countries of the Middle East, which control over 65% of the world oil reserves [API 1991], and the knowledge that oil reserves are not inexhaustible, it seems important to resurrect these activities.

In light of current societal concerns outlined above, the specific issues surrounding signal timing (Section 1.2), the professional and academic interest in signal timing (Section 1.1), and the demonstrated gains resulting from the implementation of improved timing plans (Section 8.6.1), I choose to explore the cost effectiveness of smart traffic signals. These are signals which are made intelligent by sensors reporting real-time traffic conditions at and near each signalized intersection. My
primary goal is to determine whether or not a national investment in smart traffic signals provides a significant return on investment when measured by three performance measures: (1) the time delay experienced by the occupant(s) of a vehicle, (2) the fuel consumption rate associated with the operation of the intersection, and (3) the emissions rates of the different air pollutants associated with the operation of the intersection.

Some specific objectives associated with my primary goal are briefly stated below.

- Explore the Pareto optimal set of the performance measures for fixed-cycle control in order to ascertain the magnitude of the tradeoffs among the measures within a given control technology.

- Ascertain the values of the performance measures for a hypothetical system operating under perfect information in order to assess the value of perfect information and whether significant tradeoffs still exist between the performance measures.

- Use the results of my national survey of currently used signal control techniques to estimate the current level of the performance measures.

- Determine the costs and benefits associated with the implementation of two real-time adaptive control strategies.

The next section sketches the organization and content of my thesis.

1.3.2 Thesis Organization and Content

A review of the existing signal timing literature is presented in Chapter Two. This review focuses on modelling techniques that have been used in the past, different types of signal control methodologies that have been developed, and signal control methodologies that are currently used by practicing traffic engineers. Because of the immense amount of existing literature this review is by no means exhaustive.

Chapter Three (and Appendix B) contains a summary of the results of a national survey I conducted of practicing urban traffic engineers. A questionnaire was mailed to over 300 communities in order to determine the types of signal control in use and the techniques used to develop timing plans for the existing signals.

The performance measures of interest are presented in Chapter Four. This includes the models underlying my fuel consumption measure and pollution measure. In addition to selecting the functional form of these measures, I estimate the values of the coefficients found in the models to reflect the current vehicle fleet.
In Chapter Five, I determine and analyze the Pareto optimal set of the performance measures for fixed-cycle control. The analysis is performed for a lightly and a moderately heavy trafficked intersection.

Values of the performance measures under perfect information are presented for an isolated intersection in Chapter Six. These include values obtained using a discrete-time dynamic programming approach, OPAC-1, and those obtained using a new non-dynamic programming approach, F-COD, based on a more continuous representation of time.

Chapter Seven reviews two real-time adaptive control algorithms. The first is merely a variation of conventional vehicle-actuated control. The second (ROPAC), on the other hand, is based upon dynamic programming and explicitly attempts to optimize a specific performance measure. Slight modifications have been made to the ROPAC algorithm so it can be used with any of the key measures.

A cost-effectiveness analysis of the two smart signal policies is presented in Chapter Eight. This includes both a monetary analysis and comparisons to other nationally adopted policies designed to improve the specified measures.

Finally, Chapter Nine summarizes my work and conclusions, and outlines some future research directions.
Chapter Two

Literature Review

Traffic flow into a signalized intersection is extremely unpredictable over short time intervals. The unpredictability can give rise to different optimal control strategies designed to minimize total delay over periods of equal duration. Generally, operations research analysts associate variability within systems with ill effects. In moderately saturated queueing systems, significant variability in arrival rates and/or service rates can create periods of long queues and excessive waiting times from which the system is slow to recover. However, with respect to signal timing, the intrinsic variability in traffic flows is not necessarily detrimental: it is possible to take advantage of gaps in traffic flow on one street to service demand on another street without incurring any delay costs. On the other hand, if traffic flow was deterministic with headways smaller than the time required to service a vehicle, it is not possible to service one street without incurring delay for the motorists on the other street. Signal timing techniques have evolved over time through attempts to advantageously use the intrinsic variability in the prevailing traffic conditions. The premise being that increased traffic responsiveness leads to improved traffic flow. Hence, each new methodology seeks to be more traffic responsive by identifying better gaps within a traffic stream in real-time.

A traffic signal can operate in isolation or it can be linked with other signals, along an artery or in a network structure, and operated in coordination with them. Timing procedures exist for both isolated signals and systems of coordinated signals. Signal
timing methods for an isolated intersection usually ignore the effects adjacent signalized intersections may have on incoming traffic flow. On the other hand, coordinated timing procedures must account for interactions between adjacent signals because the traffic leaving one intersection becomes incoming traffic for another intersection. Appropriately modelling the effects of signal interactions on traffic flow makes timing procedures more complex for networks than for isolated intersections, which is itself a difficult problem.

It is the purpose of this chapter to highlight some of the contributions made to signal timing methodology over the last four decades. My material is drawn mostly from operations research, transportation science, and traffic engineering literature and emphasizes the methods most commonly employed by traffic engineers today (see the survey results in Chapter Three). Section 2.1 contains some definitions of commonly used traffic engineering terms. The following section begins the review by summarizing timing methodologies devised for isolated intersections. A review of existing timing techniques for signal networks can be found in Section 2.3. Finally, the last section summarizes the important observations and conclusions which can be drawn from the past experience.

SECTION 2.1 Definitions

Any discussion of signal timing requires the use of common traffic engineering terms to depict the various aspects of signal timing. This section contains definitions of some commonly used terms. The material in this chapter is largely taken from Fundamentals of Traffic Engineering [Homburger and Kell 1988] and Manual of Traffic Signal Design [Kell and Fullerton 1982].

First I provide basic definitions for some common terms used with respect to the parameters of signal timing plans.

- **Controller**: A device, which may or may not be under computer control, that controls the sequence and duration of indications displayed by traffic signals.

- **Cycle**: The time required for one complete sequence of signal indications (phases).

- **Phase**: That part of a signal cycle allocated to any combination of one or more traffic movements simultaneously receiving the right-of-way during one or more intervals.
• **Interval:** A discrete portion of the signal cycle during which the signal indications remain unchanged.

• **Offset:** The time difference (in seconds or in percent of the cycle length) between the start of the green indication at one intersection as related to the start of the green indication at another intersection or from system time base. The difference in offset between intersections along a street defines the speed at which traffic can travel without stopping. Offsets only make sense in the context of coordinated signal systems.

• **Split:** The percentage of a cycle length allocated to each of the various phases in a signal sequence.

• **System cycle:** A specific cycle length imposed throughout a system of coordinated signals covered by the timing plan.

Today, there are numerous types of signal control algorithms. They can be grouped into two broad categories: fixed-cycle and traffic-actuated. Traffic-actuated control uses detectors to assign the right-of-way on the basis of traffic demand. When actual values of the parameters of a specific algorithm are determined for a particular intersection, the result is called a signal timing plan. Below are descriptions of the different types of isolated intersection control techniques.

• **Fixed-cycle control** (also called fixed-time control): This control method assigns the right-of-way at an intersection according to a predetermined schedule. The time interval for each signal indication in the cycle is fixed length and is predetermined on the basis of historic traffic patterns.

• **Semi-actuated control:** This type of control is usually used at intersections where a major street having relatively steady flow is crossed by a minor street with low volumes. Operating characteristics include:
  -- Detectors on minor approaches only.
  -- Major phase receives a minimum green interval, to permit vehicles stopped between the detection point and the stop bar to clear the intersection.
  -- Major phase green extends indefinitely until interrupted by minor phase actuation.
  -- Minor phase receives green after actuation providing major phase has completed minimum green interval.
  -- Minor phase has minimum initial green period, to permit vehicles stopped between the detection point and the stop bar to clear the intersection.
  -- Minor phase green may be extended by additional actuations until preset maximum is reached or a gap in actuations greater than the preset unit extension occurs.
  -- Memory features remember additional actuations if maximum has been reached on minor phase and will return green after major phase minimum interval.
  -- Yellow intervals are preset for all approaches.

• **Fully-actuated control:** This type of control is used at an intersection of streets or roads with relatively equal volumes and traffic flow is varying and sporadic. Operating characteristics include:
  -- Detectors on all approaches.
  -- Each phase has a preset initial interval which allows vehicles between the detector and the stop bar to clear the intersection.
CHAPTER TWO.  Literature Review

-- Green interval is extended by a preset unit extension for each actuation after the expiration of the initial interval provided a gap greater than the unit extension does not occur.
-- Green extension is limited by preset maximum limit.
-- Yellow intervals are preset for each phase.
-- Each phase has a recall switch.
  (1) When both recall switches are off, the green will remain on one phase when no demand is indicated on another phase.
  (2) When one recall switch is on, the green reverts to that phase at every opportunity.
  (3) When both recall switches are on, the controller will cycle on a fixed-cycle basis in the absence of demand on either phase.

• Volume-density control: Volume density control is designed for use at intersections of major traffic flows that have considerable unpredictable fluctuations. To operate efficiently, this type of control needs to receive traffic information early enough to react to existing conditions. Therefore, it is essential that detectors be placed far in advance of the intersection. Operating characteristics include:
  -- Detectors on all approaches.
  -- Each phase has a variable initial assured green time consisting of a minimum initial green interval plus added green time to accommodate additional vehicles that have arrived on the red.
  -- Passage time is the extended green time created by each additional actuation after the assured green time has elapsed. This time is usually set to be that required to travel from the detector to the stop line. There is a preset time to reduce the passage time to a minimum gap time, and a preset minimum gap.
  -- The maximum green extension is also predetermined and set on the controller. This feature is seldom operated because of the passage time reduction feature.
  -- Yellow intervals are preset for each phase.
  -- Each phase has a recall switch that operates in the same manner as described for full-actuated control.

• Traffic responsive system: A system in which a master controller determines which approach is awarded the right-of-way based on the real-time demands of traffic as sensed by vehicle detectors, also referred to as real-time adaptive control systems.

SECTION 2.2  Isolated Intersections

In my review of existing isolated signal timing methods, I use three classifications to roughly group my findings: fixed-cycle control, vehicle-actuated control and traffic-responsive control. These are listed in order of increasing traffic responsiveness.

The general objectives of traffic control are to improve safety and minimize driver inconvenience. Though this is a noble intent, it must be reduced to quantifiable objectives to be of any practical use. Ideally, one would like to represent the traffic control problem as an optimization problem with a stated objective function, the performance criterion. Obviously, the optimum value for this function depends on the controllable parameters of the system. Models constructed in the 1960s and before
were analytical optimization problems. Significant theoretical contributions were made by mathematicians and operations researchers. Exact optimization has been obtained only for a few simplistic models with special traffic inputs.

With the advent of computers and more sophisticated controllers, the number of possible signal timing options has exploded. In addition, the signal timing options have also become more complex. To handle the increased complexity, research efforts have focussed on computer-based optimization models and simulations. Today, both analytical models and computer-based models are used to determine isolated intersection timing plans. These models are designed to determine optimal values for the parameters of a specific control technique with respect to some performance criterion. Some of the more commonly used measures include average delay per vehicle, maximum individual delay, average number of stops, and throughput of the intersection (a capacity measure).

2.2.1 Fixed-Cycle Control

Fixed-cycle control is a method which assigns the right-of-way at an intersection according to a predetermined schedule. The sequence of phases and duration of each phase is determined solely on the basis of historical data or a hypothetical arrival process. This type of control does not employ any detection devices. When developing a timing plan for a fixed-cycle signal, the controllable parameters which need to be specified are the cycle length, the splits, and the phase sequence. This naturally decomposes the creation of a timing plan into two separate problems: determining the phase order and determining the optimal length of the green intervals for each approach. First, I briefly discuss some literature relevant to the determination of the phase order and then devote the remainder of the subsection to literature relevant to the determination of optimal green durations.

When streams of traffic movements are identified and grouped into mutually compatible sets which are to receive the right-of-way together, a phase order must be determined. The basic problem is to find the best order in which to award the green to each set of compatible approaches. Given a cycle time, Guberinic, Reljic, and Senborn [1983] derived a method which determines the values of the elements of a fixed-cycle plan (the phase sequence and splits) such that an intersection's throughput is maximized. The optimization problem is stated as finding the best closed path through the graph of possible sequences and is formulated as a mixed-integer program which is solved using a branch-and-bound technique. Cantarella and Improta [1988]
propose a computer-based model in which the user can select either intersection capacity maximization or cycle time minimization as the objective function. The authors use a binary-mixed-integer linear program to describe a fixed-cycle signal assuming knowledge of the flow parameters and simultaneous crossing incompatibilities between traffic streams. Their solution method is based on graph theory. More recently, a microcomputer program denoted SIGSIGN (SIGnal deSIGN) was developed by Silcock and Sang [1990]. Their program includes phase-based optimization procedures to determine signal settings that either maximize intersection capacity, minimize cycle time, or minimize delay at an intersection. Users can specify the objective function for which they would like optimum fixed-cycle settings.

Lastly, Gallivan and Heydecker [1988] discuss the incompatibilities between combinatorial methods which produce all possible control structures and convex programming techniques which optimize a given control structure. They develop a procedure which avoids the difficulties of combining the two approaches by reformulating the problem so a control structure generated by a combinatorial method can be optimized directly.

There is a great deal of literature devoted to the analysis of signal settings which minimize the expected vehicle delay at a fixed-cycle traffic signal. Because of the vast amount of literature, I group them as analytical models and computer models.

**Analytical Models**

To determine optimal settings, it is first necessary to characterize the delay a vehicle experiences as a result of a specified fixed-cycle signal setting. Most researchers assume constant departure rates during a green phase and make various assumptions about the vehicle arrival process. Clayton [1941] and Wardrop [1952] studied the delay problem assuming constant arrival rates, the simplest model. The case when arrival inputs are Poisson was investigated by Webster [1958], McNeil and Weiss [1974], and Ohno and Mine [1973b]. Others, Winsten [1953], and Beckman, McGuire and Winsten [1956], assume binomial arrivals. Delay formulas for more general arrival processes have been developed by Darroch [1964], McNeil [1968], and Siskind [1970].

For a fixed-cycle model to be accurate for heavy traffic flows, it must account for overflow at the signal. Overflow occurs when the vehicular queue on an approach does not dissipate during the subsequent green phase, implying there is a queue, called the residual queue, on the approach at the beginning of the red phase. Haight
[1959] studied the distribution of the number of vehicles in the residual queue, assuming Poisson arrivals and constant departure times. His results are of limited use because they are conditional on the number of vehicles in the previous residual queue.

Newell [1960] investigated a bulk service queueing model to account for residual queues. His queueing model hypothesizes constant service times (the time it takes a vehicle to clear the intersection during a green light), a batch size $m$ or less (the maximum number of queued vehicles that can clear the intersection), and an arrival process of general form. His solution for the expected number of vehicles in the residual queue is in terms of the complex roots of certain transcendental equations. An exact, though complicated, series expansion for the expected number of vehicles in the residual queue, assuming simple Poisson arrivals, was obtained by Kleinecke [1964], McNeil [1968], and McNeil and Weiss [1974].

Because of the difficulty of obtaining simple, easily computable expressions for the expected number of vehicles in a residual queue, many researchers have developed approximations and bounds for this quantity [Miller 1963, McNeil 1968]. The difficulties involved in an exact analysis for more general arrival and departure processes prompted Newell [Newell 1965] to devise approximations, based on the representation of the traffic as a continuous fluid with stochastic properties. Using a law of large numbers, Newell obtained results insensitive to the detailed structure of the arrival and departure processes.

Though many individuals have researched the expected delay at an isolated fixed-cycle signal, few have attempted the optimization problem. Miller [1963] may be the only author who has obtained analytical expressions for optimum signal settings which minimize average vehicle delay. Others have tackled the optimization problem numerically. Of the numerical approaches, the formulas of particular interest are those developed by Webster [1958] and Newell [1965]. These formulas are the ones most often used in practice because of their computational simplicity. Webster derived his formulas from simulations of Poisson traffic flow into an intersection controlled by a fixed-cycle signal. He was able to fit a curve to his simulation results describing the average delay per vehicle attributable to the operation of the signal. From his curve, he obtained signal settings that perform well in practice.

The three different delay expressions and their corresponding "optimum" signal settings, published by Webster, Miller, and Newell, are compared by Allsop [1972]
and Hutchinson [1972]. Allsop reviews numerous theoretical analyses of the delay incurred under fixed-cycle control. He concludes that the derivations and formulas of Webster, Miller, and Newell are the most important and the most useful in practice. Most other analyses are merely of theoretical interest. Comparisons of the numerical results of Webster's, Miller's, and Newell's expressions are performed by Hutchinson. His comparisons are done over a wide range of arrivals rates, saturation flows, cycle lengths, green splits, and variance-to-mean ratio of the number of arrivals per cycle.

The aforementioned studies involve non-time varying forms of fixed-cycle control. Though the following fixed-cycle algorithms are time-varying, the dynamics of the control policy are not responsive to the dynamics of the traffic flow because there is no real-time traffic information. The traffic flow process is represented by the expected flow rate, a probability distribution, or a smooth function of demand versus time. Though the models do not take advantage of time-variant features of individual vehicle arrival times, they do possess aspects which make them more responsive than non-time varying fixed-control algorithms.

In 1965, Gazis and Potts [1965] constructed an optimal control strategy to minimize rush-hour delays. The authors present an optimal control for an intersection which becomes oversaturated over a given finite length of time. Two fixed-cycle timing plans are elements of the strategy. At a calculated switch-over point, control switches from one fixed cycle plan to the other. Gazis [1964] extends this procedure to a system of two oversaturated intersections. The same modelling approach has been studied by Michalopoulos and Stephanopoulos [1977, 1978]. An aspect investigated by these authors is the effect of queue length constraints on the optimal control strategy. Numerical solutions are obtainable only if the intersection demand is predictable over the entire control period. Guardabassi, Locatelli, and Papageorgiou [1984] show that if one relaxes the condition of simultaneous dissolution of all queues which was initially imposed by Gazis and Potts, an optimal control solution is always attainable in cases of more than two traffic streams. If the condition is not relaxed, the authors show that derivation of practical results becomes virtually impossible for more than two traffic streams.

A time-varying fixed-cycle control strategy for undersaturated intersections was developed by Dunne and Potts [1964]. Assuming constant arrival rates, their linear control algorithm guarantees the system, starting from any initial state, will eventually reach a limit cycle for which the average equilibrium delay per cycle is a minimum.
Grafton and Newell [1967] show that the control policy, assuming constant arrival and departure rates, derived by Dunne and Potts, minimizes delay for most traffic conditions. When the departure rates for the two traffic streams are unequal and the initial queues are very large or very small, Grafton and Newell demonstrate that the optimal policy will involve one or more policy modifications. In 1967, Dunne [1967] studied the control strategy assuming binomial arrivals and constant departure rates. Later, Newell [1969] compared a variation of Dunne and Pott's time-varying fixed-cycle control to conventional fixed-cycle control. He found that the average delay per car using the time-varying algorithm is less than conventional fixed-cycle control by a factor of about three. Newell and Osuna [1969] conclude that the gains observed over conventional fixed-cycle control for two approaches does not necessarily extend to four approaches.

While these control policies are time-varying and offer potential savings over conventional fixed-cycle control strategies, none have been incorporated into a real system [Michalopoulos and Stephanopoulos 1978]. This is partly due to the complex computational requirements of the time-varying strategies. However, the major drawbacks of these control policies is their oversimplified assumptions, e.g. constant arrival rates, and the limited traffic situations in which they are designed to function well. For these reasons, practicing traffic engineers usually rely on empirical considerations when timing fixed-cycle signals as is affirmed by the survey results in Chapter Three. During times of oversaturation, e.g. rush-hours, they implement plans that apportion green times in proportion to the queue storage areas and for times of undersaturation, they use Webster's formula or some variation of it [Michalopoulos and Stephanopoulos 1978].

Computer Models

From the early 1970s through the present, efforts have been directed towards developing computer packages designed to assist the traffic engineer in determining signal timing plans for fixed-cycle control. Currently, there exist several computer based models and packages for determining fixed-cycle signal settings which minimize delay or maximize intersection capacity. The optimization element of these programs is either based on simulation or an underlying analytical model.

Saka, Anandalingam and Garber [1986] determine the optimum cycle and green splits that minimize total average delay at an intersection. They formulate fixed-cycle control as a stochastic inventory problem and solve it using a simulation optimization
technique. Simulation models have also been developed at the University of Bradford [Salter and Tadayon 1987]. The authors use their models to simulate various traffic flow conditions. The simulation results are then analyzed to estimate cycle times which produce minimum average delay.

SIGSET (SIGnal SETtings) was the first commercially available computer package designed to compute optimal fixed-cycle signal settings without using simulation. The green times, and if required, the cycle times are calculated so as to minimize, subject to constraints that the engineer may wish to impose, the estimated delay per unit time to all traffic passing through the intersection [Allsop 1971]. A similar computer package is the Signal Operations Analysis Package, SOAP [Nemeth and Mekemson 1982, Morales 1989]. This package is capable of developing plans for six design periods in a single run. It can also analyze 15-minute data for up to 48 continuous time periods and determine which timing plan is best suited for each 15-minute period.

A computer package which is designed to maximize intersection capacity is SIGnal CAPacity or SIGCAP [Allsop 1976]. SIGCAP requires as input, the phase sequence and the intersection's traffic pattern. It computes the cycle length and splits which maximize capacity. The model is based upon the capacity model developed by Webster and Cobbe [1966].

Another objective function, not yet mentioned, is the degree of intersection saturation. Ohno and Mine [1973a] devised a procedure which establishes fixed-cycle settings to minimize intersection saturation. This is a useful control technique for intersections which quickly become oversaturated.

Lastly, a program of particular interest to me, because of its list of objective functions, is denoted TRAFSIG and was developed by Reljic [1988]. This program, when given a phase sequence, calculates the cycle time and green splits so as to minimize the selected optimization criterion. The possible criteria from which to choose include total delay, total number of stops, a linear combination of total delay and total number of stops, total cost of losses, total fuel consumption, total person delay, and the sum of squares of the queue lengths. This program is one of the few methods to make fuel consumption an explicitly available objective for an isolated intersection.

Given all the methods that have been discussed, it is surprising to learn that most fixed-cycle control plans are derived manually using Webster's formula or some variation of it [Reljic 1988, Saka et al. 1986, Homburger and Kell 1988].
2.2.2 Vehicle-Actuated Control

Vehicle-actuated control is defined to be any method which uses traffic information relayed by vehicle detectors within the immediate vicinity of the intersection in order to identify gaps in the traffic stream. This implies that vehicle-actuated control tends to respond to traffic conditions of the immediate past rather than anticipating future traffic characteristics. Clearly, vehicle-actuated control at an isolated intersection performs at least as well as fixed-cycle control. In the worst case, steady demand on all approaches forces vehicle-actuated control to operate as fixed-cycle control because each approach is allotted its maximum green. At all other times, the vehicle-actuation feature allows demand for service on a red approach to be satisfied earlier than under fixed-cycle control. It is in these situations that vehicle-actuated control outperforms fixed-cycle control.

In the case of the fixed-cycle signal it was possible to determine the expected delay, and from that, the optimum signal settings, by considering each traffic stream separately because there was no interaction between streams. This is no longer possible for the vehicle-actuated signal. The analysis becomes increasingly complicated as the number of interacting streams increases.

An early analysis of semi-actuated control was performed by Garwood [1940]. He assumes the main traffic stream is awarded the green until there is a subsidiary arrival. The signal changes to allow the subsidiary vehicle(s) to cross either at a time $T$ after the subsidiary arrival or when there is a gap of length $\tau$ minutes (or greater) in the main stream, whichever occurs first. When the signal does change, the entire subsidiary queue is serviced and then the signal reverts back to green for the main stream. Garwood uses elegant combinatorial arguments in his analysis though it is also possible to obtain his results through methods designed to calculate delays incurred by vehicles at a stop sign.

More recently, Luh and Lee [1991] developed a model which can be used to directly determine the average delay and stop probability under semi-actuated control at an isolated intersection with low traffic volumes.

Because of the complexity of the problem, early models analyzed the simplest situation, namely, an intersection of two one-way streets. Tanner [1953] was the first to consider such a model. In his study, he assumes that arrivals are Poisson and the departure times and lost times are constant. To simplify the calculations even more, he assumes no upper limit on the amount of green time allotted to any approach.
Using simulation, Grace, Morris and Pak-Poy [1964] investigated a model similar to Tanner's. They also assume Poisson arrivals and no limit on the green time but they allow variable departure rates and lost times. The authors found continuous approximations for cycle times and gap extension units which minimize the total rate of delay at the intersection. As a follow-up of the work of Grace et al., Darroch et al. [1964] derive formulas for the moments of cycle times, the average wait per vehicle, and the optimal gap extension time through mathematical methods. An analysis similar to Darroch et al. is performed by McNeil and Weiss but assuming constant departure rates and lost times [McNeil and Weiss 1974]. One drawback of the model derived by Darroch et al. is the Poisson arrival assumption though it is possible to extend their analysis to a more general arrival process [McNeil and Weiss 1974].

Because of the intrinsic complexity of vehicle-actuated control, there have not been many analytical models developed for more than two traffic streams. More recent results have included computer based algorithms. These algorithms use the information obtained from detectors and a decision criterion to decide whether or not to extend a green period. Examples of three such models are described next.

Camus, D'Amore and Ukovich [1986] propose a hierarchical multilayer structure utilizing vehicle detectors. One layer handles the vehicle actuations which occur on approaches currently receiving a green signal. These are called green-extension requests. Actuations on approaches with a red signal, insertion requests, are routed to a different layer. A request manager processes the requests that have been routed through the other two layers. Using a decision function, the request manager determines the status of the signal for the next interval: either extend the current green or switch. This algorithm has been compared to conventional vehicle actuation in simulation runs and has performed consistently better in terms of vehicle delay.

MOVA (Modernized Optimized Vehicle-Actuation) was developed by Vincent and Young [1985, 1986]. This strategy utilizes two vehicle detectors per approach, one at 40 meters and the other at 100 meters before the stop line. Initially, each approach is awarded sufficient green time to discharge vehicles queued back to the 40 meter detector. After the initial green, the time intervals between detections are examined to ascertain when a reduction in saturation flow occurs. This information is used in an optimization process which compares the value of extending the green to the disbenefit experiences by the vehicles stopped at the red. A goal of the MOVA strategy is to avoid the tendency of conventional vehicle-actuated control which
unnecessarily extends greens when traffic is flowing considerable less than saturation. Results from field tests indicate that MOVA produces less delay than vehicle-actuated control during both oversaturated and undersaturated flows conditions.

A different approach from the above two methods is taken by Habib and Derzi [1988]. The authors constructed an algorithm which utilizes information from one vehicle detector per approach. Using an initial green interval of seven seconds and a maximum green interval of 50 seconds, a decision is made, each second within this range, of whether or not to extend the current green. This decision is based upon the number of vehicles in each queue subject to maximum queue length constraints. The objective is to minimize the delay per vehicle.

An isolated traffic light simulator, a tool which I use for obtaining some of my later results, was developed by Wu [1990]. Using her simulator, Wu analyzed the effectiveness of various vehicle-actuated controllers in terms of average vehicle delay, fuel consumption, and pollutant emissions. She derives optimal control strategies for vehicle-actuated controllers using one, two and three detectors per approach. Data collected from numerous simulation runs indicates that an actuated controller using two detectors per intersection is the most cost-effective.

Again, it is surprising to learn that few signals are actually timed by the methods and models described in this section. The methodology most often used is similar to that described in a book published by the Institute of Transportation Engineers (ITE) [Kell and Fullerton 1982]. Practicing traffic engineers commonly apply ad hoc procedures coupled with their experience to devise vehicle-actuated signal settings (Chapter Three).

2.2.3 Traffic Responsive Control

The emergence of inexpensive microprocessor technologies in the past decade has opened new horizons and opportunities for traffic-responsive control techniques. Almost as important as the time these system could potentially save motorists is the time these systems could save traffic engineers. It is feasible that these systems, if designed properly, could generate the best timing plans automatically. Before implementation, it would no longer be necessary for the engineers to perform labor-intensive flow studies which are currently necessary to produce quality timing schemes for fixed-cycle and vehicle-actuated signals.
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One of the first adaptive control algorithms was proposed by Miller [1965]. Miller's strategy is dynamically self-optimizing. A decision whether or not to extend the current green allocation is repeatedly made at very short fixed-time intervals (1-2 seconds). A function which essentially estimates the difference in vehicle-seconds of delay between the gain obtained by allowing extra vehicles to traverse the intersection as a result of an extension and the loss suffered by vehicles held by the red as a result of the extension is used. Miller's strategy has been shown to produce significant gains over fixed-cycle and vehicle-actuated control in comparison tests [Nip 1975, de la Breteque and Jezequel 1979]. However, this method has a very short optimization interval and does not ensure the overall optimality of the control strategy.

A variation of Miller's algorithm, Traffic Optimization Logic (TOL) was developed, implemented, and field tested by Bång and Nilsson [1976]. The evaluation criteria used by this authors has an hierarchical structure. The overall objective is to minimize the total community cost. Lower order criteria include minimizing the sum of travel time, vehicle operating costs, and environmental costs. TOL was compared to fixed-cycle control and vehicle-actuated control both in the field and through simulation. On average, TOL realized a 25 percent reduction in vehicle delay and was demonstrated to be cost-effective in terms of fuel savings alone.

There are two adaptive control algorithms for isolated intersections which are modifications of an algorithm originally designed for networks. One such algorithm, SCOOT, has been used to time two isolated intersections. It was concluded from the field tests that though SCOOT can be used to efficiently control an isolated intersection, its ability to react to transient flow conditions is hampered by built-in logic intended to ensure that traffic progresses smoothly through the regional network, an unnecessary constraint for an isolated signal [Carden and McDonald 1985]. The second such algorithm was developed in France and is called PRODYN [Lesort 1985, Henry and Farges 1989].

Because of its suitability to multistage decision making, dynamic programming lends itself well to real-time traffic responsive signal control. If the arrival times of all vehicles over a finite period were known, it would be possible to determine the control strategy which optimizes a given objective function using dynamic programming. This approach is also appealing because no special assumptions need to be made concerning the form of the arrival distribution. A glaring defect of this approach is that it is difficult to know exact arrival times for periods as short as a minute.
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Martin-Löf [1967] developed a model of a signalized intersection in which the development of the queues in time is described by a controlled Markov chain using the state of the light as a control variable. He then uses dynamic programming to compute a control strategy as a function of the queue lengths which minimizes the expected value of a chosen cost function.

Two other dynamic programming models have been constructed by Grafton and Newell [1967], and Robertson and Bretherton [1974] for optimal signal control. The Grafton and Newell model is continuous-time and minimizes an infinite-horizon total discounted delay function. A discrete-time version of the dynamic programming model, similar the continuous-time model, was derived by Robertson and Bretherton. They minimize the total delay aggregated over all intervals of a finite horizon.

Due to the large amount of computational time required by dynamic programming and the fact that it requires exact arrival data for significant periods of time to ensure optimality, dynamic programming models are unsuitable for on-line, real-time control. In response, Gartner [1983] formulated a simplified optimization procedure that is amenable to on-line implementation and yet provides results comparable in quality to those obtained via dynamic programming. His basic building block for this traffic responsive control technique is called OPAC-2, Optimization Policies for Adaptive Control. Gartner's algorithm, ROPAC, seeks to use a realistically feasible amount of information regarding future arrivals, and to limit the problem size so that the algorithm can be used on-line. Three field tests were performed comparing ROPAC to vehicle-actuated control. It was found that ROPAC produced a 4 to 16 percent reduction in average vehicle delay relative to vehicle-actuated signals [Gartner, Tarnoff and Andrews 1991].

As a final note, it should be emphasized that traffic responsive control is infeasible without advance information regarding vehicle arrivals. The amount of such information affects the efficiency of the responsive control. A simulation based study was performed by Lin [1985] to evaluate the sensitivity of traffic responsive control efficiency to the availability and utilization of advance arrival information; he basically determines the amount of information needed to sustain adaptive real-time control at an isolated intersection.
SECTION 2.3 Signal Networks

The main objective of a synchronization scheme for a network of signals is to allow drivers to progress through the network with minimum inconvenience. Generally, a measure of inconvenience is the delay motorists incur beyond the time required to traverse the network when no obstacles (red traffic signals, other vehicles, stop signs and so on) are encountered. Another measure is the number of stops a motorist makes as she travels through the network. Most modern control systems operate under the following principle: synchronize as many signals as possible to provide the opportunity of uninterrupted flow to as many cars as possible [Gazis 1974].

Synchronization schemes for signal systems can be divided into two categories: those designed for arterial signal networks and those designed for general signal networks. Arterial signal networks usually consist of signals which lie along the same thoroughfare. General signal networks are allowed any geometric configuration.

When reviewing the development of some of the computer-controlled strategies for general signal networks and the results of the major experiments that have been conducted, I use the nomenclature defined by the Urban Traffic Control System (UTCS) project to categorize the different strategies. There are four such categories:

1. First generation control (1-GC): strategies in which fixed-cycle settings, assuming all signals have a common cycle length, are calculated offline and stored in a library. These plans are usually implemented online according to time-of-day or prevailing traffic conditions.

2. Second-generation control (2-GC): strategies which compute and implement fixed-cycle settings, assuming all signals have a common cycle length, online and in real-time based on surveillance data and predicted volumes.

3. Third-generation control (3-GC): strategies conceived to compute and implement a fully responsive online control plan. 3-GC differs from 2-GC in that timing plans are updated more frequently and cycle lengths can vary among the signals as well as at the same signal.

4. Critical Intersection Control (CIC): strategies which allow the fine-tuning of splits, or splits and offsets, at intersections which become oversaturated quickly.

Critical intersection control (CIC) is achieved by dynamically adjusting the split among the competing streams while retaining the common cycle time and the main street offset. I do not explicitly discuss any of these techniques.
In addition to the three generations of control listed above, there is also 1.5-GC. This category consists of control strategies that are essentially 1-GC but which collect, online, the input data required for the off-line optimization program.

2.3.1 Arterial Signal Networks

Arterial signal synchronization is the oldest and most widely used synchronization scheme. Traffic signals tend to group vehicles into platoons with an uniform headway and for arterial roadways, it seems desirable to maintain continuous movement of these platoons through successive signals. This technique is known as a progression method. The synchronization problem becomes one of determining signal timings which maximize the width of continuous green bands, denoted bandwidths, in both directions along the artery at the expected speed of travel.

Surprisingly, a maximum bandwidth progression synchronization does not assure simultaneous minimization of the number of forced stops and delay. Bavarez and Newell [1967] investigated signal synchronization schemes for a one-way main street which intersects \( n \) one-way side streets. An unexpected conclusion reached by these authors is that given a cycle time and splits for each intersection, there is a choice of offsets that simultaneously minimizes delay and number of stops but does not necessarily produce the maximum bandwidth. This result is based upon an idealistic model and is a theoretical result.

Because the aforementioned findings are mostly of theoretical interest, the majority of arterial synchronization schemes are designed to produce a maximum bandwidth. Before the advent of computers, traffic engineers developed these timing plans manually using time-space diagrams. Today, engineers can choose from a large variety of computerized models. These include the IBM models [Brooks 1964], Bandwidth Maximization [Morgan and Little 1964, Little 1966], SIGART [Metropolitan Toronto 1965], SIGPROG [Bleyl 1967], and PASSER-II [Messer et al. 1973]. A more recent and also the most advanced of these types of models is MAXBAND [Little, Kelson and Gartner 1981, Cohen and Little 1982]. This model is an extension of work originally done by Little in 1966. It is capable of determining a global optimal solution and calculates cycle time, offsets, progression speeds, and order of left turn phases to maximize the weighted combination of the bandwidths in the two directions along the artery. It has also been extended for application to multi-arterial closed networks [Chang et al. 1988].
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The technique which is the most commonly used by traffic engineers is PASSER-II (Chapter Three). It was developed to advantageously use methods which minimize delay and those which maximize bandwidth, with bandwidth maximization as the primary objective. Because PASSER-II models platoon formation and movements, it reduces the amount of input data required. It produces the phase sequences, cycle lengths, green splits, and offsets for arterial synchronization designed to reduce delay, stops, and to some extent, fuel [Marsden, Chang and Derr 1987].

A basic limitation of the above models is that they do not consider the actual traffic volumes on each individual link in their optimization criterion. This implies that they cannot guarantee the most suitable progression scheme for different traffic flow patterns. Gartner et al. [Gartner, Assmann, Lasaga and Hou 1991] developed a program called MULTIBAND which generates a variable bandwidth progression that incorporates a systematic traffic-dependent criterion. The variable bandwidth progression is such that each directional road section can obtain an individually weighted bandwidth. The authors use mixed-integer linear programming to perform the optimization.

All maximum bandwidth designs possess a basic deficiency in that they work best under light traffic flow. When traffic flow is heavy, progression designs become clogged because vehicle queues block the path of cars through successive green signals [Gazis 1974]. Adjustments must be made to accommodate the existence of stopped queues. This has been done either directly or indirectly in the synchronization methods for general networks which are discussed next.

2.3.2 1-GC

Off-line signal synchronization algorithms for general network configurations effectively simulate the movement of vehicles through the network, evaluate a measure of motorist inconvenience, and search over the domain of feasible offsets for a "good" set of offsets. Differences between the existing synchronization schemes occur in the underlying traffic model utilized by the algorithm and how the search (optimization) is performed.

One approach to signal synchronization is the COMBINATION METHOD [Hillier 1966, Allsop 1968a, 1968b]. This method permits the use of an arbitrary link performance function and produces globally optimal offsets for given values of the cycle length and green splits. To accomplish this, it combines links in series and parallel to reduce the network into one with fewer links making the optimization procedure, which
uses the dynamic programming principle, simple and exact. Another variation of the
COMBINATION METHOD was devised by Improta and Sforza [1982]. These
authors relax assumptions made regarding the delay-offset function used by Hillier
and use a branch-and-backtrack solution procedure applicable to both condensable and
noncondensable networks.

Little's [1966] formulation of the arterial synchronization problem was generalized
by Gartner, Little and Gabbay [1975a, 1975b] to a more general network framework.
This model is later denoted as MITROP (Mixed-Integer Traffic Optimization
Program). Similar to the COMBINATION METHOD, it uses link performance
functions but unlike the COMBINATION METHOD, MITROP performs a
simultaneous global optimization over offsets, splits, and cycle lengths. The
optimization of the cycle length is the most useful addition of this method. More
recently, Koshi [1989] formally examines the effect cycle time has on signal
coordination schemes. He concludes the cycle time has a significant effect on delay
and the number of stops, and should not be ignored when developing a traffic signal
coordination scheme.

SIGOP (SIGnal OPtimization) was developed under the auspices of the U.S.
Bureau of Public Roads (now the FHWA) by the Traffic Research Corporation in 1966
[MacGowan and Lum 1975]. This program minimizes the sum of squares of the
differences between the ideal offsets and the optimized offset for each link. It is
possible to approximate the delay for a link versus offset by a parabolic curve. The
minimum of the parabola corresponds to the ideal offset for the link. Optimal offsets
are determined by a gradient minimization technique. SIGOP has features which
allow an user to obtain a cycle length\(^4\), splits for each intersection, and offsets which
produce near optimal signal settings.

Robertson [1969, 1975] developed TRANSYT at the Transport and Road Research
Laboratory (TRRL) in England. Whereas other link performance function
methodologies assume that the events on a link are independent of upstream offsets,
TRANSYT remembers previous traffic history. TRANSYT also contains a more
detailed representation of traffic flows than the other models. However, TRANSYT's
optimization converges to a local rather than a global set of optimal settings and so
the results are dependent on the initial settings. TRANSYT searches over one

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\(^4\) To determine the cycle length, SIGOP allows several cycle lengths to be evaluated in one run [Kaplan
and Powers 1973].
variable at a time when estimating optimal decision variables which in principle means all control variables (offsets, splits and cycle length) can be examined sequentially. In practice though, due to computational slowness, the search is primarily confined to the offsets. To choose the cycle time which performs best, successive runs of the program are made, with different cycle times, and from this set, the best time is identified [Wilbur 1985, Chard and Lines 1987].

TRANSYT-7F is a modification of TRANSYT that was developed for U.S. traffic engineers by the National Signal Timing Program (NSTOP) under the direction of the FHWA [Euler 1983, Wallace 1983]. There were four major revisions made to the seventh release of TRRL's TRANSYT model: (1) the signal timing inputs and outputs were changed to conform with U.S. convention, (2) a fuel consumption estimation procedure was added, (3) default values were provided for certain complex program control inputs to transfer the computational burden from the user, and (4) the outputs were revised to improve their usefulness and readability and to include a printout of a space-time diagram. These modifications have made the TRANSYT timing tool more accessible to U.S. traffic engineers and as a result its use has become wide-spread (Chapter Three).

Some additional 1-GC methods of some interest include FLEXIPROG [Gartner 1982], a dynamic programming based formulation by Gartner [1972b], SCAT [Sims 1979, Luk 1984] and FORCAST [Computran Systems Corporation 1985].

FLEXIPROG was part of a major signal timing experiment conducted in Glasgow. It is a vehicle-actuated flexible progressive system which allows phases to be skipped if there is no demand detected for them.

The work done by Gartner is interesting because he formulates the problem of finding optimal offsets for synchronization in terms of a dynamic programming optimization problem and develops an efficient network partitioning algorithm for solving the problem.

SCAT (Sydney Co-ordinated Adaptive Traffic) is a technique which divides a network into smaller subareas which share a common cycle time. The cycle time of a subarea may be updated based on the information about the subarea saturation level gathered through detectors. There is also an algorithm which determines the coordination between adjacent subareas. A simplistic optimization technique is used and the optimization is only performed for one-way arterial progressions.
Finally, FORCAST features include timing plans generated from data required to manually produce an answer, a solution approach based on the methodology used by most traffic engineers, the ability to generate time-space plots, and integration capabilities with other software packages.

2.3.3 1.5-GC

This is software which automates the traffic signal timing plan update process. Rather than requiring engineers to collect all the necessary data, the system utilizes volume counts derived from detector data that continually updates a network database file. These data are used as input to the TRANSYT-7F model for development of new optimized timing plans [Rowe 1991]. The system also alerts the operator when traffic flow measures have changed sufficiently to warrant the effort of developing new area-wide signal timing plans. Note that this is basically a 1-GC strategy except that data collection is performed automatically.

2.3.4 2-GC

The defining characteristic of 2-GC control is that the timing plans are calculated on-line based upon future traffic conditions which are forecasted by predictor algorithms. These algorithms make use of current immediate area surveillance data and historical data when projecting future traffic conditions.

Four such strategies have been tested in past field experiments: EQUISAT (EQUal degree of SATuration) [Gartner 1982], dynamic plan generation [Holroyd 1972], RTOP (Real-Time Optimization Program) [MacGowan and Fullerton 1979-80] and UTCS 2-GC [MacGowan and Fullerton 1979-80]. Under the EQUISAT model, cycle times and phasings are fixed as is the offset time. The allocation of green time is varied to equalize a smoothed average of the degree of saturation on each approach.

The dynamic plan generation method calculates the common cycle time and red-green splits according to Webster’s method using smoothed estimates of flow on each approach. Offsets are calculated to minimize delay or stops using measured or estimated traffic speeds. Under this method, settings can be recalculated every three cycles however, a transition between settings may require 1.5 cycles.

RTOP computes signal timing parameters on-line for successive 15-minute control periods. This duration was selected as a compromise between the requirements of traffic responsiveness and the disruptive nature of timing plan switches. Under RTOP, a range of possible cycle lengths and corresponding splits are calculated using Webster’s method. The offsets are determined by a method which combines the link
performance functions from the COMBINATION METHOD and the gradient optimization technique from SIGOP for each cycle length in the feasible range. The settings which produce the smallest value of the performance index is chosen for implementation.

The UTCS 2-GC program, denoted TANSTP (Traffic Adaptive Network Signal Timing Program) repeats its optimization procedure at 5-minute intervals, but new plans can be implemented at most every 10 minutes to avoid developing control parameters which are based on conditions that exist during signal timing transitions. TANSTP utilizes the optimization logic of the SIGOP model because of its short execution time. Its predictor algorithm predicts the volume and speed of traffic as a function of historical volume data. Finally, this program also possesses a transition model which allows the transition from the current timing plan to a newly computed plan to be explicitly optimized through offset adjustments [MacGowan and Fullerton 1979-1980].

2.3.5 3-GC

There have been few attempts to develop truly demand-responsive control strategies for signal network control. One of the reasons is the amount of real-time information needed and which until now has not been available.

A 3-GC tactic tested in the Glasgow experiments is PLIDENT (PLatooon IDENTification). PLIDENT has no formal cycle time or signal linking. It attempts to identify the movement of platoons of traffic on the road network and by predicting their arrival times, based on actual data, aims to operate the signals so as to process the platoons on designated priority routes. Only one approach per signal can be assigned priority. Adjustments are made to a signal's green time allocation according to the estimated platoon length, again obtained from actual upstream data [Gartner 1982].

An UTCS 3-GC scheme was also developed for testing. The 3-GC software was designed to permit the cycle length of each controller to vary from cycle to cycle. Signal timing is computed for each individual controller at least once every 3.5 minutes. Offsets and splits are determined to minimize vehicle delay and stops along each signal's approach and to provide network coordination. The optimization is accomplished by the CYRANO (Cycle-Free Responsive Algorithm for Network Optimization) procedure. Signal coordination is achieved by implementing a coarse simulation of traffic flow and then systematically adjusting the signal settings of each controller to minimize a linear combination of vehicle stops and delay aggregated over
all approaches. Transition routines are not needed for this control scheme because the system is constantly in a state of transition as it responds to changing traffic scenario [MacGowan and Fullerton 1979-80]. It was found that CYRANO increases delay significantly. This may be due in part because it's primarily goal was to generate plans which allowed variable cycle lengths rather than to minimize delay.

SCOOT (Split, Cycle, and Offset Optimization Technique) utilizes vehicle detectors to perform incremental optimization [Hunt et al. 1982, Robertson 1986, Luk 1984, Robertson and Bretherton 1991]. The three key ambitions of SCOOT are to measure traffic flows in real-time, to update an on-line model of queues continuously, and to perform incremental optimization of signal settings. Basically, the coordination plan responds to new traffic situations in a series of frequent but small increments. Its coordination plan can be either stretched or shrunk to match the latest situation of flow profiles. A phase split is increased or decreased a small amount (4 seconds) to minimize the sum of the queue lengths in the network. Once every cycle, the offset optimizer decides whether the performance index on the streets around each intersection can be reduced by altering the offset of the intersection by up to four seconds either way. Favorable splits and offset alterations are implemented immediately. In a similar fashion, it is decided if the cycle time for a group of intersections should be adjusted up or down, by a few seconds, once every few minutes. A small adjustment amount dampens the effects of normal fluctuations around the average flow. If the fluctuations do indeed indicate a long-term prevailing change, the sum of the small incremental changes effect a larger signal change adapted to the flow change over a period of time. Improvements can be made to SCOOT's modelling capability so it utilizes vehicle-actuated controllers more effectively [Luk 1984]. Currently, SCOOT is the only established real-time adaptive control scheme for signal networks but is not used in the United States (Chapter Three).

Two other real-time traffic responsive control strategies which have been developed for signal networks include PRODYN [Henry and Farges 1989] which was developed in France and UTOPIA [CCCT 1989] which was developed and implemented in Turin, Italy.

2.3.6 1-GC, 2-GC and 3-GC Experimental Results

There have been several comparison studies made both within the same generation of control strategies and across the generations. The major studies performed have been in Glasgow [Kaplan and Powers 1973, MacGowan and Lum 1975, Robertson

In the Glasgow experiments, the strategies tested include COMBINATION METHOD, TRANSYT, SIGOP, FLEXIPROG, EQUISAT, dynamic plan generation and PLIDENT. Based on the field results, it was concluded that the most effective strategy is to operate the traffic system on a fixed-cycle basis or under 1-GC. Of the 1-GC strategies tested, SIGOP and TRANSYT performed best with no statistical differences in their performances.

In San Jose, tests were also conducted between SIGOP and TRANSYT in 1972. Again it was found that there was no statistical difference in the performance of timing plans produced by TRANSYT and SIGOP. It was concluded from this study that the prediction model used in TRANSYT is more accurate than the one used by SIGOP [Kaplan and Powers 1973].

An extensive study of the effectiveness of various control strategies was conducted by the Corporation of Metropolitan Toronto in the mid-1970s [Gartner 1985]. Three 1-GC strategies, SIGOP, TRANSYT, and COMBINATION METHOD and one 2-GC strategy, RTOP, were tested. Among the 1-GC strategies, the COMBINATION METHOD proved to be slightly superior. RTOP tests yielded mixed results.

The largest and most comprehensive study undertaken in the United States was through the Urban Traffic Control System (UTCS) project conducted by the U.S. Department of Transportation in the early 1970s. The study was performed in Washington D.C. and was focussed towards the development and testing of three generations of control: 1-GC, 2-GC, and 3-GC. The different UTCS control strategies were designed to provide increasing degree of traffic responsiveness through a reduction of the update interval which in turn would potentially improve the quality of traffic flow. These expectations were not realized in the field studies.

The 1-GC strategies performed best overall and demonstrated significant delay savings over the previous coordinated timing system. The 2-GC was a mixed-bag of results but overall inferior to 1-GC. It demonstrated some improvement for arterials but degraded traffic flow in a network. The UTCS 3-GC seriously degraded traffic flow for most of the conditions under which it was tested.

Based on the Washington D.C. simulation tests, it was concluded that though there was no significant difference in the performance, the plans produced by TRANSYT
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appeared to be lightly superior to those produced by SIGOP. Given this and the fact that TRANSYT's prediction model appears to be more accurate and sophisticated, it was decided to use TRANSYT to generate UTCS timing plans. Today, TRANSYT is the most widely used timing strategy for network synchronization and has become a standard against which new timing strategies are tested [Chard and Lines 1987].

SECTION 2.4 General Observations

There are three observations which can be drawn from the literature which are relevant to my thesis.

The first observation is what may be intended to be a more traffic responsive system may not be so in reality. If the methodology attempts to achieve traffic responsiveness through shorter update intervals of off-line timing methodologies, it does not work. The unsuccessful 2-GC and 3-GC strategies discussed above were basically off-line procedures implemented on-line using detectors to sample current traffic conditions and a predictor algorithm to generate future traffic flow data necessary for the optimization subroutine. The results of the various field tests have indicated that there was no improvement and sometimes even a degradation in performance with these procedures.

One defect with this past approach is that the off-line optimization procedure must be simplified so that plans can be generated fast enough to be implemented in response to recent traffic conditions. The simplification of the optimization procedure naturally causes it to produce inferior settings. Secondly, there is an inherent lag between the actual traffic flow measurements and the implementation of the settings based upon these measurements. This lag causes potential mismatches between the implemented plan and the current traffic conditions which again produces inferior performance. Thirdly, frequent transitions between plans induce a high delay cost which is usually not recouped during the operation of the newly implemented plan. Finally, it is extremely difficult to accurately predict short-term fluctuations around an average flow rate. The shorter the prediction interval, the less accurate the prediction [Kreer 1976].

The second observation is that currently, in the United States, there is no established real-time traffic responsive control strategy for either isolated signals or coordinated signal networks. It is my goal to examine the cost-effectiveness of two
aforementioned existing real-time traffic responsive strategies for isolated intersections: ROPAC [Gartner 1982] and vehicle-actuated control using two detectors per approach [Wu 1990].

The final observation is that most of the models that have been discussed only use a delay criterion as their performance measure. Today, given heightened concerns about fuel consumption and pollution, it seems that there may be better solutions than those that minimize delay. It is important to understand the tradeoffs between delay, fuel consumption, and air pollution for each signal control method currently in use and when devising and evaluating new control technologies.
Chapter Three

National Survey Results

In order to ascertain (to some degree) the types of control policies and the types of signal methodologies which are actually used by traffic engineers, I conducted a survey of 320 urban traffic engineers. Furthermore, I was also interested in the concerns the engineers had regarding signal timing. A copy of the questionnaire which was mailed to the engineers and the compilation of the raw survey results can be found in Appendix B.

A discussion of the survey results will be organized along the same lines as the survey. Section 3.1 begins the discourse with who was selected for the survey, the number and composition of respondents, and flaws in the survey design. The following section, discusses the findings regarding the types of control strategies in use and the corresponding fraction of signals which are operated under each distinct control strategy for isolated signals and coordinated signal systems. Section 3.3 presents the timing methodologies favored by the urban traffic engineer, again for both the isolated intersections and systems of coordinated signals. This chapter closes with a discussion of the results of the miscellaneous information section of the survey.

SECTION 3.1 Preliminaries

The survey was mailed to 320 traffic engineers across the United States. The communities which were chosen to be included in the survey were not chosen in a
completely random fashion. Each of the fifty states were represented in the survey; the states with larger populations had a greater number of communities chosen for inclusion. Names and addresses of the urban traffic engineers were selected from the Institute of Transportation Engineers Directory 1990 [ITE 1990]. This directory contains a listing of urban traffic engineers alphabetically by city. When choosing the communities to be surveyed, my only explicit criterion was that at least one major city from each state be included. For states with larger populations, I included a larger number of major cities and other communities. Because I wanted to get a cross-section of community sizes, I intentionally selected communities which I did not recognize in an attempt to obtain smaller sized communities. There was no formal scheme used to proportionately represent the community sizes found in the United States in my choice of the surveyed communities.

My selection process is biased in the sense that only communities which have at least one person as is a member of ITE in their traffic engineering department could be selected. This may have excluded numerous communities with traffic signals that are maintained and operated by individuals who are not members of ITE.

Finally, my selection process only sampled community owned signals. Many traffic signals are owned and operated by state organizations and (for the large part) these are not represented in my survey.

3.1.1 Respondents

A total of 172 completed surveys were returned representing all fifty states. This implies a response rate of close to 54%\(^5\). Included in the total are surveys for two entire states. Traffic signals in these states are owned and operated by the state versus the local municipalities. Hence, the surveys which were received by communities in these states were forwarded to the appropriate state agency.

Figure 3.1 is a histogram for the population of the 170 communities which chose to participate. The relatively large number of smaller sized communities is the result of two factors: there were a larger proportion of surveys mailed to smaller communities (because there is a larger proportion of smaller communities in the US), and the time required to fill out the survey increases with the number of traffic signals within the community which favors smaller communities.

\(^5\) I am aware of at least one community without traffic signals which received a survey
Because the needs of a community under 50,000 and that of one greater than 250,000 obviously differ, I grouped the results into four distinct groups based on community size. A reason for doing so is to superficially identify different trends among communities of different sizes. I chose the categories so that there were roughly the same number of respondents in each. The four categories are (1) communities with population under 50,000, (2) communities with a population greater than or equal to 50,000 and less than 100,000, (3) communities with a population greater than or equal to 100,000 and less than 250,000, and (4) communities with a population greater than or equal to 250,000.

Sizes of Responding Communities

Total Number = 170*

* Does not include the survey for the two states.

Figure 3.1 Histogram of the population sizes of the respondent communities.

3.1.2 Survey Design Flaws

There are basically several different types of problems with the survey which I identified in hindsight: ambiguously worded questions, the omission of certain questions, and the omission of state agencies and consulting firms from the survey.

Reading through the survey answers brought forth some glaringly ambiguous questions. The first such question occurred on page one of the survey. Since my research aim is focussed primarily on isolated intersection control techniques, I only
wanted detailed information concerning the types of signals used at isolated intersections. Some individuals misunderstood the questions on the first page to be with regard to all signals: those operated in isolation and those operated in networks. In most of these cases, it was necessary to call the community to get the numbers which were applicable only to isolated intersections. This problem carried over into the questions regarding signal technologies which can be seen by the inventory of timing software packages listed in Section B.2. Some of the packages, e.g. FORCAST, listed for isolated intersections are primarily designed to develop a timing plan for coordinated signals.

The second such error occurs in the miscellaneous information section of the survey. In retrospect, the second question should have been worded as "What percentage of your signal networks were last retimed?" As it is, some individuals interpreted the fixed-time to refer to the signal controller hardware. These individuals did not think the question was applicable to them because their community had a network of coordinated actuated signals. I meant for the question to refer to coordinated signal networks in which the individual signals operated with a common background cycle. I received some phone calls requesting clarification of this question while in other cases, individuals indicated the question was not applicable to their community but it clearly was applicable given their answers to the previous questions.

The next problem with the survey pertains to omitted questions. In particular, it would have been useful to know the average number of vehicle detectors used per approach per actuated signal. This information is desirable for determining the cost effectiveness of one of the smart signal technologies I am investigating: vehicle actuated signals with two vehicle detectors built into the control logic.

Finally, since many municipalities rely on state owned signals\(^6\), it would have been useful to determine the relevant characteristics with respect to the operation of these signals. Clearly, signals owned by the state differ in character from those operated in municipalities. For one thing, states probably operate fewer networks of coordinated signals. In fact, I hypothesize that states probably own a higher fraction of isolated signals than do individual communities. Similarly, since some municipalities use consulting firms to develop and implement new signal timing plans, especially for

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\(^6\) One community in the population range of 50,000 to 99,999, indicted that there were a total of 100 signals within the city limits of which 46 were state owned and operated.
coordinated signal systems, it would have been worthwhile to solicit answers from these firms.

SECTION 3.2  Fractions of and Types of Signal Control Policies Used

The results of tables B.1-B.4 in Section B.2.1 (Appendix B) have been aggregated below in Table 3.1.

<table>
<thead>
<tr>
<th>Population Group</th>
<th>Total No. of Signals</th>
<th>Total No. of Isolated Signals</th>
<th>Isolated Intersections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fixed-Cycle</td>
</tr>
<tr>
<td>0 - 49,999</td>
<td>1,061</td>
<td>510</td>
<td>134</td>
</tr>
<tr>
<td>50,000 - 99,999</td>
<td>3,465</td>
<td>1,522</td>
<td>398</td>
</tr>
<tr>
<td>100,000 - 249,999</td>
<td>9,155</td>
<td>3,648</td>
<td>1,038</td>
</tr>
<tr>
<td>250,000+</td>
<td>31,168</td>
<td>8,386</td>
<td>1,687</td>
</tr>
<tr>
<td>TOTAL</td>
<td>44,849</td>
<td>14,066</td>
<td>3,257</td>
</tr>
<tr>
<td>% of TOTAL</td>
<td>31.4%</td>
<td>7.3%</td>
<td>6.1%</td>
</tr>
</tbody>
</table>

Table 3.1  Summary of the types of isolated signal control strategies and the fraction of signals which are operated under each type.

A few general observations can be made regarding the results. The first is that communities with larger populations tend to have a smaller fraction of isolated intersections. There are two obvious reasons for this. The major rationale is that larger communities naturally have signals which are closer together in conjunction with a greater congestion problem. The combination of these factors require signal coordination to help alleviate congestion. Secondly, larger communities usually have more resources in the form of larger engineering staffs and more funds, and so are more capable of developing and implementing coordinated signal systems.

A second observation is that the most common form of isolated intersection control is vehicle-actuation: over half of the isolated intersections are fully vehicle-actuated and over three-quarters of all isolated intersections are either semi-actuated or fully-actuated (including volume density actuation). Since vehicle-actuated control is the most traffic responsive control available, this reflects the communities' desire to provide motorists with the benefits of traffic-responsive control. Unfortunately, I did
not ask what percentage of the detectors for the vehicle-actuated signals are functional.

Finally, none of the communities indicated any form of traffic responsive control beyond the vehicle-actuation. The most advanced traffic responsive control for an isolated intersection found in the survey results was vehicle-actuation with volume density control. This type of control represents approximately 1% of the fully-actuated signals. Again, none of the vehicle-actuated control strategies incorporate any explicit optimization logic. This confirms the observation made in Chapter Two that currently in the United States, there are virtually no signals that are traffic responsive in the sense of optimizing a given performance measure for the prevailing traffic conditions.

Table 3.2 is the aggregation of the results found in Tables B.5-B.8 of Section B.2.1.

<table>
<thead>
<tr>
<th>Population Group</th>
<th>All Networks</th>
<th>Arterial Networks</th>
<th>Grid Networks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. of Networks</td>
<td>No. of Signals</td>
<td>No. of Networks</td>
</tr>
<tr>
<td>0-49,999</td>
<td>87</td>
<td>551</td>
<td>81</td>
</tr>
<tr>
<td>50,000-99,999</td>
<td>237</td>
<td>1,943</td>
<td>193</td>
</tr>
<tr>
<td>100,000-249,999</td>
<td>499</td>
<td>5,507</td>
<td>405</td>
</tr>
<tr>
<td>250,000+</td>
<td>1,531</td>
<td>22,783</td>
<td>1,791</td>
</tr>
<tr>
<td>TOTAL</td>
<td>2,354</td>
<td>30,783</td>
<td>1,791</td>
</tr>
<tr>
<td>% of TOTAL</td>
<td>76.1%</td>
<td>47.2%</td>
<td>23.9%</td>
</tr>
<tr>
<td>Avg no. of Signals</td>
<td></td>
<td>8.1</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2 Summary of coordinated signal network systems.

An obvious observation based on Table 3.2, which is implied by Table 3.1, is that larger communities have a larger fraction of coordinated signal networks.

An interesting observation is that over three-quarters of all coordinated signal systems are arterial networks but they account for a little under one-half of all coordinated signals. This implies that the average number of signals per grid network is greater than the average number of signals per arterial networks. This makes intuitive sense. Grid networks are generally found in central business districts (CBDs) where there is a high concentration of signals over a small area. This makes it advantageous to have one timing plan controlling all the intersections of which there is a large number. On the other hand, many arterial networks consist of signals along a single street which implies a smaller number of signals.
SECTION 3.3 Timing Technologies Used

The significant information regarding signal timing technologies for isolated intersections has been collected in Tables 3.3-3.5. These results are based upon Tables B.9-B.12 of Section B.2.2.

<table>
<thead>
<tr>
<th>Population Group</th>
<th>No. of Responses</th>
<th>Manual</th>
<th>Computer Package</th>
<th>Both</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 49,999</td>
<td>26</td>
<td>21</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>50,000 - 99,999</td>
<td>33</td>
<td>25</td>
<td>7</td>
<td>1</td>
</tr>
<tr>
<td>100,000 - 249,999</td>
<td>32</td>
<td>28</td>
<td>15</td>
<td>7</td>
</tr>
<tr>
<td>250,000 +</td>
<td>24</td>
<td>23</td>
<td>13</td>
<td>7</td>
</tr>
<tr>
<td>TOTAL</td>
<td>115</td>
<td>97</td>
<td>38</td>
<td>16</td>
</tr>
</tbody>
</table>

Table 3.3 Signal timing technologies used for isolated fixed-cycle control.

<table>
<thead>
<tr>
<th>Population Group</th>
<th>No. of Responses</th>
<th>Manual</th>
<th>Computer Package</th>
<th>Both</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 49,999</td>
<td>29</td>
<td>21</td>
<td>7</td>
<td>2</td>
</tr>
<tr>
<td>50,000 - 99,999</td>
<td>37</td>
<td>28</td>
<td>14</td>
<td>6</td>
</tr>
<tr>
<td>100,000 - 249,999</td>
<td>39</td>
<td>32</td>
<td>18</td>
<td>5</td>
</tr>
<tr>
<td>250,000 +</td>
<td>29</td>
<td>23</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>TOTAL</td>
<td>134</td>
<td>104</td>
<td>54</td>
<td>23</td>
</tr>
</tbody>
</table>

Table 3.4 Signal timing technologies used for isolated semi-actuated control.

<table>
<thead>
<tr>
<th>Population Group</th>
<th>No. of Responses</th>
<th>Manual</th>
<th>Computer Package</th>
<th>Both</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 49,999</td>
<td>23</td>
<td>16</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>50,000 - 99,999</td>
<td>38</td>
<td>30</td>
<td>14</td>
<td>8</td>
</tr>
<tr>
<td>100,000 - 249,999</td>
<td>42</td>
<td>34</td>
<td>19</td>
<td>7</td>
</tr>
<tr>
<td>250,000 +</td>
<td>33</td>
<td>30</td>
<td>29</td>
<td>15</td>
</tr>
<tr>
<td>TOTAL</td>
<td>136</td>
<td>110</td>
<td>70</td>
<td>34</td>
</tr>
</tbody>
</table>

Table 3.5 Signal timing technologies used for isolated fully-actuated control.
CHAPTER THREE. National Survey Results

There is a significant fraction of isolated signal control schemes operated under timing plans which were manually developed. It seems that manual methods are used roughly twice as much as computer packages and half the communities which use computer packages still operate some manually developed timing plans. The larger communities make greater use of available software packages than do the smaller communities. This probably is a reflection of larger traffic engineering departments and more funding.

Some of the more widely used computer packages include SOAP, PASSER II and the Highway Capacity Manual software (which is used to determine timing to allow for full intersection capacity).

The analogous information with respect to signal network systems is found in Tables 3.6 and 3.7 based on Tables B.13-B.16 (Section B.2.2).

<table>
<thead>
<tr>
<th>Population Group</th>
<th>No. of Responses</th>
<th>TRANSYT-7F</th>
<th>PASSER II</th>
<th>Other Software</th>
<th>Manual</th>
<th>Both</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 49,999</td>
<td>6</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>50,000 - 99,999</td>
<td>23</td>
<td>13</td>
<td>6</td>
<td>3</td>
<td>8</td>
<td>1</td>
</tr>
<tr>
<td>100,000 - 249,999</td>
<td>40</td>
<td>27</td>
<td>9</td>
<td>6</td>
<td>18</td>
<td>11</td>
</tr>
<tr>
<td>250,000+</td>
<td>40</td>
<td>26</td>
<td>6</td>
<td>17</td>
<td>26</td>
<td>16</td>
</tr>
<tr>
<td>TOTAL</td>
<td>109</td>
<td>68</td>
<td>22</td>
<td>28</td>
<td>56</td>
<td>29</td>
</tr>
</tbody>
</table>

Table 3.6 Signal timing technologies used for grid network systems.

<table>
<thead>
<tr>
<th>Population Group</th>
<th>No. of Responses</th>
<th>TRANSYT-7F</th>
<th>PASSER II</th>
<th>Other Software</th>
<th>Manual</th>
<th>Both</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 49,999</td>
<td>31</td>
<td>8</td>
<td>11</td>
<td>4</td>
<td>17</td>
<td>2</td>
</tr>
<tr>
<td>50,000 - 99,999</td>
<td>39</td>
<td>11</td>
<td>22</td>
<td>9</td>
<td>15</td>
<td>7</td>
</tr>
<tr>
<td>100,000 - 249,999</td>
<td>47</td>
<td>16</td>
<td>33</td>
<td>8</td>
<td>23</td>
<td>15</td>
</tr>
<tr>
<td>250,000+</td>
<td>39</td>
<td>12</td>
<td>28</td>
<td>12</td>
<td>26</td>
<td>22</td>
</tr>
<tr>
<td>TOTAL</td>
<td>156</td>
<td>47</td>
<td>94</td>
<td>33</td>
<td>81</td>
<td>46</td>
</tr>
</tbody>
</table>

Table 3.7 Signal timing technologies used for arterial network systems.

By far, the two most popular computer packages for timing signal networks are TRANSYT-7F and PASSER II (84, 87, and 90). This is not a particularly surprising
result since these packages have been developed and supported by the Federal Highway Administration (FHWA). For grid networks, TRANSYT-7F is more popular and for arterial networks, PASSER II is more popular. This is understandable because these two packages were designed for grid network timing and arterial network timing, respectively. Note that over half of the communities still have timing plans in place which were developed manually. In fact, one community in the range of 100,000-249,999 indicated that one of their signal networks is still operated under a plan which was developed in the 1950s.

A comment should be made regarding the use of PASSER II and TRANSYT-7F. Though PASSER-II and TRANSYT-7F were developed for arterial and grid networks respectively, they can be used for developing coordinated plans for the other type of system. Clearly, an arterial network is merely a special case of a grid network and so packages used for grid networks can be used on arterial networks. On the other hand, some traffic agencies find it necessary to impose some arterial progression scheme for the main streets upon the TRANSYT solution for grid networks. Their aim is to obtain a smoother flow of traffic on principal arteries than is allowed by TRANSYT settings alone [Liu 1988].

There are two major reasons that all communities have not switched to computer software packages. The first is that some communities lack the funding to acquire the packages and the trained personnel to use the packages. Secondly, there are some communities which can afford to buy the software but the amount of data collection required by the software package makes using it infeasible.

Finally, a significant number of communities indicated that they did not think software packages were user-friendly nor the program output particularly useful.

SECTION 3.4 Miscellaneous Information

From Table B.17, the percentage of isolated fixed-cycle signals surveyed which were last retimed less than a year ago was 14.1%. The percentage of these signals which were last retimed over three years ago was over 50%. For fixed-cycle coordinated networks, 23.7% were last retimed less than a year ago and 38.3% were last retimed more than three years ago, thus indicating that more effort is taken to keep signal network timing current. This is a practical policy in that networks are used much more heavily than isolated intersections and therefore offer the greater
potential for savings in each of the various performance measures. On the other hand, timing plans for both isolated and coordinated signals should be updated yearly to adjust to changes in traffic flow. It has been estimated that old plans deteriorate to cause an extra 3% of delay per year [Robertson and Bretherton 1991]. These numbers indicate that significant savings could be potentially realized by optimally timing the signals with the currently available techniques.

The next question in the miscellaneous information section of the survey asked the traffic engineers what percentage of signals they felt were optimally timed. This is a very ambiguously worded question and obviously very subjective. For example, one community indicated that more than 65% of their isolated fixed-cycle signals were last retimed over three years ago and that there was absolutely no signal coordination among any signals but nevertheless felt 100% of their signals were optimally timed. In contrast, another community of roughly the same size (within the 50,000-99,000 range) which had retimed essentially all their signals, both isolated and coordinated, within the last two years felt that only 25% of their signals were optimally timed.

These examples illustrate that this number can not be taken literally. However, this number can be used as an indicator of the traffic engineers satisfaction with the current status of the signal timing plans given the current technology. Keeping this in mind, across all the communities which responded to this question, it is felt that currently, approximately 60% of the nation's signals are optimally timed using available techniques. Surprisingly, this number is fairly constant across the four population groups.

Because I am interested in the tradeoffs between three particular performance measures, delay, fuel consumption, and air pollutant emissions, I asked the engineers which of the measures they explicitly consider when timing signals. They could indicate any number of four measures in response to the question. In addition to the three measures of interest to me, I also included maximizing the intersection capacity as a measure. This is a primary performance criterion for intersections which easily become oversaturated because it can be used to help avoid gridlock. The results of this question indicate that the most widely used measure was delay. Over 85% of the respondents indicated they used a delay measure, roughly 19% use a fuel measure, 6% use an air pollutant emissions measure, and 74% use some sort of capacity measure.

Finally, there were some recurrent concerns expressed among the traffic engineers in response to the last question of the survey. The three most common concerns were
lack of needed resources both money and personnel, the need to use the plans produced by the software packages as a starting point rather than an end-product, and general dissatisfaction with the available software packages.

Many individuals indicated their departments lacked the staff and funds to properly maintain quality signal timing plans. The population group in which this concern was raised most often was for communities with populations between 100,000 and 250,000. It appears as though smaller communities do not experience severe manpower problems because they are only responsible for a small number of signals. On the other hand, more medium- to large-sized communities tend to suffer from an inadequate manpower crunch probably because they are large enough to be able to afford to invest in timing software but not large enough to properly collect all the data needed to use the software effectively. Most of the largest communities (at least the ones which had enough personnel to respond to the survey) seem to have enough personnel, and/or funds to contract a consulting firm, for signal timing projects.

The last two concerns mentioned above are related. Many individuals felt that there does not exist a single software package which provides timing plans that work well on the streets without some in field fine-tuning and modifications. Some engineers feel that no matter how accurate the data may be, computer generated timing plans are just approximations which require engineering experience and expertise for fine-tuning adjustments to ensure they work effectively on the streets. This is a valid assertion. Since the models underlying the software packages make simplifying assumptions and are based on heuristics, it should not be expected that they will work well upon implementation without some modifications. Perhaps in the future, less emphasis should be placed on developing models which require extremely accurate input data and whose solutions require field adjustments anyway, and more emphasis should be placed on developing models which provide an equally good starting point solution but do not require such a large amount of input data.

Which leads to the last of the most common complaints. People perceive current computer packages, such as TRANSYT-7F, as requiring a huge amount of data which is extremely labor intensive to collect. Many individuals also do not perceive packages as being user-friendly. The solution from one traffic engineer’s viewpoint is a software package which would provide more accurate timing plans and has simple input and instructions.
CHAPTER THREE. National Survey Results

One individual also indicated another area for further investigation in addition to developing better software packages (see comment B.47). Essentially his observation was that if the hardware is not properly maintained, then it is not possible to reap the benefits of a good signal timing plan. Unfortunately, my survey did not attempt to ascertain the condition of the existing signal software. It may be worthwhile to determine the savings in the various performance measures which could be obtained through repairing existing signal hardware and to compare this to the savings obtainable through smart traffic signals.

Several communities sent me additional information regarding retiming projects that their communities have undertaken in the past. Some of this information is found in Section 8.6.1.

As a lead into what follows, there is one particular comment made by a traffic engineer that seems apt. This individual observed that

In the past few years very little research was devoted to isolated signal timing methodology. Hopefully, the 1990 Clean Air Act will trigger some awareness and focus on isolated intersections which are mostly located in smaller cities and towns. Improved signal timing at intersections yields in minimized delay, fuel consumption, air pollution and maximized capacity. Consequently, improve quality of life.

In the remainder of this thesis, I concentrate on isolated traffic-responsive timing methodology. This type of upgraded signal timing is shown to lead to improvements in the delay measure, fuel consumption measure and air pollutant emissions measure. To begin, I discuss and specify the performance measures of interest to me in the next chapter.
Chapter Four

Delay, Fuel Consumption, and Air Pollutant Emissions Measures

In the 1950s and 1960s, signal timing researchers were primarily concerned with two vehicular traffic problems: improving safety and improving operational efficiency measured in terms of increased intersection capacity and less delay. Two new dimensions to the problem were introduced in the 1970s: fuel consumption and air pollution. The nation's growing concern about these issues at that time are evidenced by the passing of the Clean Air Act in 1970 and the adoption by congress of mandatory fuel-efficiency standards through the corporate average fuel economy laws (CAFE) in 1975.

Not surprisingly, around the time of the enactment of the two aforementioned laws, studies began to appear regarding signal timing plans and fuel savings. In addition, there appeared studies which modelled fuel consumption and air pollution for urban traffic. Section 4.1 reviews some of the past studies which have devised models for urban fuel consumption and pollutant emissions. Also included are studies which specifically use these models to develop signal timing plans designed to minimize fuel consumption.

Based upon the studies cited in Section 4.1 and Chapter Two, I define the three major measures of particular interest to me. The functional forms for these measures are presented in Section 4.2.
CHAPTER FOUR. Delay, Fuel Consumption, and Air Pollutant Emissions Measures

For my chosen fuel consumption and air pollution measures, I require values for the coefficients which appear in them. Many studies cited in Section 4.1 derive values for these quantities. Section 4.3 presents the values for the fuel consumption model that have been used in the past. From these, I estimate values which better reflect the current vehicle fleet. The corresponding numbers for the pollutant emissions measures are in Section 4.4.

This chapter concludes with a summary of the key performance measures. This summary provides the functional form of each measure including the estimates for the required coefficients.

SECTION 4.1 Previous Fuel Consumption and Air Pollutant Emissions Studies

Fuel consumption and pollutant emissions models found in the literature can be classified into one of two broad categories:

(i) analytical expressions which represent fuel consumption and pollutant emissions as a function of the different modes of vehicular operation such as the free flow speed, the amount of delay, the number of vehicular stop/start maneuvers, etc., and

(ii) microscopic computer simulation methods which predict fuel consumption or pollution emissions directly from individual vehicle trajectories.

For my purposes, models within the first category are the most useful.

4.1.1 Fuel Consumption Models

Factors which affect fuel consumption may be grouped into three categories: vehicle design (weight of the vehicle, transmission type, engine design, etc.), use characteristics (vehicle occupancy and load, trip length, driver behavior), and driving conditions including vehicle speed, amount of stopped delay, road surface and gradient, and the magnitude and frequency of speed changes. Signal timing plans only directly affect the third category.

In 1983 Akçelik et al. [Akçelik, Bayley et al. 1983] published a paper which categorized existing fuel consumption models. Three model types (or levels) use traffic related variables only and of these, two types use analytical expressions (vs. simulation). The authors denote these as level II and level III models.
Basically, level III models compute fuel consumption per trip as a function of total travel time and distance (hence average travel speed). Some models in this category include other variables such as number of stops, number of brake applications, etc. These additional terms increase the prediction accuracy of the model but tend to sacrifice its sensitivity to small changes in traffic conditions as a result of the problem of multi-collinearity [Akçelik, Bayley et al. 1983].

A typical level III model is one developed by Evans, Herman and Lam [1976] of GM Research Labs. The authors perform a multivariate analysis of 17 traffic factors that influence fuel consumption in urban driving. From their results, they conclude that average trip time per unit distance is the single most important factor in explaining the variability of fuel consumption. The equation they propose as a predictor of urban fuel consumption is

\[ F = k_1 D + k_2 T \]  \hspace{1cm} (4.1)

where \( F \) is the total amount of fuel consumed during the trip, \( D \) is the length of the trip and \( T \) is the time required to complete the trip. The parameters \( k_1 \) and \( k_2 \) reflect properties of the individual vehicle and the traffic conditions. These variables can be determined for trips taken in a variety different traffic conditions. After (4.1) has been calibrated for a specific vehicle, it can be used to predict the average fuel consumption of the vehicle for urban trips by noting only the distance of the trip and time it took to complete it. Other models of this type have been developed by Evans and Herman [1976], and Newman, Alimoradian and Lyons [1989].

In general, Level III models, or basic fuel consumption regression models, are descriptive relations and so work well to estimate fuel consumption per trip but are not well suited for analysis in terms of changes in the values of individual predictor variables such as speed, idling time, and number of stops/starts. This is a result of correlations among the independent (predictor) variables [Akçelik, Bayley et al. 1983]. For a summary of additional level III models and their accuracy see Pitt et al. [Pitt, Lyons et al. 1987].

A major differentiating characteristic between level II and level III models is that level II models account for speed-smoothness of traffic not easily attainable with level III models. They accomplish this by assuming that the predictor variables are independent and try to measure the coefficient values directly instead of using regression. One particular type of model in this category is the elemental model.
Elemental models usually require as input such items as idling time, cruise travel time and speed, the number of stop/start operations, and travel distance. Other types of models in this category include positive kinetic energy (PKE) and positive inertial power (PIP) models. Input required by these models include total travel time, minimum and maximum speeds during each positive acceleration, and travel distance. An alternative form of the PKE model uses separate measurements of idling time and running time.

Though the PIP and PKE models are macro-level models, their input data requirements are of the micro-level thus implying the need for costly data collection. A paper published by Pitt et al. [Pitt, Lyons et al. 1987] evaluates a PKE model, PIP model, an elemental model, and various level III models. The authors conclude that both the PKE model and PIP model outperform all other models. It can be surmised that if an accurate overall fuel consumption estimate (per link or per trip) is required, a PKE or PIP model should be used. However, these models are not suitable for analyzing incremental effects of stop/start operations and delay [Akçelik, Bayley et al. 1983]. Furthermore, PKE and PIP input data are difficult to measure by normal traffic engineering techniques [Pitt, Lyons et al. 1987].

The remaining model is the elemental model. This model has enjoyed widespread use by traffic engineers to predict urban fuel consumption. Usually, these models express fuel consumption as a function of three traffic performance variables: the amount of travel at cruising speed, the delay time, and number of stops.

Use is made of an elemental model to show that signal timing plans, for isolated fixed-cycle control, designed to minimize delay do not minimize fuel consumption. Bauer [1975] constructed an elemental model which expresses fuel consumption as a function of idle time and the number of stop/start maneuvers. From his results, he concludes that the cycle length required to minimize fuel consumption is longer than one which minimizes total intersection delay. As an element of interest, Bauer did not solve for a red-green split which would minimize fuel consumption. Instead, he held the splits fixed at a value proportional to the relative critical volumes, and then determined the optimal cycle lengths. As I show later (Chapter Five), this is not the split value which necessarily minimizes fuel consumption.

Courage and Parapar [1975] extend Bauer's results to vehicle-actuated control for an isolated intersection. From simulation results, the authors conclude that longer extension periods are needed to minimize fuel consumption than to minimize delay.
Their result seems to be more a consequence of the longer cycle lengths used (determined by Bauer's work) rather than from a new method for the calculation of the extension intervals suited to minimize fuel consumption.

Neither Bauer, nor Courage and Parapar refer to the fuel consumption model they use as an elemental model. In 1981, Akçelik used this terminology to describe his model. His model expresses fuel consumption as a function of the amount of distance covered at cruising speed, the delay time, and the number of stops. The functional form of his fuel consumption expression is

\[ f = f_1 l + f_2 d_s + f_3 h \]  

where

- \( f \) = average fuel consumption per vehicle (gal/veh),
- \( f_1 \) = fuel consumption rate while cruising (gal/veh-mile),
- \( f_2 \) = fuel consumption rate while idling (gal/veh-hr),
- \( f_3 \) = fuel consumption rate per complete stop (gal/stop),
- \( l \) = cruising distance (miles),
- \( d_s \) = average stopped delay (idling) per vehicle (hours),
- \( h \) = stop rate (number of complete stops per vehicle).

Akçelik [1981] argues that this type of model provides a good degree of accuracy in relative terms, i.e. for fuel consumption comparisons between two different signal timing plans, rather than precise accuracy in overall fuel consumption prediction. Using this model, he also concludes that a longer cycle length is necessary to minimize fuel consumption than is necessary to minimize delay. The main contribution of an elemental model is its suitability for predicting the incremental effects of delays and number of stops/starts due to traffic signal timing plans [Akçelik, Bayley et al. 1983]. A more recent paper [Ferreira 1985] also uses an elemental model to predict fuel consumption by each element of an urban trip.

4.1.2 Pollutant Emissions Models

The Clean Air Act limits the carbon monoxide (CO), hydrocarbon (HC) and oxides of nitrogen (NO\(_x\)) which can be emitted through tail-pipe exhaust. These three pollutants were targeted because of their proven harmful effects. From a health standpoint, carbon monoxide which is lethal in large doses, can be tolerated at the levels found in cities by healthy individuals. However, individuals with lung or heart deficiencies are adversely affected by the CO concentrations found in the worst polluted urban areas because it hinders oxygen transport by the blood. NO\(_x\) and HC
are products of inefficiently burnt fuel. They are only mildly harmful by themselves however they are major components in the production of secondary pollutants, ozone and nitrogen dioxide, which are both harmful to vegetation and human health.

As is true of fuel consumption, emissions from vehicles in urban traffic can be described in terms of a cruise component, an idle component, and a component consisting of the excess emissions of a stop/start maneuver above the vehicle's emissions in the cruise mode [Patterson 1975]. There are basically two different aspects that can be modelled. The first is to describe the emissions from vehicles as a result of given traffic conditions, and the second is to determine the air pollution concentration at various points along a roadway or at an intersection. Obviously, the results of the first model can be used as input for the second model.

My primary aim is to model emissions from vehicles as a result of signal control techniques so I require a model of the first kind. Because pollutant emissions can be described by the various aspects of the vehicle operation mode, an elemental model can be used. Patterson [1975, 1976] makes use of such a model to examine the effect of traffic signal settings on pollution levels. From his results, he concludes that cycle lengths longer than those which minimize delay are needed to minimize CO emissions.

Akçelik [1981] proposes the same elemental model for emissions as for fuel consumption. His emissions measure has the identical form as equation (4.2). The only things which differ are the coefficients \( f_1, f_2, \) and \( f_3 \). Al-Khalili [1985] also uses an elemental model for emissions. His model differs from that of Akçelik in that he adds another term; in addition to the number of stops made by vehicles, Al-Khalili includes the number of partial stops made and so his pollution emissions measure has four terms instead of only three.

Other modelling techniques exist for pollution emissions in addition to the elemental model. See a paper by Post et al. [Post, Kent et al. 1984] for a power demand model. Disadvantages of this type of model include the large amount of data required and the relative insensitivity to changes in the number of stop/start maneuvers and idling time. A more recent paper [Al-Omishy and Al-Samarrai 1988] describes a simulation model for predicting pollution emissions.
SECTION 4.2 Functional Form for the Delay, Fuel Consumption, and Pollutant Emissions Measures

In the remainder of my thesis, I use two delay measures. Either measure can be obtained from the other because I use the expected values of the measures. One of the measures I use is the delay per randomly arriving vehicle, denoted by $d$. The other measure is the total delay per unit time caused by the operation of the signal, denoted by $D$.

Because I am primarily interested in comparing the delay, fuel consumption and pollution emissions under different signal control methods and different signals timing plans, my fuel and pollution measures are based upon an elemental model. Practically all the effects observed at a signalized intersection can be expressed through stopped delay at the signal and speed changes at or near the intersection and so a reasonable approximation for my work is to ignore the signal's influence on mid-block speed and speed changes. This leaves only two factors in the estimation of the change in fuel consumption and pollution emissions due to signal timing: total stopped delay and the number of vehicles required to stop. This is essentially the same model used by Courage and Parapar [1975], and Bauer [1975]. As pointed out by Akçelik [1981], an elemental model can also be used to express pollution emissions. Hence these two measures have the same functional form but different conversion coefficients.

The functional forms of fuel consumption measure and air pollution measure which I use are

$$f = \alpha D + \beta S,$$

and

$$p_i = \gamma_i D + \delta_i S, \quad i = \text{CO, HC and NO}_x,$$

(4.3)

(4.4)

respectively. The parameters of the equations are defined below.

- $f$ = amount of fuel consumed per minute (gal/min)
- $\alpha$ = a fuel conversion coefficient (gal/veh-min)
- $\beta$ = a fuel conversion coefficient (gal/veh-stop)
- $D$ = total amount of veh-delay per minute (veh-min/min)
- $S$ = total number of vehicle stops per minute (stops/minute)
- $p_i$ = amount of pollutant $i$ emitted per minute (g/min)
- $\gamma_i$ = a pollutant conversion coefficient for pollutant $i$ (g/veh-min)
- $\delta_i$ = a pollutant conversion coefficient for pollutant $i$ (g/veh-stop)
CHAPTER FOUR. Delay, Fuel Consumption, and Air Pollutant Emissions Measures

For the models to be useful, representative values must be assigned to the coefficients. Values which have been developed for past studies, and values which are more appropriate for today, are presented next for the fuel consumption measure. The corresponding material for the pollution emissions measures is presented Section 4.4.

SECTION 4.3 Values for Fuel Consumption Conversion Coefficients

There are two coefficients necessary for the fuel consumption measure: \( \alpha \) which is the coefficient for the stopped delay and is equivalent to the idling fuel consumption rate expressed in gal/h (or gal/veh-min), and \( \beta \) which is the coefficient associated with a stop/start maneuver (assuming a 30 mph cruising speed) expressed in gal/stop. First I present the values which have been used for these coefficients in previous studies and then I present my rough estimates for representative values of these coefficients today.

4.3.1 Previously Used Values

In 1971, Claffey [1971] published a report entitled "Running Costs of Motor Vehicles as Affected by Road Design and Traffic". Within his report, he presents relevant measures for a composite car. He determined this car on the basis of observing a total of 35,000 vehicles on principal highways in eight states. Test data were obtained for typical cars in each weight class for operation on all grades and road surfaces, for speed change cycles, for different traffic conditions, and for stationary idling. Using the results of Claffey's study, the values of the coefficients for his composite car are \( \alpha = 0.58 \text{ gal/h} \) and \( \beta = 0.0097 \text{ gal/stop} \).

Bauer [1975] states that measured values of \( \alpha \) for individual automobiles under laboratory conditions are between \( \alpha = 0.083 \text{ gal/h} \) and \( \alpha = 0.66 \text{ gal/h} \). Based on this range, he uses \( \alpha = 0.50 \text{ gal/h} \) as a typical standard idling fuel consumption rate. Similarly, Bauer provides a range of values for \( \beta \), \( \beta = 1/30, 1/50, 1/100 \) and \( 1/200 \) gallons per stop. For his analysis, he used \( \beta = 1/30 \text{ gal/stop} \).

The values used for \( \alpha \) and \( \beta \) in the Courage and Parapar [1975] study are based upon the results of Claffey's [1971] study. These authors use \( \alpha = 0.60 \) and \( \beta = 0.010 \).
Values used by Claffey [1976] in a study he performed to determine the effect of individual driving habits and vehicle maintenance on fuel consumption were $\alpha = 0.73 \text{ gal/h}$ and $\beta = 0.0072 \text{ gal/stop}$. Robertson et al. [Robertson, Lucas and Baker 1980] use TRANSYT (Section 2.3.2) to predict the amount of fuel consumed within a network of coordinated signals as a function of the distance travelled, the total delay time, and the number of stops. Their values of $\alpha$ and $\beta$ are $\alpha = 0.39 \text{ gal/h}$ and $\beta = 0.0037 \text{ gal/stop}$.

Akçelik [1981] did his own review of numbers in the literature for $\alpha$ and $\beta$. Based upon the previously used numbers, he makes a best guess of what the appropriate numbers should be and uses these in his examples. The value he uses for $\alpha$ is 0.57 gal/h and for $\beta$ is 0.01 gal/stop. He also provides numbers for heavy vehicles which includes busses and trucks: $\alpha = 0.65 \text{ gal/h}$ and $\beta = 0.02 \text{ gal/stop}$.

Other estimates found in the literature include: $\alpha = 0.43 \text{ gal/h}$ and $\beta = 0.0080 \text{ gal/stop}$ [Post, Kent et al. 1984], $\alpha = 0.52 \text{ gal/h}$ and $\beta = 0.010 \text{ gal/stop}$ [Al-Khalili 1985], $\alpha = 0.31 \text{ gal/h}$ and $\beta = 0.006 \text{ gal/stop}$ [Ferreira 1985], and $\alpha = 0.76 \text{ gal/h}$ and $\beta = 0.003 \text{ gal/stop}$ [Pitt, Lyons et al. 1987]. As can be seen, the values of the coefficients which have been used in the past cover a relatively wide range.

Table 4.1 summarizes all the values for the fuel consumption conversion coefficients which I found in the literature.

4.3.2 Estimate for Current Values of the Coefficients

Unfortunately, it is hard to obtain an adequate database from which the vehicle specific parameters $\alpha$ and $\beta$ can be determined for vehicles today. I have not been able to find a study comparable to the 1971 Claffey study. In addition to the literature, other sources from which I have tried to obtain these values include the EPA, the Sloan Automotive Lab at MIT, the Department of Transportation, and GM Research Labs.

To obtain the needed numbers, it is necessary to test cars in each weight class which adequately represent the class and then obtain the weighted average where the weights reflect the current mix of cars on urban streets. In the past, authors have done this or similar things. The value for the idling fuel consumption rate only depends on whether or not the engine is warm and so do not vary greatly over many tests on the same auto. On the other hand, there is a wide-variation in the stop-start fuel penalty at any given velocity. This is a result of the different stop-start velocity profiles which occur in real driving conditions. For example, Post, Kent et al. [1984]
found that for a stop-start maneuver of a given vehicle, $\beta$ ranged from 0.0039 gal to 0.0169 gallons.

<table>
<thead>
<tr>
<th>Study Cited</th>
<th>$\alpha$: Idling Fuel Consumption Rate (gal/h)</th>
<th>$\beta$: Excess Fuel Consumption per Stop (gal/stop)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Claffey [1971]*</td>
<td>0.58</td>
<td>0.0097</td>
</tr>
<tr>
<td>Bauer [1975]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>range of values</td>
<td>[0.83, 0.66]</td>
<td>(1/30, 1/50, 1/100, 1/200)</td>
</tr>
<tr>
<td>typical value</td>
<td>0.50</td>
<td>0.0333</td>
</tr>
<tr>
<td>Courage and Parapar [1975]</td>
<td>0.60</td>
<td>0.0100</td>
</tr>
<tr>
<td>Claffey [1976]*</td>
<td>0.73</td>
<td>0.0072</td>
</tr>
<tr>
<td>Roberson, Lucas and Baker [1980]*</td>
<td>0.39</td>
<td>0.0037</td>
</tr>
<tr>
<td>Akçelik [1981]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>cars</td>
<td>0.57</td>
<td>0.0100</td>
</tr>
<tr>
<td>Busses and Trucks</td>
<td>0.65</td>
<td>0.0200</td>
</tr>
<tr>
<td>Post, Kent et al. [1984]*</td>
<td>0.43</td>
<td>0.0080</td>
</tr>
<tr>
<td>Al-Khalili [1985]</td>
<td>0.52</td>
<td>0.0100</td>
</tr>
<tr>
<td>Ferreira [1985]*</td>
<td>0.31</td>
<td>0.0060</td>
</tr>
<tr>
<td>Pitt, Lyons et al. [1987]*</td>
<td>0.76</td>
<td>0.0030</td>
</tr>
</tbody>
</table>

* These studies directly measured the coefficient values.

Table 4.1 Values for $\alpha$ and $\beta$ used in previous studies.

Due to my lack of success in locating current values for $\alpha$ and $\beta$, I resigned myself to fabricating estimates for these values. Since I have $\alpha$ and $\beta$ values for a 1971 composite USA car, I try to appropriately rescale these numbers to reflect the current car fleet. From the *Basic Petroleum Data Book* [API 1991], the average passenger car obtained 13.73 miles per gallon in 1971. In 1989, the comparable number was 20.62 miles per gallon. Assuming that the types of miles driven has remained unchanged and that the relative change in miles per gallon accurately reflects the change in the average idling fuel consumption rate and expected fuel consumed during a stop/start maneuver, I estimate today's $\alpha$ and $\beta$ values to have changed by the same percentage amount as the average miles per gallon over this period.

The percentage change in the average miles per gallon has been $13.73/20.62 = 0.666$. Hence, as a crude, first-order approximation for current $\alpha$ and $\beta$ values, I use $\alpha = (0.58)(0.666) = 0.39$ gal/h and $\beta = (0.010)(0.666) = 0.0067$ gal/stop.
One check on my estimates is that they are lower than the corresponding numbers for the 1971 fleet as they should be because cars have become more fuel efficient. Another check is to compare my estimated values to those presented in Table 4.1. Upon examining the values used for studies performed in the 1980s which measured $\alpha$ and $\beta$ directly, it can be seen that my values are similar. Specifically, the Ferreira [1984] study which uses data from Leeds has numbers that are very close to my estimates. His values are a little smaller which is not unexpected because the relatively higher European gas prices has traditionally meant that European cars obtain better gas mileage than US cars.

Finally, in a recent article in the Seattle Times/Seattle Post-Intelligencer [McGrath 1991], it was reported that idling cars burn between 0.5 gallons and 1.0 gallons per hour. If these cited estimates are true, the estimates I use are underestimates and would produce conservative values for the amount of fuel used due to the operation of traffic signals. There is no reason to believe that my rescaling results in accurate values for $\alpha$ and $\beta$. Though, it is my belief that my estimates provide a closer approximation of reality than would be obtained by using the 1971 numbers.

SECTION 4.4 Values for Pollutant Emissions Conversion Coefficients

For each $i$, $i = \text{CO}, \text{HC}$ and $\text{NO}_x$, there are two coefficients required for the pollutant emissions measure: $\gamma_i$, which is the coefficient for the amount of idling time expressed in g/h and $\delta_i$, which is the coefficient associated with a stop/start maneuver (assuming a 30 mph cruising speed) expressed in g/stop. I present values which have been used for these coefficients in previous studies, and then I present my rough estimates for representative values of these coefficients today.

4.4.1 Previously Used Values

A major relevant study was performed in 1974. This study was entitled "Automotive Exhaust Emissions Modal Analysis Model" [Kunselman, McAdams et al. 1974]. The authors developed an analytical modal model to describe and predict emissions from vehicles. They built a model which is flexible enough to handle any specific driving sequence. Within the model, there are 37 distinct operating modes allowed for the automobiles and of these modes, five assume a constant speed and the remaining 32 describe various acceleration or deceleration maneuvers characterized by an average value. Data were used for 1020 vehicles representative of model year,
manufacturer and drive train equipment, accumulated mileage, state of maintenance, and geographic location. The values of the coefficients of interest to me were obtained from this study by Sachs [1990]. These are $\gamma_{CO} = 975$ g/h, $\gamma_{HC} = 81$ g/h, $\gamma_{NOx} = 6$ g/h, $\delta_{CO} = 3.189$ g/stop, $\delta_{HC} = 0.164$ g/stop and $\delta_{NOx} = 0.02$ g/stop.

Patterson [1975] only presents coefficient values for CO: $\gamma_{CO} = 842$ g/hr and $\delta_{CO} = 6.406$ g/stop. In his 1976 study, Patterson uses the same values but in slightly different form. Post et al. [Post, Kent et al. 1984] only model HC and NO$_x$ emissions because their power demand model could not accurately describe CO emissions: $\gamma_{HC} = 52.8$ g/h and $\gamma_{NOx} = 0.0$ g/h. From their paper, it was not possible to obtain the values for $\delta_{HC}$ and $\delta_{NOx}$.

Al-Khalili [1985] argues that since the main pollutant is CO and since there is a linear relationship between the amount of CO emitted and other pollutant gasses, then by minimizing CO, the other exhaust gasses are also minimized. Al-Khalili uses the values $\gamma_{CO} = 842$ g/h and $\delta_{CO} = 6$ g/h.


Table 4.2 summarizes the values of the pollution conversion coefficients I found in the literature.

<table>
<thead>
<tr>
<th>Study Cited</th>
<th>$\gamma_i$: Idling Coeff. (g/h)</th>
<th>$\delta_i$: per stop coeff (g/stop)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{CO}$</td>
<td>$\gamma_{HC}$</td>
<td>$\gamma_{NOx}$</td>
</tr>
<tr>
<td>Kunselman, McAdams et al.[1974]* (extrapolated by Sachs[1990])</td>
<td>975</td>
<td>81</td>
</tr>
<tr>
<td>Post, Kent et al. [1984]*</td>
<td>NA</td>
<td>52.8</td>
</tr>
<tr>
<td>Al-Khalili [1985]</td>
<td>842</td>
<td>NA</td>
</tr>
<tr>
<td>Al-Omishy and Al-Samarrai [1988] (extrapolated by Wu [1990])</td>
<td>NA</td>
<td>68</td>
</tr>
</tbody>
</table>

* These studies directly measured the coefficient values.

Table 4.2  Values for $\gamma_i$ and $\delta_i$ used in previous studies, $i = CO, HC$ and $NO_x$.

4.4.2 Estimate for Current Values of the Coefficients

Shattanek, Kahng and Stratou [1990], under the auspices of the EPA, have developed a model called CAL3QHC which is a microcomputer-based modelling methodology developed to predict the level of CO, or other inert pollutants,
concentrations from motor vehicles travelling near street intersections. The study assumes that vehicles are either in motion or idling and does not specifically account for the amount of pollutants emitted during a stop/start maneuver. As input, this program requires idling emissions rates and emissions rates at a steady speed. As can be seen from Table 4.2, the majority of the pollutants are emitted during idling as a result of a signal timing plan.

Because of the input required by CAL3QHC and similar models, there have been other models developed which only ascertain idling emissions rates and cruising emission rates. Thus, it is difficult to obtain emissions rates per stop/start maneuver and I was not successful in locating recent values for these rates. My main source for emissions rates and emissions data was the EPA.

To obtain the idling emissions rates for the vehicles currently on the road today, I used a program called MOBILE4 [EPA 1989]. The emissions rates are based on actual values measured by the EPA. This program provides the emissions rates for idling vehicles and for vehicles moving at any specified speed. Emissions rates for vehicles depend on the temperature: CO emissions are greater at lower temperatures while HC emissions will be greater at higher temperatures. Because of this, I choose to use the worst case analysis for each gas, i.e. for CO emissions rate, I assume a low temperature (30°F) and for HC emissions I assume a higher temperature (70°F).

To obtain the values of $\gamma_i$, $i = \text{CO, HC and NO}_x$, I used a PC-based program called MOBILE4. For all factors, I assume that the current year is 1991, the federal default for vehicle mix and thermal state, no inspection or maintenance program in place, and the American Society for Testing and Materials (ASTM) volatility class C. For CO emissions rates, I assume an average temperature of 30°F and Reid vapor pressure (RVP) of 13.5. For HC modelling, I use an average temperature of 70°F and RVP of 9.0 psi. Using these parameters and applying the appropriate correction factors to obtain the idle emission rates [EPA (memo) 1989], I obtained $\gamma_{\text{CO}} = 652.88 \text{ g/h}$, $\gamma_{\text{HC}} = 35.94 \text{ g/h}$ and $\gamma_{\text{NO}_x} = 4.63 \text{ g/h}$.

To obtain the excess amount of pollutant $i$ emitted per stop for $i = \text{CO, HC, NO}_x$, I assume an average deceleration rate of 3 mph/sec and an acceleration rate of 2.75 mph/sec. Then, I compute the speed for each second and apply the emissions factor corresponding to that speed. This gives me an estimate of the amount of pollutant $i$ per stop/start maneuver. To calculate the excess amount of pollutant $i$ emitted, I subtract the amount of pollutant $i$ emitted if the same distance were travelled at 30
mph. Thus the values I calculated were $\delta_{CO} = 3.90 \ g/stop$, $\delta_{HC} = 0.26 \ g/stop$ and $\delta_{NOx} = 0.13 \ g/stop$.

The idling emissions factors can be considered to be accurate for the conditions specified. On the other hand, the excess emissions per stop/start depends on the individual stop/start profiles and is only applicable to an "average" stop/start maneuver which I have specified above. Furthermore, my values are a rough estimate based upon the average speed of the vehicle per second during its acceleration and deceleration modes.

SECTION 4.5 Summary

The model I have chosen to describe both the amount of fuel consumed and the amount of pollutant $i$, $i = CO$, HC and NO$_x$, emitted per unit time, is an elemental model. Previous researchers have shown this type of model to be appropriate for comparing incremental changes in stopped delay time and number of stops as a consequence of different signal control strategies.

Using the available values for the necessary fuel consumption coefficients found in previous studies, I make a rough estimate of the appropriate value for these coefficients today. These estimates are only a crude first-order approximation and are not guaranteed to be correct though they are probably closer to the true values than the 1971 values that are available. It is my belief that these are underestimates for the true amount.

For the pollutant emission equations, I was able to obtain accurate and current coefficients for the idling emissions rates. For the stop/start excess emissions rates, I estimated the numbers based upon the currently available data. The amount of pollutants emitted during a stop/start maneuver are relatively small compared to the amount emitted during a cruising mode and idling mode.

Note that all my measures only pertain to automobiles. This is because of the lack of available data for trucks and busses. Currently, autos account for 70.1 percent of all urban vehicles [EPA 1989].

Table 4.3 contains my performance measures for fuel consumption and pollutant emissions. The coefficients in each equation are meant to reflect the current passenger car fleet.
<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fuel Consumption Rate (gal/min)</td>
<td>$f = 0.0065 , D + 0.0067 , S$</td>
</tr>
<tr>
<td>CO Emissions Rate (g/min)</td>
<td>$\rho_{CO} = 10.881 , D + 3.90 , S$</td>
</tr>
<tr>
<td>HC Emissions Rate (g/min)</td>
<td>$\rho_{HC} = 0.599 , D + 0.26 , S$</td>
</tr>
<tr>
<td>NO\textsubscript{x} Emissions Rate (g/min)</td>
<td>$\rho_{NO_x} = 0.077 , D + 0.13 , S$</td>
</tr>
</tbody>
</table>

Table 4.3. Equations for the performance measures where $D = \text{delay rate (veh-min/min)}$ and $S = \text{rate of stops (stops/min)}$. 
Chapter Five

Pareto Optimal Sets for Fixed-Cycle Signal Control

Traffic signal timing should be viewed as a multivariate value problem. Like most complex problems, it has multiple conflicting objectives. Some timing plans are clearly inferior because alternative plans exist which yield better results in all performance measures. Such a situation can occur when comparing a plan of a less traffic-responsive technology to one for a more traffic-responsive technology. But within a given traffic signal technology, it is generally not possible to derive a plan which optimizes all the performance measures simultaneously. Thus, the traffic engineer encounters the problem of trading improvement in one measure at the expense of degrading the performance in at least one other measure.

Today, most signals in the United States are timed with only one explicit objective function: minimize some measure of expected delay (Chapters Two and Three). Such a policy does not produce a plan which minimizes the total fuel consumption rate or total pollutant emissions rate. A simple reason for this is that plans devised to minimize total delay are indifferent between stopping two vehicles for five seconds and one vehicle for ten seconds. Whereas, a plan designed to minimize the total fuel consumption rate would choose to delay one vehicle for ten seconds because it is relatively more expensive, in terms of the amount of fuel consumed, to stop and start a vehicle than to have it idle for a short period of time.
Given that it is necessary to tradeoff improvement in one performance measure against improvement in another performance measure, the essence of the issue then becomes how much is society willing to pay in terms of degrading the performance of some measures for a fixed amount of improvement in a specified measure. There have been few attempts to answer this question. Some of the studies cited in section 4.1 explore tradeoffs between delay and fuel consumption, though none of them formally model this tradeoff.

Obviously, this tradeoff issue is a value question for society and requires some subjective judgement on the part of the municipal department responsible for traffic signal timing. There is no clear right or wrong answer. How does one value a gallon of gasoline in terms of seconds of delay? Are monetary values necessarily the correct values to use? What is the value of a unit of air pollution in terms of seconds of delay or gallons of gas?

Since it is difficult to explicitly formalize the societal value structure so that the contending alternative signal timing plans can be compared, I instead determine the set of nondominated fixed-cycle timing plans with respect to the measures. This set is denoted a Pareto optimal set or an efficient frontier. There are five performance measures (of which three pertain to air pollution) of particular interest to me: CO emissions, HC emissions, NO_x emissions, delay, and fuel consumption. The main reason I focus on fixed-cycle technology is that useful expressions can be obtained for the expected delay. As the forms of control become more complex, expressions for expected delay become intractable. Though I have chosen the fixed-cycle signal for illustrative reasons, the tradeoffs can have an impact today because there are approximately 17,500 isolated signals operated in the United States.

Section 5.1 reviews a definition of a Pareto optimal set with respect to a signal timing plan. The notation and the assumptions associated with the general fixed-cycle traffic signal timing problem are introduced in Section 5.2.

The following section introduces a simplistic model which assumes that vehicles queued at the intersection cross instantaneously upon the commencement of a green signal. This is one of the simplest ways to illustrate the Pareto optimal frontiers.

Sections 5.4 and 5.5 develop more complicated models for the fixed-cycle signal. These models are more realistic and illustrate the manner in which the results obtained for the simple model change. They also highlight features which appear to remain constant across all three models.
CHAPTER FIVE. Pareto Optimal Sets for Fixed-Cycle Signal Control

The last section summarizes my findings and draws some relevant conclusions.

SECTION 5.1 Definition of Pareto Set for Signal Timing Plans

Let the timing plans \( t' \) and \( t'' \) have values \( x^t = (x_d, x_i, x_p) \) and \( x^{t''} = (x_d'', x_i'', x_p'') \) where \( x_d \) is the amount of expected delay per randomly arriving vehicle, \( x_i \) is the expected amount of fuel consumed per unit time, and \( x_p \) is the expected amount of pollutants emitted per unit time. It is said that \( x^t \) dominates \( x^{t''} \) when

\[
\begin{align*}
(1) & \quad x_i' \leq x_i'', \quad \text{all } i \\
(2) & \quad x_i' < x_i'', \quad \text{for some } i.
\end{align*}
\]

The above states that if \( x^t \) dominates \( x^{t''} \), then the timing plan \( t' \) is at least as good as \( t'' \) in each performance measure (5.1) and is strictly better for at least one (5.2).

Let \( X \) be the set of performance measure vectors that are associated with timing plans in \( T \). The set of performance measure vectors of \( X \) that are not dominated is called the efficient frontier of \( X \) or is known as the Pareto optimal set. Once the Pareto optimal set is known, it can be used to help community leaders informally weigh the tradeoffs in the performance measures among the available timing options and based on their preferences, choose a "best" plan.

SECTION 5.2 General Assumptions and Notation

Before obtaining a Pareto optimal set of timing plans for a fixed-cycle signal it is necessary to obtain the expected delay for a randomly arriving vehicle, the expected amount of fuel consumed per minute, and the expected amount of pollutant \( i \) emitted per minute (for all \( i \)). Factors affecting these quantities are the cycle length, the fraction of the total useable cycle length the light is effectively green for each approach, the number of queued vehicles which can pass through the intersection during a green interval, and the vehicular arrival process and average arrival rate. The cycle length and red-green splits are parameters specified by traffic engineers based on the traffic flow characteristics which are obtained through field observations.
5.2.1 Assumptions

With regard to the operating characteristics of the fixed-cycle intersection, I make four assumptions: vehicular arrivals are Poisson, vehicular departure times are constant, the intersection has two conflicting streams of traffic, and the cycle consists of a red period, an effective green period and lost time. Though all my analysis assumes one lane of traffic per approach, it could be easily generalized to handle multiple lanes.

Poisson Vehicle Arrivals

All my calculations are done with respect to an isolated intersection. In the 1988 edition of the Manual on Uniform Traffic Control Devices [USDOT 1988], it is recommended that signals within 1/2 mile of each other be coordinated. For signal separations of 1/2 mile or greater, the Poisson assumption is reasonable. In practice, not all signals within 1/2 mile of one another are coordinated, but assuming Poisson arrivals as a first-order approximation seems reasonable. The calculations performed in this chapter can be done for more general arrival processes [Gazis 1974].

Constant Vehicle Departure Times

This assumption is commonly made in the literature (Chapter Two). It implies that during a green phase, vehicles depart in such a manner that the intervals between successive departures are constant. It is reasonable to expect the departure time of the first vehicle to be greater than those of the succeeding vehicles and this can be allowed for by increasing the value of \( L \), the lost time (defined below). Another assumption is that vehicles which arrive during a green signal, when there is no queue, are not delayed.

Clearly, constant departure times are not accurate for an intersection that allows turns. If special lanes are provided for turning traffic, the measures for the turning traffic stream may be considered separately from the main stream. As a first-order approximation, I ignore the effects of turning vehicles. Insights regarding the tradeoff relationships among the performance measure are still possible assuming constant departures. For models which assume nonconstant departures see Gazis [1974], Newell [1969], Gordon and Miller [1966], Little [1961] and Darroch [1964]. Most of these authors consider the delay to the turning vehicles and not the delay to vehicles proceeding straight.

Two Conflicting Streams of Traffic

A typical intersection consists of four approaches corresponding to two-way traffic flow on the two intersecting streets. This appears to make my assumption of two
traffic streams unrealistic. It is reasonable to assume that the flows on a shared street receive common red and green intervals. When determining the green split for a street, the common green constraint forces one to focus on only one of the flows on the street. Hence when determining the optimal signals settings, I determine them for the heaviest flow for each of the two intersecting streets. For example, if the approaches are denoted by the direction from which the traffic enters the intersection, then $\lambda_1$, the flow on approach 1, is chosen to be $\lambda_1 = \max(\lambda_{\text{NORTH}}, \lambda_{\text{SOUTH}})$ and similarly, $\lambda_2 = \max(\lambda_{\text{EAST}}, \lambda_{\text{WEST}})$. In this sense, assuming two conflicting traffic streams is not unrealistic. To determine the performance measures for the entire intersection, it is possible to incorporate any number of traffic streams since each can be analyzed separately and easily combined to provide measures for the entire intersection.

**Structure of the Cycle**

A cycle is assumed to consist of a red interval, an effective green interval and lost time. The red interval is the period an approach receives the red signal. The effective green interval is the portion of the green indication which can be used by vehicles; it is the length of time an approach is awarded the green indication minus half the total intersection lost time. Lost time is the penalty for signal changes. Associated with each signal change (there are two per cycle) there is a period when the intersection is unused. One traffic stream has cleared the intersection during yellow light and is prevented from entering the intersection by the subsequent red light and the other stream has not yet begun to use the intersection. In reality, lost time at an intersection is usually the latter part of the yellow intervals plus any all-red periods. Figure 5.1 pictorially shows the relationship between the cycles for the two conflicting streams.

![Diagram](image)

**Figure 5.1** Diagram depicting the relationship between the cycles for two conflicting traffic streams.
5.2.2 Notation

In addition to the notation associated with the performance measures defined in section 4.2, I use the following conventions:

- $\lambda$ = total average arrival rate (veh/min) of vehicles
- $\lambda_i$ = average arrival rate (veh/min) of vehicles on approach $i$, $i = 1, 2$
- $R_i$ = length of the red interval (min) for approach $i$, $i = 1, 2$
- $G_i$ = length of effective green interval (min) for approach $i$, $i = 1, 2$
- $C$ = length of a cycle (min)
- $s$ = amount of time a queued vehicle requires to clear the intersection (min)
- $\pi_i$ = fraction of total effective green time ($C-L$) assigned to approach $i$, $i = 1, 2$
- $W$ = total delay per cycle (veh-min)

- $L/2$ = amount of lost time per cycle per approach

Using the above notation and the assumptions in subsection 5.2.1, it is possible to derive the following useful relationships.

\[ C = R_i + G_i + \frac{L}{2} \quad \text{for } i = 1, 2 \]  
(5.3)
\[ C = G_1 + G_2 + L \]  
(5.4)
\[ C = R_1 + R_2 \]  
(5.5)
\[ \pi_i = \frac{G_i}{C-L} \quad i = 1, 2 \]  
(5.6)
\[ \lambda = \lambda_1 + \lambda_2 \]  
(5.7)

5.2.3 Two Example Intersections

Throughout the remainder of my thesis, I refer to two "average" intersections when computing results. The vehicular arrival process for both types of intersections is assumed to be Poisson. One intersection is a lightly trafficked intersection. For this intersection, I assume the average arrival rate for approach 1 is $\lambda_1=3$ veh/min and the average arrival rate for approach 2 is $\lambda_2=5$ veh/min. For the other intersection, denoted a moderately heavy trafficked intersection, the average arrival rate for approach 1 is $\lambda_1=8$ veh/min and for approach 2, the average arrival rate is $\lambda_2=12$ veh/min.

SECTION 5.3 MODEL1: Simplest Light Traffic Model

As an introduction and first approximation to the problem, I assume that all queued vehicles instantaneously clear the intersection at the beginning of each green interval.
This implies that $s=0$ and that any vehicle which arrives during a green interval experiences zero delay. This model provides a good approximation for the key performance measures for sufficiently light traffic conditions because the bulk of the delay experienced by a vehicle occurs during a red interval.

5.3.1 Expected Delay per Randomly Arriving Vehicle

To calculate this quantity it is easiest to condition on each approach and then appropriately combine the results. Given a randomly arriving vehicle approaches the intersection from direction 1, its expected delay is $\frac{C-G_1}{2}$ if it arrives during the red or lost time interval. The probability that the vehicle arrives in a red-lost time interval is equal to the proportion of the combined intervals over the entire cycle length, $\frac{C-G_1}{C}$, and the probability that a randomly arriving vehicle approaches from direction 1 is $\frac{\lambda_1}{\lambda}$. A similar argument can be used to derive an analogous result for a vehicle approaching on approach 2. Unconditioning and simplifying yields

$$\bar{d}_1 = \frac{\lambda_1(C-G_1)^2 + \lambda_2(C-G_2)^2}{2\lambda C}.$$ \hspace{1cm} (5.8)

Rewriting $\bar{d}_1$ in terms of $\pi_1$ (use equations (5.4) and (5.6)) produces,

$$\bar{d}_1 = \frac{\lambda_1[(C-L)\pi_1]^2 + \lambda_2[(C-L)\pi_1+L]^2}{2\lambda C}.$$ \hspace{1cm} (5.9)

**Claim 5.1:** For a given cycle length $C$, the expected delay per randomly arriving vehicle, $\bar{d}_1$, is a strictly convex function in $\pi_1$.

**Proof:** Obviously, $\bar{d}_1$ is quadratic in $\pi_1$. Holding the cycle time $C$ fixed, the second derivative of $\bar{d}_1$, with respect to $\pi_1$ is

$$\frac{d^2\bar{d}_1}{d\pi_1^2} = \frac{(C-L)^2}{C} > 0$$

because $C > 0$. \hfill \Box

Given a cycle length $C$, it is possible to determine the proportion of useable cycle time for which approach 1 (and approach 2) should be allotted a green interval such that the expected delay per randomly arriving vehicle is minimized by taking the partial derivative of $\bar{d}_1$ with respect to $\pi_1$. Denote this quantity $\pi_{1,\bar{d}_1}^\ast$. Performing the relevant calculations gives

$$\pi_{1,\bar{d}_1}^\ast = \frac{\lambda_1 C - \lambda_2 L}{\lambda(C-L)}.$$ \hspace{1cm} (5.10)
When \( L = 0 \) (i.e. there is no lost time), \( \pi_1^* = \frac{\lambda_1}{\lambda} \) as one would expect. Also, as \( C \to \infty \), \( \pi_1^* \to \frac{\lambda_1}{\lambda} \). This means that as the cycle time grows larger, adjusting for the delay which occurs during the lost time becomes insignificant and again it is best to allocate each approach a green time proportional to its arrival rate.

For a given set of values for the parameters, various relationships can be explored.

As the cycle length increases, the \( \pi_1^* \) value shifts towards \( \frac{\lambda_1}{\lambda} \) as discussed (Figure C.1, Appendix C). Moreover, as cycle lengths become longer, the difference in the delay values between \( \pi_1^* \) and non-optimal \( \pi_1 \) values increases. When the cycle length is greater than 0.5 minutes, it can be seen that the delay is relatively insensitive to the allocation of green time whereas when \( C = 2 \) minutes, it is much more sensitive. The optimal proportion, \( \pi_1^* \), also varies with cycle time (Figure C.2, Appendix C). It is interesting to note that for the common range of cycle lengths, \( C = 0.5 \) to \( C = 2 \) minutes, the optimal value for \( \pi_1 \) is insensitive to changes in cycle length. Finally, the relationship between the delay per randomly arriving vehicle and \( \pi_1 \) can be explored for various different flow ratios \( r = \frac{\lambda_1}{\lambda_2} \) (Figure C.3, Appendix C).

As expected, \( \pi_1^* \) favors the approach with the higher flow rate. Furthermore, for larger magnitudes of difference between \( r \) and \( 1/2 \), \( \tilde{d}_1 \) is more sensitive to the \( \pi_1 \) value. This suggests that there is greater potential savings in \( \tilde{d}_1 \) associated with providing intersections with highly uneven flow rates with an optimal timing plan for a chosen \( C \).

Besides choosing the appropriate \( \pi_1 \), a traffic engineer must also choose a cycle length, \( C \) when determining a fixed-cycle timing plan.

**Claim 5.2:** For a given proportion of useable green time awarded to the vertical approach, \( \pi_1 \), the expected delay per randomly arriving vehicle, \( \tilde{d}_1 \) is a strictly convex function in \( C \).

**Proof:** Taking the second partial derivative of \( \tilde{d}_1 \) (5.9) produces

\[
\frac{\partial^2 \tilde{d}_1}{\partial C^2} = \frac{L^2}{\lambda C^3} \left[ \lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2 \right] > 0
\]

because \( \lambda_1 > 0 \), \( \lambda_2 > 0 \) and \( C > 0 \).

Given \( \pi_1 \), it is possible to determine the cycle length \( C_{\tilde{d}_1}^* \) which minimizes the expected delay per randomly arriving vehicle. Such a value exists because given \( \pi_1 \),
\( \bar{d}_1 \) is a convex function in \( C \). A restriction on \( C \) is that it cannot be less than the cycle lost time, \( L \). If the value of \( C \) which minimizes \( \bar{d}_1 \) is less than \( L \), then because \( \bar{d}_1 \) is convex in \( C \) for a given \( \pi_1 \), the value of \( C_{d_1}^* \) is \( L \). Doing the calculus yields

\[
C_{d_1}^* = \max \left( L, L \cdot \sqrt[3]{\frac{\lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2}{\lambda_2 \pi_1^2 + \lambda_2 (1-\pi_1)^2}} \right).
\]  

(5.11)

If \( \pi_1 = \varepsilon \), initially it seems that \( C_{d_1}^* \) should be essentially \( L \) because of the assumption that queued vehicles instantaneously clear the intersection at the beginning of each green period. When the cycle length is equal to \( L \), the signal changes as soon as the lost time has past and no vehicle is forced to wait longer than \( L \) minutes. However, as \( \pi_1 \rightarrow 0 \), equation (5.11) indicates that for \( \lambda_2 > \lambda_1 \),

\[
C_{d_1}^* \rightarrow L \sqrt[3]{\frac{\lambda_2}{\lambda_1}}
\]

which is larger than \( L \). At first this result seems surprising. Upon reflection, when \( C_{d_1}^* = L \) and \( \lambda_2 > \lambda_1 \), the expected wait per randomly arriving vehicle is \( L/2 \). It is possible to improve upon this. A longer cycle time (and the fact that \( \pi_1 = \varepsilon \)) means the approach 2 is awarded a slightly longer green time because the fraction of useable green time awarded to the approach 1 is essentially zero. Because flow is heavier in direction 2, there are more vehicles spared having to stop in a lost interval. This delay savings is greater than the penalty of holding cars on the lighter trafficked vertical approach for a slightly longer time. The same argument is applicable when \( \pi_1 = 1-\varepsilon \) and \( \lambda_1 \) is greater than \( \lambda_2 \). When \( \pi_1 = 1/2 \) then \( C_{d_1}^* = L \) as is expected. All other cases, are as easily determined.

Upon examining the relationship between the optimal cycle length and the \( \pi_1 \) values for varying ratios of flows on each approach (Figure C.4, Appendix C), it can be observed that as flow ratios approach one, the difference in the optimal cycle length and \( L \) becomes smaller. The further in magnitude the ratio is from one, the more sensitive the cycle time is to the \( \pi_1 \) values.

Because of the unrealistic assumption that cars instantaneously clear the intersection, no realistic cycle times can be obtained from this model though it seems safe to observe that shorter cycle lengths produce smaller expected delays per randomly arriving vehicles.

A final conjecture is that \( \bar{d}_1 \) is a convex function in \( C \) and \( \pi_1 \). It has been shown that holding one variable fixed, \( \bar{d}_1 \) is convex in the remaining variable but that is not proof that \( \bar{d}_1 \) is convex in \( C \) and \( \pi_1 \). For specific values of \( \lambda_1, \lambda_2 \) and \( L \), it is possible to
check if $\overline{d}_1$ is convex in $C$ and $\pi_1$. However, for the general case when the constants are not specified, the calculus and algebra become extremely cumbersome.

5.3.2 Expected Fuel Consumption and Pollution Emissions

To determine the expected total fuel consumption per minute and the expected total emissions of pollutant $i$ ($i = \text{CO, HC, NO}_{x}$) per minute, both $\overline{D}_1$ and $\overline{S}_1$ must be computed where $D$ and $S$ are defined in Section 4.2. 

Let $\overline{D}_1$ denote the expected total delay per minute. It can be expressed as the expected total delay per cycle, $\overline{W}$, times the number of cycles per minute, $1/C$. $\overline{W}$ is equal to the total delay accumulated on approach 1 per cycle plus the total delay accumulated on approach 2 per cycle. To determine the total delay on approach 1, it is easiest to condition on the number of vehicles which queue during a red-lost time interval. The expected total delay per cycle given that $i$ vehicles queue is $\frac{C-G_1}{2} \cdot i$. The probability that $i$ vehicles queue is equal to the probability of $i$ Poisson arrivals in an interval of length $C-G_1$. Putting all the pieces together produces

$$E[\text{total approach 1 delay per cycle}] = \sum_{i=0}^{\infty} \frac{C-G_1}{2} i \cdot \left[ \frac{\lambda_i(C-G_1)}{i!} \right] e^{-\lambda_i(C-G_1)}$$

$$= \frac{\lambda_i(C-G_1)^2}{2}.$$  \hspace{1cm} (5.12)

A similar argument applied to approach 2 yields

$$E[\text{total approach 2 delay per cycle}] = \frac{\lambda_2(C-G_2)^2}{2}.$$  \hspace{1cm} (5.13)

Hence,

$$\overline{W} = \frac{\lambda_1[(C-(C-L)\pi_1)^2 + \lambda_2(C-(C-L)\pi_1+L)^2]}{2}. \hspace{1cm} (5.14)$$

Expression (5.14) is consistent with the expression derived for $\overline{d}_1$, (5.9). Over a large number of cycles, it is expected that the mean delay per randomly arriving vehicle is equal to the mean total delay per cycle divided by the mean number of vehicles to arrive per cycle, $\lambda C$. A comparison of the quantities shows that they satisfy this expectation.

The expression for $\overline{D}_1$ is

$$\overline{D}_1 = \overline{W} \cdot \frac{1}{C} = \frac{\lambda_1[(C-(C-L)\pi_1)^2 + \lambda_2((C-L)\pi_1+L)^2]}{2C}.$$  \hspace{1cm} (5.15)

As for the calculation of $\overline{D}_1$, it is easier to calculate $\overline{S}_1$, the expected number of vehicles required to stop per minute, by calculating the mean number of vehicles which queue per cycle and then multiplying by the number of cycles per minute, $1/C$. Due to
the light traffic assumption, the expected number of vehicles which stop per minute is merely equal to the expected number of vehicles which must queue. Because the arrivals are Poisson, the expected number of stops per cycle is \( \lambda_1(C-G_1) + \lambda_2(C-G_2) \).

This implies that

\[
\tilde{s}_1 = \frac{\lambda_1[(C-L)\pi_1] + \lambda_2[(C-L)\pi_1 + L]}{C}.
\]

(5.16)

The expected fuel consumption can now be written as

\[
\tilde{f}_1 = \alpha \left( \frac{\lambda_1[(C-L)\pi_1]^2 + \lambda_2[(C-L)\pi_1 + L]^2}{2C} \right) + \beta \left( \frac{\lambda_1[(C-L)\pi_1] + \lambda_2[(C-L)\pi_1 + L]}{C} \right).
\]

Likewise, the expression for the expected amount of pollutant \( i \) released per minute can be written

\[
\tilde{p}_{i,1} = \gamma_i \left( \frac{\lambda_1[(C-L)\pi_1]^2 + \lambda_2[(C-L)\pi_1 + L]^2}{2C} \right) + \delta_i \left( \frac{\lambda_1[(C-L)\pi_1] + \lambda_2[(C-L)\pi_1 + L]}{C} \right)
\]

for \( i = \text{CO, HC, and NO}_x \).

(5.17)

(5.18)

Claim 5.3: For a given cycle length \( C \), the expected fuel consumption per minute, \( \tilde{f}_1 \), and the expected amount of pollutant \( i \), \( \tilde{p}_{i,1} \), released per minute \( (i = \text{CO, HC, NO}_x) \) are convex functions in \( \pi_1 \).

Proof: It is clear that, \( \tilde{f}_1 \) and \( \tilde{p}_{i,1} \) (for all \( i \)) are quadratic in \( \pi_1 \). Holding the cycle length \( C \) fixed, the second partial derivatives of \( \tilde{f}_1 \), and \( \tilde{p}_{i,1} \) (for all \( i \)) with respect to \( \pi_1 \) are

\[
\frac{\partial^2 \tilde{f}_1}{\partial \pi_1^2} = \frac{\alpha \lambda(C-L)^2}{C} > 0
\]

and

\[
\frac{\partial^2 \tilde{p}_{i,1}}{\partial \pi_1^2} = \frac{\gamma_i \lambda(C-L)^2}{C} > 0
\]

respectively. Because all the constants are positive, both second partial derivatives are positive.

For a given \( C \), the proportion of useable green time allocated to the vertical approach which minimizes \( \tilde{f}_1 \), \( \pi^*_1 \tilde{f}_1 \), is

\[
\pi^*_1 \tilde{f}_1 = \frac{\lambda_1(C-L) - \lambda_2L}{\lambda(C-L)} + \frac{\beta}{\alpha \lambda(C-L)}(\lambda_1 - \lambda_2)
\]

\[
= \pi^*_1 \tilde{f}_1 + \frac{\beta}{\alpha \lambda(C-L)}(\lambda_1 - \lambda_2).
\]

(5.19)

Equation (5.19) indicates that the proportion of green time allocated to the approach with the higher flow rate must be larger to minimize the expected fuel consumption per
minute than is required to minimize \( \bar{d}_1 \). Assume \( \lambda_1 > \lambda_2 \), then as \( \pi_{1, \bar{d}_1}^* \) is increased, approach 1 (the heavier trafficked approach) is awarded more green time resulting in fewer vehicles forced to stop per minute overall. On the other hand, approach 2 (lighter trafficked approach) accumulates more delay. The total net benefit is a savings in fuel consumption. Increasing \( \pi_1 \) from \( \pi_{1, \bar{d}_1}^* \) only decreases expected fuel consumption to a point. When \( \pi_1 \) becomes too large, the number of idling vehicles on the horizontal approach consume more fuel per minute than is saved by causing fewer vehicles overall to queue (or stop). The optimal amount to increase \( \pi_1 \) above \( \pi_{1, \bar{d}_1}^* \) is

\[
\frac{\beta}{\alpha \lambda(C-L)}(\lambda_1-\lambda_2). 
\]

An analogous argument can be applied if \( \lambda_1 < \lambda_2 \).

Clearly, because the \( \bar{p}_{i,1} \) have the same functional form as \( \bar{f}_1 \), the value of \( \pi_1 \) which produces the minimum \( \bar{p}_{i,1} \) value for \( i = \text{CO, HC, NO}_x \) is

\[
\pi^*_{1, \bar{p}_{i,1}} = \pi^*_{1, \bar{d}_1} + \frac{\delta_i}{\gamma_i \lambda(C-L)}(\lambda_1-\lambda_2). 
\]

(5.20)

for given cycle length \( C \).

Appendix C contains a similar analysis for the fuel consumption measure as was done for the expected delay measure in the previous subsection. In general, all the observations regarding the relationships among \( \bar{f}_1 \), the cycle length, and \( \pi_1 \) are the same as for the expected delay measure.

Given \( \pi_1, \bar{f}_1 \) and \( \bar{p}_{i,1} \) are also strictly convex in \( C \).

**Claim 5.4:** For a given proportion of usable green time awarded to the vertical approach, \( \pi_1, \bar{f}_1, \) and, \( \bar{p}_{i,1} \), \( (i=\text{CO, HC, NO}_x) \) are strictly convex functions in \( C \).

**Proof:** Taking the second partial derivative of \( \bar{f}_1 \) and \( \bar{p}_{i,1} \) (for \( i = \text{CO, HC and NO}_x \)) yields

\[
\frac{\partial^2 \bar{f}_1}{\partial C^2} = \frac{\alpha L^2}{C^3}[\lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2] + \frac{2 \beta L}{C^3}[\lambda_1 \pi_1 + \lambda_2 (1-\pi_1)] > 0
\]

and

\[
\frac{\partial^2 \bar{p}_{i,1}}{\partial C^2} = \frac{\gamma_i L^2}{C^3}[\lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2] + \frac{2 \delta_i L}{C^3}[\lambda_1 \pi_1 + \lambda_2 (1-\pi_1)] > 0
\]

for \( i = \text{CO, HC, NO}_x \), respectively. Because all the constants are positive, both second partial derivatives are positive. \( \square \)
Given $\pi_1$, it is possible to determine the cycle lengths $C_{\tilde{d}_1}^*$ and $C_{\tilde{p}_{l,1}}^*$ which minimizes $\tilde{d}_1$ and $\tilde{p}_{l,1}$, respectively. After some calculations, these values are the maximum of zero or
\[
C_{\tilde{d}_1}^* = \sqrt{\left(C_{\tilde{d}_1}^* \right)^2 + \frac{2\beta L}{\alpha} \frac{\lambda_1 \pi_1 + \lambda_2 (1-\pi_1)}{\lambda_2 \pi_1^2 + \lambda_1 (1-\pi_1)^2}} \text{,} \tag{5.21}
\]
and
\[
C_{\tilde{p}_{l,1}}^* = \sqrt{\left(C_{\tilde{p}_{l,1}}^* \right)^2 + \frac{2\gamma L}{\gamma_i} \frac{\lambda_1 \pi_1 + \lambda_2 (1-\pi_1)}{\lambda_2 \pi_1^2 + \lambda_1 (1-\pi_1)^2}} \text{,} \tag{5.22}
\]
subject to the constraint that $C_{\tilde{d}_1}^* > L$ and $C_{\tilde{p}_{l,1}}^* > L$. One thing to note from (5.21) and (5.22) is that the cycle length which minimizes $\tilde{d}_1$ and $\tilde{p}_{l,1}$ is always greater than that which minimizes $\tilde{d}_1$. Also, for $\pi_1$ values which differ significantly from 0.5, the cycle length increases and the optimal cycle lengths are largest for uneven flow rates (Figure C.8, Appendix C).

5.3.3 Pareto Optimal Set

Throughout the analysis contained in this section, I keep the cycle length $C$ fixed and examine the set of performance measure vectors in the Pareto optimal set $\mathcal{T}_1$. $\mathcal{T}_1$ contains vectors of the form $[\tilde{d}_1, \tilde{f}_1, \tilde{p}_{c_{0,1}}, \tilde{p}_{h_{c,1}}, \tilde{p}_{n_{o,1}}]$. Corresponding to each vector in $\mathcal{T}_1$ there is an unique value of $\pi_1$ so it is possible to speak of a Pareto optimal set in terms of the $\pi_1$ values. Denote $\mathcal{P}_1^*$ as the set of $\pi_1$ values which correspond to the set of Pareto optimal performance vectors, $\mathcal{T}_1$.

**Theorem 5.5:** The $\pi_1$ values (where $0 \leq \pi_1 \leq 1$) associated with the Pareto optimal set for $\tilde{d}_1, \tilde{f}_1, \tilde{p}_{c_{0,1}}, \tilde{p}_{h_{c,1}}, \tilde{p}_{n_{o,1}}$ are:

(i) $\mathcal{P}_1^* = (\pi_1, \tilde{d}_1, \pi_1, \tilde{p}_{n_{o,1}})$ if $\lambda_1 > \lambda_2$;

(ii) $\mathcal{P}_1^* = (\pi_1, \tilde{p}_{n_{o,1}}, \pi_1, \tilde{d}_1)$ if $\lambda_1 < \lambda_2$;

(iii) $\mathcal{P}_1^* = \pi_1, \tilde{d}_1$ if $\lambda_2 = \lambda_1$.

**Proof:**

**Case (i):** $\lambda_1 > \lambda_2$

From equations (5.19) and (5.20),
\[
\pi_1^* = \pi_1^* + \frac{\beta}{\alpha \lambda (C-L)} (\lambda_1 - \lambda_2)
\]
and
\[
\pi_1^* = \pi_1^* + \frac{\delta_{cd}}{\gamma_{co} \lambda (C-L)} (\lambda_1 - \lambda_2)
\]
It is clear that $\pi_i^* > \pi_i^*$, for $i = \text{CO, HC and NO}_X$, and $\pi_i^* > \pi_i^*$ because $\lambda_1 > \lambda_2$ and all the constants are positive. This implies that all the performance measures are decreasing as $\pi_1$ increases from 0 to $\pi_1^*$. Hence all points associated with $\pi_1 \epsilon(0, \pi_1^*)$ are dominated by the vector of performance measures associated with $\pi_1^*$. Therefore points corresponding to $\pi_1 \epsilon(0, \pi_1^*)$ are not in the Pareto optimal set.

The values for $\pi_i^*$, for $i = \text{CO, HC, and NO}_X$, and $\pi_i^*$ differ only in the coefficient of the second term. Because $\frac{\delta_{\text{NO}_X}}{\delta_{\text{NO}_X}}>\frac{\delta_{\text{CO}}}{\delta_{\text{HC}}}>\frac{\delta_{\text{CO}}}{\delta_{\text{HC}}}$ (coefficient values are in Section 4.5), $\pi_1^* = \max(\pi_{\text{CO}}, \pi_{\text{HC}}, \pi_{\text{NO}_X})$. For values of $\pi_1 \epsilon(\pi_1^*, 1)$, all three performance measures are greater than those associated with $\pi_1^*$. Hence the vector of performance measures associated with $\pi_1^*$ dominates all those associated with $\pi_1 \epsilon(\pi_1^*, 1)$.

Finally, for any $\pi_i \epsilon[\pi_i^*, \pi_i^*]$, there exists another $\pi_i^* \epsilon[\pi_i^*, \pi_i^*] (\pi_i^* \neq \pi_i^*)$ such that at least one of $d_i, f_i, p_{\text{CO}}, p_{\text{HC}}, p_{\text{NO}_X}$ is worse and at least one is better. For any chosen point $\pi_i$ choose $\pi_i^* = p_{\text{CO}}^*$ (if $p_{\text{CO}} = p_{\text{NO}_X}$, then choose $\pi_i^* = d_i^*$ and use a similar argument). In this case $p_i^* \text{ CO}$ is better than $p_i^* \text{ CO}$ and either $d_i^*$ is worse, $d_i^*$ is worse or they both are. Therefore, all values in $[\pi_i^*, \pi_i^*]$ correspond to vectors in the Pareto optimal set.

Case (ii): $\lambda_1 < \lambda_2$
Proof is analogous to Case (i).

Case (iii): $\lambda_2 = \lambda_1$

In this case, $\pi_i^* = p_{\text{CO}}^* = p_{\text{HC}}^* = p_{\text{NO}_X}^* = p_{\text{HC}}^*$. Hence there is one value of $\pi_1$ which minimizes $d_1, f_1, p_{\text{CO}}, p_{\text{HC}}, p_{\text{NO}_X}$, and $p_{\text{NO}_X}$ simultaneously. By definition, this point is in the Pareto optimal set. It is the only such point because given a cycle length $C$, all three functions are strictly convex in $\pi_1$ which implies they each have only one minimum value.

For a given cycle length $C$, the functions $d_1, f_1, p_{\text{CO}}, p_{\text{HC}}, p_{\text{NO}_X}$ have been rescaled and drawn of the same graph (Figure 5.2) assuming the same values for the
arrival rates and lost time as used previously in Appendix C for $\bar{d}_1$ and $\bar{f}_1$. The corresponding Pareto optimal set is $\mathcal{P}_1 = (0.0, 0.3571)$. The Pareto frontier for $\bar{d}_1$ and $\bar{f}_1$ is presented in Figure 5.3. The Pareto optimal sets for some other pairs of measures are found in Appendix C.

![Graph showing Pareto optimal sets for various measures]

Figure 5.2  Rescaled performance measures (MODEL1) as a function of the $\pi_1$ values and the corresponding Pareto optimal set.

5.3.4 An Example Analysis Using the Pareto Optimal Set of Performance Measures

The following is one type of analysis which can be performed when the Pareto optimal set of the performances measures is known. The analysis is applied to the lightly trafficked intersection. It is assumed that the fixed-cycle signal operates with a cycle length of $C=1$ minute. The lost time of the intersection is 4 seconds (2 seconds per each signal change).

In a national survey of urban traffic engineers (see Section 3.4), 85% of the respondents indicated that they explicitly use a delay measure when developing signal timing plans, 19% use a fuel consumption measure, and only 6% indicated they use a
pollutant emissions measure. From the survey results, it is obvious that of the key
performance measures, the two that traffic engineers are primarily interested in are 1) 
a measure of delay and 2) a measure of fuel consumption. Given this, I focus on these
two measures.

![Graph showing the relationship between E[delay per veh] in minutes and E[fuel consumption rate] in gal/min.]

Figure 5.3 Efficient frontier for the delay and fuel performance measures (MODEL1).

Each performance measure, \( \bar{d}_1 \), \( \bar{f}_1 \) and \( \bar{p}_{CO,1} \), is evaluated for the \( \pi_1 \) values which 
minimizes one of them (the CO values are included for illustrative purposes). The 
results are summarized in Table 5.1.

<table>
<thead>
<tr>
<th>( \pi_1 )</th>
<th>( \bar{d}_1 )</th>
<th>( \bar{f}_1 )</th>
<th>( \bar{p}_{CO,1} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \pi_1^*_{d_1} = 0.3571 )</td>
<td>0.1333 min = 8.00 sec</td>
<td>0.0337 gal/min = 2.02 gal/hr</td>
<td>27.206 g/min = 1632.4 g/hr</td>
</tr>
<tr>
<td>( \pi_1^*_{f_1} = 0.0765 )</td>
<td>0.1665 min = 9.99 sec</td>
<td>0.0320 gal/min = 1.92 gal/hr</td>
<td>28.087 g/min = 1685.2 g/hr</td>
</tr>
<tr>
<td>( \pi_1^*<em>{p</em>{CO,1}} = 0.2611 )</td>
<td>0.1374 min = 8.241 sec</td>
<td>0.0327 gal/min = 1.96 gal/hr</td>
<td>26.857 g/min = 1611.4 g/hr</td>
</tr>
</tbody>
</table>

Table 5.1 Summary of the MODEL1 performance measure values for each \( \pi_1 \) value which 
minimizes one measure.
The quantities in Table 5.1 assume that there are only two incoming streams of traffic. Because I have chosen the heaviest flows on each street of the intersection, the total performance values for an intersection with four incoming traffic streams would be bounded by the values in Table 5.1 and twice those values. For a specific intersection with known flow rates for all four streams, the value of the performance measures could be computed directly.

If the signal were timed to minimize $\overline{d}_i$ (the upper left corner of Figure 5.3), which corresponds to the common practice today, the average delay per randomly arriving vehicle is approximately 8 seconds (Table 5.1). This implies an expected total delay rate of 64 min/hr because the combined arrival rates for intersection is 8 veh/min. The corresponding expected fuel consumption rate is 2.02 gal/hr. On the other hand if the signal setting corresponded to the lower right endpoint of the curve, the setting which minimizes $\overline{f}_i$, the expected total delay rate is 79.2 min/hr and the expected fuel consumption rate is 1.92 gal/h.

What is required to make a signal timing plan corresponding to a point further down on the Pareto optimal frontier of $\overline{f}_i$ and $\overline{d}_i$ more attractive to society? A way to answer this is to analyze the differences among the two performance measures for the point on the curve corresponding to current practice, which maps to $\pi^*_1, \overline{d}_i$, and for a point within a radius $\delta$ ($\delta$ close to zero) of the left endpoint which maps to $\overline{\omega}_1, \overline{d}_i = \pi^*_1, \overline{d}_i + \varepsilon$ ($\varepsilon$ close to zero). At the point which corresponds to $\overline{\omega}_1, \overline{d}_i$, there exist an unique tangent line to the efficient frontier with an approximate slope of $-31$. This indicates that to remain at or near the left endpoint, society is willing to sacrifice an average of 31 gal/min of fuel for a savings of one minute of expected delay per randomly arriving vehicle. This translates to an average of 1,860 gal/hr of gas are worth less than 480 total minutes of delay per hour.

If these quantities are measured in monetary units, this implies the dollar value of a minute of person-delay is greater than 3.875 times the price of one gallon of gasoline. Today, the average income of a single person household is approximately $17,500 [US Statistical Abstract 1990]. Using this figure to evaluate an individual's time suggests an hour is worth approximately $8.45 (assuming 260 eight-hour workdays per year). This value is comparable to $8.20 per hour used in a study of the impact of declining mobility in major cities performed by the Texas Transportation Institute in 1986 [TTI 1988]. Currently, a gallon of gas costs approximately $1.25. Thus to remain at the left
endpoint of the efficiency frontier implies that $(1.25 \text{ per gal})(1860 \text{ gal/hr}) = 2325 \text{ per hour}$ is worth less than $(8.45 \text{ per hour})(8 \text{ vch-hour/h})(1.25 \text{ people/veh} [\text{TTI 1988}]) = 84.50 \text{ per hour}$. Clearly, it does not make sense to remain at the left endpoint from a monetary perspective.

The policy of remaining at the left endpoint probably makes even less sense than indicated by the above example. Evaluating people's time at an average hourly wage rate may not be accurate. Obviously, individual's place a value on their time but what is this value? Although results of statistical studies vary, Morrison [1986] reports that individuals appear to value in-vehicle travel time at not more than one-half their hourly wage rate. The American Association of State and Transportation Officials [Morrison 1986] recommends that in-vehicle travel savings be valued at $0.21 per hour for individual savings less than five minutes, $1.80 per hour for individual savings between 5 and 15 minutes, and $3.90 per hour for individual savings in excess of 15 minutes (all expressed in 1975 dollars). The equivalent values for today are approximately $0.48, $4.14 and $8.97, respectively. These studies suggest that an hour of person-delay is worth less than $8.45. In addition to the issue of overpricing delay time, there are issues concerning oil supplies and gas prices: (1) the uncertainty associated with gas prices and supplies due to instabilities in the oil producing countries of the Middle East, and (2) claims that the real price of gas is too low [Hubbard 1991]. All these factors strongly suggest that from a monetary value perspective, it would be irrational for society to remain at the left endpoint. Unfortunately, the analysis is only valid for a small region around the left endpoint and so does not indicate which point further down the Pareto optimal curve would be more appropriate.

An analogous examination of the other endpoint reveals that to remain at the right endpoint, society considers one gal/min of fuel worth more than a savings of 1000 minutes of delay per randomly arriving vehicle or 60 gal/h of gas are worth more than 480,000 total minutes of delay per hour. Clearly from a fiscal point of view (using a cost of $8.45 per hour of delay and assuming average vehicle occupancy of 1.25), this point is not attractive and would only become attractive if gas were $1408 per gallon! (An event which hopefully will not occur.) If it were assumed that people's in-vehicle delay time is no more than half their hourly rate, the right endpoint would only become attractive if gas cost $704 per gallon. Finally, since the delay at the intersection is clearly less than five minutes per passenger the $0.48 per hour rate might be more
suitable and then the right endpoint would become attractive only if gas cost $80 per gallon. Clearly, the right endpoint is also not attractive under any reasonable pricing of an individual's in-vehicle delay time.

It is possible to examine selected points along the curve in this manner until one is found such that the monetary tradeoff between the fuel measure and delay measure is approximately equal. This point coincides with the signal timing plan, given a cycle length of one minute, which is the best for this particular intersection among all the points in the Pareto optimal set from a dollars and cents angle.

Up to this point, I have ignored the pollution measure. One could extend the preceding monetary analysis to include the pollution measures. Using the same approach, it is possible to discover the point in the Pareto optimal set at which the fiscal tradeoffs among all measures are equal. A major difficulty of this approach lies in establishing the price of a unit of air pollution. It is hard to assess the cost of the long-term health effects of air pollution. Even more difficult and controversial is the task of quantifying the long-term harmful effects air pollution has on the environment.

This last point raises the question of whether a monetary evaluation of the relative differences among the performance measures for different Pareto optimal plans is the most appropriate for selecting a "best" plan. There are other attributes associated with each of the performance measures in addition to cost. For example, associated with delay is motorist frustration, and for pollution emissions, there is the possible consequence that if the global warming trend remains unchecked future human existence could be in jeopardy. Also, associated with fuel consumption is the attribute of the United States' dependence on petroleum imports (approximately one-half of our petroleum is imported [API 1991]) which has had a negative impact on our economy in the past. These other attributes suggest the use of a multivariate utility analysis.

SECTION 5.4 MODEL2: More Realistic Light Traffic Model

This model differs from MODEL1 because it is assumed that $s>0$. Specifically, when queued vehicles are free to depart from the intersection, the times between successive departures is a positive constant. Vehicles which arrive in a green period when there is no queue are not delayed, as usual.
5.4.1 Additional Notation

In addition to previously defined quantities, I make use of the following quantities.

\[ Q(t) \equiv \text{the number of vehicles in the traffic queue at time } t \text{ after the commencement of the combined red-lost time period for a given approach.} \]

\[ A(t) \equiv \text{the number of vehicles which arrive in the time period } (0,t) \text{ where 0 is the commencement of the combined red-lost time period for a given approach.} \]

\[ \lambda \equiv \text{average arrival rate for the approach under consideration.} \]

\[ W \equiv \text{total delay per cycle for the approach under consideration.} \]

\[ R \equiv \text{the length of the combined red time and lost time interval (red-lost time interval) for the approach under consideration.} \]

\[ G \equiv \text{the length of the effective green for the approach under consideration.} \]

The arrival process is still assumed to be Poisson. This implies that \( E[A(t)] = \lambda t \).

5.4.2 Expected Total Delay per Cycle for an Approach

To do an analysis similar to the one in section 5.3, it is easiest to compute the expected total delay per cycle and then to derive the performance measures from this quantity. The methodology I use was developed by McNeil in 1968 [McNeil 1968]. The analysis is done for one approach and the result is used to compute the performance measures for two conflicting streams of traffic (easily extended to any number of conflicting traffic streams).

For a cycle length \( C \), the total delay to those vehicles at the signal during the interval \( (t, t+\Delta t) \) is \( Q(t) \cdot \Delta t + o(\Delta t) \). This implies the total delay incurred during a cycle on a specified approach is

\[
W = \int_0^C Q(t) dt
\]

Let \( W_R \) be the delay incurred during the combined red and lost time period, \( (0,R) \), for the approach and \( W_G \) be the delay incurred during the effective green period, \( (R,C) \). Then \( W \) can be expressed as

\[
W = W_R + W_G \quad (5.23)
\]

\( W_R \) can be expressed as the delay to those vehicles which were in the queue at the start of the red-lost time interval plus the delay to those vehicles which arrived after the start of this interval and before the beginning of the effective green,

\[
W_R = \int_0^R [Q(0) + A(t)] dt
\]
Because the expectations are finite,
\[ E[W_R] = \int_0^R E[Q(0)] \, dt + \int_0^R \lambda_t \, dt \]
and so,
\[ E[W_R] = RE[Q(0)] + \frac{1}{2} \lambda R^2. \tag{5.24} \]

To determine \( E[W_G] \), consider a queueing process \( X(t) \) which has Poisson arrivals, constant service times \( s \) and \( X(0) = Q(R) \). This is an M/D/1 queueing system which starts with \( Q(R) \) vehicles in queue, the number of queued vehicles at the end of a red signal. Now let \( \hat{W}_G \) denote the total delay in the busy period which starts with \( Q(R) \) vehicles in queue and determine \( E[\hat{W}_G] \). Note that \( \hat{W}_G \) is equal to \( W_G \) when the cycle length is infinite, \( C = \infty \).

For the queueing process \( X(t) \), let
\[
A_1 \equiv \text{number of arrivals in the interval } (R, R+sQ(R)) \\
A_2 \equiv \text{number of arrivals in the interval } (R+sQ(R), R+s[Q(R)+A_1]) \\
A_3 \equiv \text{number of arrivals in the interval } (R+s[Q(R)+A_1], R+s[Q(R)+A_1+A_2]) \\
\text{and in general,} \\
A_n \equiv \text{number of arrivals in the interval } (R+s[Q(R)+A_1+\ldots+A_{n-2}], R+s[Q(R)+A_1+\ldots+A_{n-1}]) \quad \text{for } n = 3, 4, 5, \ldots.
\]

This defines a sequence of random variables \( A_1, A_2, \ldots \) recursively such that the conditional distribution of \( A_n \) given \( Q(R) \), \( A_1, A_2, \ldots, A_{n-1} \) only depends on \( A_{n-1} \).

Let
\[
\Gamma_0 = R + sQ(R) \quad \text{and} \\
\Gamma_n = \Gamma_0 + s[A_1+\ldots+A_n] \quad \text{for } n = 1, 2, 3, \ldots.
\]

Now \( \hat{W}_G \) can be rewritten as
\[ \hat{W}_G = \int_R^{R+sQ(R)} X(t) \, dt + \sum_{n=0}^{\Gamma_n} \int_0^{\Gamma_n} X(t) \, dt. \tag{5.25} \]

From basic probability,
\[ E\left[ \int_R^{R+sQ(R)} X(t) \, dt \right] = E\left( E\left[ \int_R^{R+sQ(R)} X(t) \, dt \mid Q(R) \right] \right). \]

The integral on the right-hand side can be split into two pieces. One piece represents the delay to the \( Q(R) \) vehicles in queue at time \( R \) and the other piece represents the delay to those vehicles which arrive in the interval \( (R, R+sQ(R)) \). For \( t \epsilon (R, R+sQ(R)) \), the number of vehicles that arrive in \( (R, t) \) can be expressed as \( X(t) - X(R) \). Upon breaking the integral into two pieces and simplifying:
\[
\mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_n+1} X(t) \, dt \right] = \mathbb{E} \left[ \frac{sA_n+1(A_{n+1}+1)}{2} + \mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_n+1} [A(t) - A(\Gamma_n)] \, dt \right | A_{n+1} \right]
\]

\[
= \mathbb{E} \left[ \frac{sA_n+1(A_{n+1}+1)}{2} + \mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_n+1} [\lambda t - \lambda \Gamma_n] \, dt \right | A_{n+1} \right]
\]

\[
= \frac{s}{2} \mathbb{E} \left[ A_{n+1} + A_n^2 + (1+\rho) \right]
\] (5.28)

Because \( A_{n+1} \) only depends upon \( A_n \), condition (5.28) upon \( A_n \):

\[
\mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_n+1} X(t) \, dt \right] = \frac{s}{2} \mathbb{E} \left[ \mathbb{E} \left[ A_n + A_{n+1}^2 + (1+\rho) \right | A_n \right]
\] (5.29)

Calculate \( \mathbb{E} [A_{n+1} | A_n] \) and \( \mathbb{E} [A_{n+1}^2 | A_n] \):

\[
\mathbb{E} [A_{n+1} | A_n] = \lambda s A_n = \rho A_n \quad ,
\] (5.30)

\[
\mathbb{E} [A_{n+1}^2 | A_n] = (\mathbb{E} [A_{n+1} | A_n])^2 + \text{VAR} (A_{n+1} | A_n)
\]

\[
= (\lambda s A_n)^2 + \lambda s A_n
\]

\[
= (\rho A_n)^2 + \rho A_n \quad .
\] (5.31)

Hence, upon substitution of (5.30) and (5.31) into (5.29),
\[
\mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_{n+1}} X(t) \, dt \right] = \frac{s}{2} \mathbb{E} \left[ \rho A_1 + (1+\rho) \rho \rho^{n+1} A_1 + \rho A_1 + A_1 \right].
\]

Continuing this process of conditioning and simplifying,
\[
\mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_{n+1}} X(t) \, dt \right] = \frac{s}{2} \mathbb{E} \left[ \rho^n A_1 + (1+\rho) \rho^n A_1 + \rho^{n+1} A_1 + \cdots + \rho A_1 + A_1 \right].
\] (5.32)

Finally, conditioning (5.32) on \( Q(R) \) where
\[
\mathbb{E}[A_1 \mid Q(R)] = \rho Q(R), \quad \text{and} \quad \mathbb{E}[A_1^2 \mid Q(R)] = \rho^2 Q^2(R) + \rho Q(R)
\]
produces
\[
\mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_{n+1}} X(t) \, dt \right] = \frac{s}{2} \mathbb{E} \left[ \rho^{n+1} Q(R) + (1+\rho) \rho^{n+1} \left( \rho^{n+1} Q^2(R) + \rho^n Q(R) + \cdots + \rho Q(R) + Q(R) \right) \right]
\]
\[
= \frac{s}{2} \left( \rho^{n+1} \left[ \frac{(1+\rho)(1-\rho^{n+1})}{1-\rho} \right] \mathbb{E}[Q(R)] + (1+\rho) \rho^{2n+2} \mathbb{E}[Q^2(R)] \right). \quad (5.33)
\]

Remembering that \( \mathbb{E}[\hat{W}_G] = \mathbb{E} \left[ \int_R^{R+s Q(R)} X(t) \, dt \right] + \sum_{n=0}^\infty \mathbb{E} \left[ \int_{\Gamma_n}^{\Gamma_{n+1}} X(t) \, dt \right] \) and substituting (5.26) for the first term and (5.33) for the second term and performing the summation yields
\[
\mathbb{E}[\hat{W}_G] = \frac{s}{2(1-\rho)^2} \left( \mathbb{E}[Q(R)] + (1-\rho) \mathbb{E}[Q^2(R)] \right). \quad (5.34)
\]

The remaining step is to obtain an expression for \( \mathbb{E}[W_G] \). Assume that the queue is in statistical equilibrium. A necessary and sufficient condition for this is that the average number of arrivals per cycle be less than the average number of vehicles which can be serviced in a green period,
\[
\lambda C < \frac{G}{s} \quad \text{or} \quad \rho < \frac{G}{C}. \quad (5.35)
\]

\( \mathbb{E}[W_G] \) can be expressed as
\[
\mathbb{E}[W_G] = \mathbb{E}[\hat{W}_G \mid X(0) = Q(R)] - \mathbb{E}[\hat{W}_G \mid X(0) = Q(C)]
\]
\[
= \frac{s}{2(1-\rho)^2} \left( \mathbb{E}[Q(R)] - \mathbb{E}[Q(C)] \right) + \frac{s}{2(1-\rho)} \left( \mathbb{E}[Q^2(R)] - \mathbb{E}[Q^2(C)] \right). \quad (5.36)
\]

In steady-state the expected number of vehicles in queue at the beginning of a red interval is equal to the expected number in queue at the end of a green interval. Also, the second moments of these quantities are equal. The above statements can be expressed as
\[
\mathbb{E}[Q(0)] = \mathbb{E}[Q(C)], \quad (5.37)
\]
and
\[ E[Q^2(0)] = E[Q^2(C)] . \tag{5.38} \]

It is also true that the number of vehicles in queue at the end of a red period must equal the number in queue at the beginning of a red period plus the number that arrived during the red period,
\[ Q(R) = Q(0) + A(R) . \tag{5.39} \]

Using equations (5.37) - (5.39) it is possible to derive
\[ E[Q(R) - Q(C)] = E[Q(R) - Q(0)] = E[A(R)] = \lambda R , \tag{5.40} \]

and
\[ E[Q^2(R) - Q^2(C)] = 2E[Q(0)]E[A(R)] + E[A^2(R)] = 2\lambda R E[Q(0)] + (\lambda R)^2 + \lambda R . \tag{5.41} \]

Finally substitution of (5.40) and (5.41) into (5.36) yields
\[ E[W] = \frac{s\lambda R}{2(1-\rho)^2} \left[ 1 + (1-\rho) \left( 2E[Q(0)] + \lambda R + 1 \right) \right] . \tag{5.42} \]

Now it is possible to write down an expression for \( E[W] \) using (5.24) and (5.42).
\[ E[W] = \frac{\lambda R}{2(1-\rho)} \left[ \frac{1}{\lambda} E[Q(0)] + R + \left( \frac{2-\rho}{1-\rho} \right) \right] . \tag{5.43} \]

If the traffic is sufficiently light, the term involving \( E[Q(0)] \) may be neglected since it is unlikely that any vehicle will remain in queue at the end of the green period. For values of \( r < 1 \), it has been suggested [Miller 1963], that \( E[Q(0)] \) is not much different from zero. In the remainder of this section, I assume that \( r < \frac{C}{2C} \). MODEL3 of the next section handles the case when \( E[Q(0)] \) is significantly different from zero. When \( E[Q(0)] \) can be neglected (5.43) becomes
\[ E[W] = \frac{\lambda R}{2(1-\rho)} \left[ R + \frac{2-\rho}{1-\rho} \right] . \]

Using the fact that \( R \) is the length of the combined red period and lost period for an approach and using (5.3)
\[ E[W] = \frac{\lambda(C-G)}{2(1-\rho)} \left[ C-G + \frac{2-\rho}{1-\rho} \right] . \tag{5.44} \]

From this point on, the calculations concerning the expected delay per randomly arriving vehicle, the expected fuel consumption rate and the expected CO emissions rate can be found in Appendix C. The remainder of this section concerns the results of those calculations and how they differ from MODEL1.
5.4.3 Expected Delay per Randomly Arriving Vehicle

Using equation (5.44), it is possible to derive $\bar{d}_2$, the expected delay per randomly arriving vehicle. The expression for $\bar{d}_2$ (from Appendix C) is

$$\bar{d}_2 = \frac{\lambda_1(C-(C-L)p_1)}{2\lambda C(1-p_1)} + \frac{\lambda_2((C-L)p_1+L)}{2\lambda C(1-p_2)}$$

(5.45)

As a check on (5.45), when $s=0$, $\bar{d}_2 = \frac{\lambda_1(C-(C-L)p_1)^2 + \lambda_2((C-L)p_1+L)^2}{2\lambda C}$ is equal to $\bar{d}_1$ (5.9) as it should be.

From Appendix C, it is known that given $\pi_1$, $\bar{d}_2$ is strictly convex in $C$ and similarly, given $C$, $\bar{d}_2$ is strictly convex in $\pi_1$. For a given cycle length $C$, the value of $\pi_1$ which minimizes $\bar{d}_2$, $\pi_1^*, \bar{d}_2$, is

$$\pi_1^*, \bar{d}_2 = \frac{C\lambda_1 - \lambda_2 L}{(C-L)\lambda_1} + \frac{s}{2(C-L)} \cdot \frac{\lambda_1'(1-p_2)(2-p_1) - \lambda_2'(1-p_1)(2-p_2)}{\lambda_1(1-p_1)(1-p_2)}$$

where

$$\lambda_1' = \lambda_1(1-p_2)$$

$$\lambda_2' = \lambda_2(1-p_1)$$

and

$$\lambda' = \lambda_1(1-p_2) + \lambda_2(1-p_1)$$

(5.46)

For a given $\pi_1$, $C^*_d$, the cycle length which minimizes $\bar{d}_2$ is

$$C^*_d = \sqrt{L^2 \cdot \frac{\lambda_1'(1-p_2)^2}{\lambda_2'^2 + \lambda_1'(1-p_1)^2} + \frac{Ls}{\lambda_2'(1-p_2)} \cdot \frac{\lambda_1(1-p_2)(2-p_1) + \lambda_2(1-p_1)(2-p_2)}{\lambda_2'^2 + \lambda_1'(1-p_1)^2}}$$

where

$$\lambda_1' = \lambda_1(1-p_2)$$

and

$$\lambda_2' = \lambda_2(1-p_1)$$

(5.47)

The analyses of (1) the relationship between the optimal $\pi_1$ values and the cycle length, $C$, (2) the effect that different cycle lengths have on $\bar{d}_2$, and (3) the relationship between $\bar{d}_1$ and $\bar{d}_2$ for a given cycle length, are found in Appendix C.

The other parameter of concern is the cycle length, $C$. Once more, it is conjectured that the expected delay per randomly vehicle is convex in $C$ and $\pi_1$ but is not proved. Again a brief analysis of the optimal cycle length is found in Appendix C.

5.4.4 Expected Fuel Consumption and Pollution Emissions

Derivations of the expressions and proofs for the results in this subsection are in Appendix C. Again they are similar to the derivations and proofs for the matching quantities in subsection 5.3.3. One exception is the calculation of the expected number of stops, $\bar{S}$. For this model, one has to consider the vehicles which arrive
during a green but must stop because the initial queue has not yet cleared the intersection.

The expressions for the expected fuel consumption, $\tilde{f}_2$, and the expected emission rates for pollutant $i$, $i = \text{CO}, \text{HC}$ and $\text{NO}_x$, $\bar{p}_{i2}$ are given below.

$$\tilde{f}_2 = \frac{\alpha}{2C} \left[ \frac{\lambda_1 [(C-L)\pi_1]}{(1-\rho_1)} \right] \left[ C(C-L)\pi_1 + \frac{2(2-\rho_1)}{(1-\rho_1)} \right] + \frac{\lambda_2[(C-L)\pi_1+L]}{(1-\rho_2)} \left[ (C-L)\pi_1+L+s \frac{2-\rho_2}{(1-\rho_2)} \right]$$

$$\bar{p}_{i2} = \frac{\gamma_i}{2C} \left[ \frac{\lambda_i [(C-L)\pi_1]}{(1-\rho_1)} \right] \left[ C(C-L)\pi_1 + \frac{2(2-\rho_1)}{(1-\rho_1)} \right] + \frac{\lambda_2[(C-L)\pi_1+L]}{(1-\rho_2)} \left[ (C-L)\pi_1+L+s \frac{2-\rho_2}{(1-\rho_2)} \right]$$

$$+ \frac{\delta_i}{\gamma_i (C-L)\pi_1} \left[ \frac{\lambda_i [(C-L)\pi_1]}{(1-\rho_1)} \right] + \frac{\lambda_2[(C-L)\pi_1+L]}{(1-\rho_2)} \left[ (C-L)\pi_1+L+s \frac{2-\rho_2}{(1-\rho_2)} \right]$$

(5.48)  

(5.49)

Given a cycle length, both $\tilde{f}_2$ and $\bar{p}_{i2}$, $i = \text{CO}, \text{HC}$, and $\text{NO}_x$, are convex in $\pi_1$ (Appendix C). The values of $\pi_1$ which minimize each of them respectively are

$$\pi_1^*\tilde{f}_2 = \pi_1^*d_2 + \frac{\beta}{\alpha(C-L)\lambda_1} \left( \lambda_1' - \lambda_2' \right)$$

(5.50)  

$$\pi_1^*\bar{p}_{i2} = \pi_1^*d_2 + \frac{\delta_i}{\gamma_i (C-L)\lambda_1} \left( \lambda_1' - \lambda_2' \right)$$

(5.51)

where

$$\lambda_1' = \lambda_i(1-\rho_2)$$

$$\lambda_2 = \lambda_2(1-\rho_1)$$

and

$$\lambda' = \lambda_2' + \lambda_1'$$

As for MODEL1, to minimize $\tilde{f}_2$ or $\bar{p}_{i2}$, $i = \text{CO}, \text{HC}$ and $\text{NO}_x$, the optimal fraction of useable green time is longer for the heavier trafficked approach than when minimizing $d_2$. The functional form of the equations (5.50) and (5.51) correspond to (5.19) and (5.20). A graph depicting the relationship between $\tilde{f}_1$ and $\tilde{f}_2$ can be found in Appendix C.

For a fixed proportion of useable green time allocated to the vertical direction, $\pi_1$, both $\tilde{f}_2$ and $\bar{p}_{i2}$, $i = \text{CO}, \text{HC}$ and $\text{NO}_x$, are convex in $C$. The values of $C$ which minimizes each respectively are

$$C_{\tilde{f}_2}^* = \sqrt{\left( C_{d_2}^* \right)^2 + \frac{2\beta L}{\alpha} \frac{\lambda_1/(1-\pi_1) + \lambda_2(1-\pi)}{\lambda_2/(1-\pi_1) + \lambda_1'(1-\pi)}}$$

(5.52)  

$$C_{\bar{p}_{i2}}^* = \sqrt{\left( C_{d_2}^* \right)^2 + \frac{2\delta_i L}{\gamma_i} \frac{\lambda_1/(1-\pi_1) + \lambda_2(1-\pi)}{\lambda_2/(1-\pi_1) + \lambda_1'(1-\pi)}}$$

(5.53)
where \( \lambda_1 \), \( \lambda_2 \) and \( \lambda' \) are defined are above. These too have the same form as the expressions (5.21) and (5.22) except that the arrival rates have been modified. As before, a longer cycle length is required to minimize \( \bar{r}_2 \) and \( \bar{p}_{i;2} \), \( i = \text{CO}, \text{HC} \) and \( \text{NO}_x \), than to minimize \( \bar{d}_2 \).

### 5.4.5 Pareto Optimal Set

The Pareto optimal set is determined in the same manner as for MODEL1.

**Theorem 5.6:** Assuming the intersection is stable, the \( \pi_1 \) values (where \( 0 \leq \pi_1 \leq 1 \)) associated with the Pareto optimal set for \( \bar{d}_2 \), \( \bar{r}_2 \) and \( \bar{p}_{i;2} \), \( i = \text{CO}, \text{HC}, \) and \( \text{NO}_x \), are:

(i) \( \pi_1^* = (\pi_1^*, \bar{d}_2, \pi_1^*, \bar{p}_{i;2}) \) if \( \lambda_1' > \lambda_2' \);

(ii) \( \pi_1^* = (\pi_1^*, \bar{d}_2, \pi_1^*, \bar{p}_{i;2}) \) if \( \lambda_1' < \lambda_2' \);

(iii) \( \pi_1^* = (\pi_1^*, \bar{d}_2) \) if \( \lambda_1' = \lambda_2' \).

Proof is found in Appendix C.

For a cycle length of one minute, the curves depicting \( \bar{d}_2 \), \( \bar{r}_2 \) and \( \bar{p}_{i;2} \), \( i = \text{CO}, \text{HC}, \) \( \text{NO}_x \), as a function of \( \pi_1 \), have been rescaled and drawn on the same graph (Figure 5.4). As usual, \( \lambda_1 = 3 \) veh/min, \( \lambda_2 = 5 \) veh/min, \( L = 1/15 \) minutes and \( s = 1/30 \) minutes. The corresponding Pareto optimal set is \( \pi_1^* = (0.0, 0.3249) \). The efficient frontier for the \( \bar{d}_2 \) and \( \bar{r}_2 \) is found in Figure 5.5 and the efficient frontiers for other pairs of measures are found in Appendix C. All the graphs have basically the same shape as the analogous graphs for MODEL1.

### 5.4.6 Analysis of the Results

Each performance measure \( \bar{d}_2 \), \( \bar{r}_2 \) and \( \bar{p}_{\text{CO}_2} \) is again evaluated for the \( \pi_1 \) values which minimizes one of them for a lightly trafficked intersection. The results are summarized in Table 5.2.

<table>
<thead>
<tr>
<th>( \pi_1 )</th>
<th>( \bar{d}_2 )</th>
<th>( \bar{r}_2 )</th>
<th>( \bar{p}_{\text{CO}_2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \pi_1^*, \bar{d}_2 = 0.3249 )</td>
<td>0.1728 min = 10.37 sec</td>
<td>0.0394 gal/min = 2.36 gal/hr</td>
<td>35.758 g/min = 2145.5 g/hr</td>
</tr>
<tr>
<td>( \pi_1^*, \bar{r}_2 = 0.0944 )</td>
<td>0.2234 min = 13.04 sec</td>
<td>0.0368 gal/min = 2.20 gal/hr</td>
<td>34.100 g/min = 2046.0 g/hr</td>
</tr>
<tr>
<td>( \pi_1^*, \bar{p}_{\text{CO}_2} = 0.2152 )</td>
<td>0.1790 min = 10.74 sec</td>
<td>0.0379 gal/min = 2.27 gal/hr</td>
<td>32.226 g/min = 1933.6 g/hr</td>
</tr>
</tbody>
</table>

Table 5.2 Summary of the MODEL2 performance measure values for each \( \pi_1 \) value which minimizes one measure.
Assuming a total of 17,500 isolated fixed-cycle intersections in the United States (Section 3.2) and assuming each intersection is lightly trafficked, the expected amounts of the performance measures which correspond to current practice (minimizing delay) can be calculated. There would be an average of 24,200 veh-hr/hr or 435,600 veh-hr per 18-hr day of delay incurred, 41,300 gal/h or 743,400 gal per 18-hr day of gas consumed, and 37,550 kg/hr or 675,900 kg per 18-hr day of CO emitted as the result of the operation of these signals across the U.S.

If the U.S were to adopt a strict fuel conservation policy, the expected amount of fuel which would be conserved, with respect to current practice, is 17,500(0.16)=2,800 gal/h or 50,400 gallons per 18-hr day. This amounts to approximately 2,670 barrels of crude oil per day or about 0.03% of the 1989 average daily import amount. Again, the savings would probably be even greater because this figure assumes that each
intersection only has two incoming streams of traffic and also assumes that all fixed-cycle intersections are lightly trafficked.

![Efficient frontier for the delay and fuel performance measures (MODEL2).](image)

The potential amount of fuel which can be conserved at isolated intersections is at least 6.8% with respect to current practice. This compares to 4% to 8.9% realized under the right-turn-on-red policy and with 1.5% from engine tune-ups [Claffey 1976]. Though these different savings are not directly comparable, the magnitude of the potential fuel savings realizable from retiming existing isolated fixed-cycle signals seems to indicate the current practice should be rethought.

Finally, if at some point in the future the nation takes an extreme pro-environmental stance, the most appropriate policy would be to time the signals so that the amount of pollutants emitted is minimized. If such a policy were adopted, there would be an average savings of 66,750 kg per 18-hr day in the amount of CO emitted in the U.S., with respect to the current signal timing practice. This represents a 9.8% savings.
Assuming the extreme environmental policy were adopted and lightly trafficked conditions, there would be a 3.6% increase in delay and a 3.8% decrease in fuel consumption per isolated fixed-cycle intersection. On the other hand, under light traffic conditions, an extreme fuel conservation policy produce a 29.3% increase in delay, a 6.8% decrease in fuel consumption, and a 4.6% decrease in CO emissions per isolated fixed-cycle intersection. Under the environmental policy, minimize CO emissions, it appears as though over half of the fuel conservation benefits can be realized at a much smaller cost in terms of increased delay in addition to realizing all the potential CO emission reduction. This suggests that if a single performance measure had to be selected for optimization, the most reasonable choice may be CO emissions.

SECTION 5.5 MODEL3: Moderately Heavy Traffic Model

The difficulty of determining easily computable expressions for E[Q(0)] has led researchers to derive bounds and approximations for this quantity. An obvious lower bound is E[Q(0)]=0 which is used in both MODEL1 and MODEL2. In this section, I present an exact expression for this quantity and then an approximation. These are used in MODEL3 to more accurately describe the key measures under moderately heavy to heavy traffic conditions.

5.5.1 An Exact Expression for E[Q(0)]

An exact expression for E[Q(0)] can be obtained when the arrivals are Poisson and the vehicle service times are constant. The expression [Gazis 1974] for a given approach is

\[
E[Q(0)] = (1-\rho) \sum_{k=1}^{m-1} \frac{1}{1-\xi_k} + \frac{1}{2}(1-\rho) + \frac{C-(C-L)\pi_1}{2(1-\rho)[(C-L)\pi_1-\rho C]} - \frac{1}{2\delta^2}[C-(C-L)\pi_1-\rho C-2\rho(C-G)]
\]

(5.54)

where each \( \xi_k \) is the unique root within the unit circle of

\[
\xi_k \exp\left(\frac{\rho C(1-\xi_k)}{(C-L)\pi_1}\right) = \exp\left(\frac{2\pi i k}{m}\right)
\]

Though there are computer programs which can solve these equations, for my purposes it is better to use a more easily computable expression. My overall intent is to determine the cost-effectiveness of smart traffic signals rather than provide accurate signal timing plans for fixed-cycle control. Furthermore, without accurate
flow information for specific intersections, the accuracy of the exact expression would be lost in my other first-order approximations.

5.5.2 Approximation for $E[Q(0)]$

The quantity $Q(0)$ is the number of vehicles left in the queue at the end of a green interval or at the beginning of a combined red-lost time interval. Denote $Q_i(0)$ as the number of vehicles in queue at the beginning of the $i$th red-lost time interval. The following derivation is based on work done by Miller [1963].

If $A_i(C)$ is the number of arrivals during the entire $i$th cycle and $G/s$ is the number of possible departures during the green phase (assume $G$ is a multiple of $s$ to avoid algebraic complications) then,

$$Q_{i+1}(0) = \max[Q_i(0) + A_i(C) - G/s, 0].$$  \hspace{1cm} (5.55)

Let $A_i$ be an indicator or compensating function then (5.55) can be rewritten as

$$Q_{i+1}(0) = Q_i(0) + A_i(C) - G/s + \Delta_i.$$ \hspace{1cm} (5.56)

$\Delta_i$ can be regarded as the number of additional vehicles which could have passed through the intersection during the green interval if they had been available. So,

$$\Delta_i = \begin{cases} 0 & \text{if } Q_i(0) + A_i(C) \geq G/s \\ G/s - (Q_i(0) + A_i(C)) & \text{otherwise} \end{cases}.$$ \hspace{1cm} (5.56a)

Notice that if $Q_{i+1}(0) \neq 0$ then $\Delta_i = 0$ and if $\Delta_i \neq 0$ then $Q_{i+1}(0) = 0$. Hence, $Q_{i+1}(0) \neq 0$ if $\Delta_i = 0$.

Assuming equilibrium and taking expectations of both sides of (5.56) yields

$$E[\Delta_i] = E[G/s - A_i(C)].$$ \hspace{1cm} (5.57)

Rewriting (5.56) as

$$Q_{i+1}(0) - (\Delta_i - E[\Delta_i]) = Q_i(0) - (G/s - A_i(C) - E[G/s - A_i(C)])$$

and taking the expectation of the squares of both sides produces

$$E[Q_{i+1}^2(0)] + 2E[Q_{i+1}(0)]E[\Delta_i] + \text{Var}[\Delta_i] = E[Q_i^2(0)] + \text{Var}[G/s - A_i(C)].$$

Because of the equilibrium assumption, $E[Q_{i+1}^2(0)] = E[Q_i^2(0)]$, and so upon simplification and rearranging

$$E[Q(0)] = \frac{\text{Var}[G/s - A_i(C)] - \text{Var}[\Delta_i]}{2E[G/s - A_i(C)]}.$$ \hspace{1cm} (5.58)

The only unknown quantity is $\text{Var}[\Delta_i]$. It is known that as the arrival rate increases, $\text{Var}[\Delta_i]$ tends to zero, thus
\[
\mathbb{E}[Q(0)] \to \frac{\text{var}[G-A(C)]}{2\mathbb{E}[G-A(C)]} \quad \text{as } \lambda \to \infty
\]

Miller [1963] does a numerical study of the values for \(\frac{\text{var}[\Delta_i]}{\mathbb{E}[\Delta_i]}\) for Poisson arrivals with average arrival equal to 10 veh/cycle. From this study he draws the conclusion that a good first order approximation for \(\frac{\text{var}[\Delta_i]}{\mathbb{E}[\Delta_i]}\) is \(\frac{\text{var}[\Delta_i]}{\mathbb{E}[\Delta_i]} \approx 1\). Though better approximations can and have been derived [Gazis 1974], this approximation will suffice for my purposes. Substituting this quantity into (5.58) gives

\[
\mathbb{E}[Q(0)] = \frac{\lambda C}{2(\lambda C - \lambda)} \frac{1}{2} = \frac{\rho C}{2(\lambda C - \rho C)} \frac{1}{2}
\]

(5.59)

Gazis [1974] calculates the exact value of \(\mathbb{E}[Q(0)]\) (5.54) and compares expressions for a measure of expected delay using it to expressions using the above approximation. He concludes that (5.59) produces a value which is larger than the exact value. The bound becomes better as cycle time increases. Because I am more interested in the relative values of the performance measures, instead of exact values, the simpler computational expression in (5.59) makes it more attractive than (5.54).

5.5.3 Expected Delay per Randomly Arriving Vehicle

Substituting (5.59) into (5.43) yields an approximation for the expected total delay per cycle,

\[
\mathbb{E}[W] = \lambda_1 \frac{C - (C-L)\pi_1}{2(1-\rho_1)} \frac{\rho_1 C}{\lambda_1 (C-L)\pi_1 - \lambda_1} \frac{1}{\lambda_1} + C - (C-L)\pi_1 + s \frac{2-\rho_1}{1-\rho_1}
\]

\[+
\lambda_2 (C-L)\pi_1 + L \frac{\rho_2 C}{\lambda_2 (C-L)(1-\pi_1) - \lambda_2} \frac{1}{\lambda_2} + (C-L)\pi_1 + L + s \frac{2-\rho_2}{1-\rho_2}
\]

(5.60)

From this \(\bar{d}_3\) is obtained by dividing by \(\lambda C\) yielding,

\[
\bar{d}_3 = \lambda_1 \frac{C - (C-L)\pi_1}{2\lambda C(1-\rho_1)} \frac{\rho_1 C}{\lambda_1 (C-L)\pi_1 - \lambda_1} \frac{1}{\lambda_1} + C - (C-L)\pi_1 + s \frac{2-\rho_1}{1-\rho_1}
\]

\[+
\lambda_2 (C-L)\pi_1 + L \frac{\rho_2 C}{\lambda_2 (C-L)(1-\pi_1) - \lambda_2} \frac{1}{\lambda_2} + (C-L)\pi_1 + L + s \frac{2-\rho_2}{1-\rho_2}
\]

(5.61)

Miller [1963] solves for \(C\) given \(\pi_1\) by disregarding small terms and only considering the approach with the largest flow. Doing this, he obtains

\[
\hat{C}_{d_3} = \frac{\pi_1(1-\pi_i)L + \pi_i \sqrt{\frac{sL(1-\rho_i)}{\pi_i}}}{(1-\pi_i)(1-\rho_i)} , \quad \text{where } i = \arg \max(\lambda_2, \lambda_1).
\]

(5.62)
For this value of $C$, he found (through elimination of small terms) the value of $\hat{\pi}_{1,\overline{d}_3}^*$

$$\hat{\pi}_{1,\overline{d}_3}^* = \frac{\sqrt{p_1} + \frac{C}{C-L}\sqrt{p_1p_2(\sqrt{p_1} - \sqrt{p_2})}}{\sqrt{p_1} + \sqrt{p_2}}.$$  \hspace{1cm} (5.63)

It is possible, for a given intersection, to obtain the values of $C$ and $\pi_1$ which minimize $\overline{d}_3$, when keeping one of the two parameters fixed, without having to get rid of the small terms as Miller did. This could be done using a software package similar to Mathematica. Also, for a given intersection, one could prove whether $\overline{d}_3$ is convex in the unspecified parameter. My conjecture is that under stable intersection conditions, $\overline{d}_3$ is convex in $\pi_1$ given $C$ and in $C$ given $\pi_1$.

Appendix C contains a graph (Figure C.17) of $\overline{d}_3$ as a function of the $\pi_1$ values for three different cycle lengths.

5.5.4 Expected Fuel Consumption and Pollution Emissions

As done in the previous two sections, expressions can be derived for $\overline{f}_3$ and $\overline{p}_{i,3}$ using equation (5.60) to calculate $\overline{D}_3$. A adjustment has to be made to the previous $\overline{S}$ calculations to obtain $\overline{S}_3$. The derivations are found in Appendix C. Below are the expressions for $\overline{f}_3$ and $\overline{p}_{i,3}$.

$$\overline{f}_3 = \alpha^* \frac{\lambda_1[C-(C-L)\pi_1]}{2C(1-\rho_1)} - \frac{\rho_1C}{\lambda_1[(C-L)\pi_1 - \rho_1C]} - \frac{1}{\lambda_1} + C - (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1}$$

$$+ \frac{\lambda_2[C-(C-L)\pi_1 + L]}{2C(1-\rho_2)} - \frac{\rho_2C}{\lambda_2[(C-L)(1-\pi_1) - \rho_2C]} - \frac{1}{\lambda_2} + (C-L)\pi_1 + L + \frac{2-\rho_2}{1-\rho_2}$$

$$+ \left\{ \frac{\lambda_1[C-(C-L)\pi_1]}{C(1-\rho_1)} + \frac{\lambda_2[(C-L)\pi_1 + L]}{C(1-\rho_2)} + \frac{\rho_1C}{[(C-L)\pi_1 - \rho_1C]} + \frac{\rho_2C}{[(C-L)(1-\pi_1) - \rho_2C]} - 1 \right\}$$  \hspace{1cm} (5.64)

$$\overline{p}_{i,3} = \gamma^* \frac{\lambda_1[C-(C-L)\pi_1]}{2C(1-\rho_1)} - \frac{\rho_1C}{\lambda_1[(C-L)\pi_1 - \rho_1C]} - \frac{1}{\lambda_1} + C - (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1}$$

$$+ \frac{\lambda_2[(C-L)\pi_1 + L]}{2C(1-\rho_2)} - \frac{\rho_2C}{\lambda_2[(C-L)(1-\pi_1) - \rho_2C]} - \frac{1}{\lambda_2} + (C-L)\pi_1 + L + \frac{2-\rho_2}{1-\rho_2}$$

$$+ \delta^* \left\{ \frac{\lambda_1[C-(C-L)\pi_1]}{C(1-\rho_1)} + \frac{\lambda_2[(C-L)\pi_1 + L]}{C(1-\rho_2)} + \frac{\rho_1C}{[(C-L)\pi_1 - \rho_1C]} + \frac{\rho_2C}{[(C-L)(1-\pi_1) - \rho_2C]} - 1 \right\}$$  \hspace{1cm} (5.65)

Values for $\pi_{1,\overline{f}_3}^*$, $\pi_{1,\overline{p}_{i,3}}^*$, $C_{\overline{f}_3}^*$ and $C_{\overline{p}_{i,3}}^*$ could be calculated in the same manner as Miller did for $\pi_{1,\overline{d}_3}^*$ and $C_{\overline{d}_3}^*$. Unless done for a specific intersection, the algebra is extremely cumbersome.
One thing to note is that this model is best for moderate to heavily trafficked intersections. The approximation used for $\mathbb{E}[Q(0)]$ is an upper bound. For light to moderately trafficked intersections, MODEL2 is more accurate.

5.5.5 Pareto Optimal Set for the $\pi_i$ Values

Assuming a moderately heavy trafficked intersection, $C=1.5$ minutes and the usual values for $L$ and $s$, I have rescaled and graphed $\bar{d}_3$, $\bar{f}_3$, and $\bar{p}_{i3}$, $i = \text{CO, HC and NO}_x$, on the same plot (Figure 5.6). The curves have the same shape as in the previous two models. The Pareto optimal set is $\mathcal{P}^*=(0.3966, 0.4750)$. Table 5.3 contains the optimal $\pi_i$ splits for each of the performance measures. The striking characteristic of these results is that the four measures which explicitly penalize stops have optimal splits that are indistinguishable for practical purposes. Figure 5.7 shows the efficient frontier for $\bar{f}_3$ and $\bar{d}_3$.

![Figure 5.6 Rescaled performance measures (MODEL3) as a function of the $\pi_i$ values and the corresponding Pareto optimal set.](image-url)
### Table 5.3  
Optimal MODEL3 π₁ splits for each of the performance measures.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Optimal π₁ Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>delay</td>
<td>0.4750</td>
</tr>
<tr>
<td>fuel consumption</td>
<td>0.3972</td>
</tr>
<tr>
<td>CO emissions</td>
<td>0.3988</td>
</tr>
<tr>
<td>HC emissions</td>
<td>0.3985</td>
</tr>
<tr>
<td>NOx emissions</td>
<td>0.3966</td>
</tr>
</tbody>
</table>

Because the optimal π₁ splits for the fuel and pollution measures are too close to be distinguishable in practice, I only evaluate the performance measures of the values of π₁ which minimize \( \bar{d}_3 \) and \( \bar{f}_3 \) for a moderately heavy trafficked intersection. These results are summarized in Table 5.4.

### Table 5.4  
Summary of the MODEL3 performance measure values for the π₁ values which minimize \( \bar{f}_3 \) and \( \bar{d}_3 \).

<table>
<thead>
<tr>
<th>π₁</th>
<th>( \bar{d}_3 )</th>
<th>( \bar{f}_3 )</th>
<th>( \bar{p}_{CO,3} )</th>
<th>( \bar{p}_{HC,3} )</th>
<th>( \bar{p}_{NOx,3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \pi_1, \bar{d}_3=0.4750 )</td>
<td>0.6588 min</td>
<td>11.358 gal/hr</td>
<td>10.112 kg/hr</td>
<td>0.612 kg/hr</td>
<td>0.198 kg/hr</td>
</tr>
<tr>
<td>( \pi_1, \bar{f}_3=0.3972 )</td>
<td>.7251 min</td>
<td>19.512 gal/hr</td>
<td>9.325 kg/hr</td>
<td>0.565 kg/hr</td>
<td>0.182 kg/hr</td>
</tr>
</tbody>
</table>

Assuming a total of 17,500 isolated fixed-cycle intersections in the United States (Section 3.2) and assuming each intersection is moderately heavy trafficked, the expected amounts of the performance measures which correspond to current practice (minimizing delay) can be calculated. There would be an average of 230,580 veh-hr/hr or 4,150,000 veh-hr per 18-hr day of delay incurred, 198,770 gal/h or 3,578,000 gal per 18-hour day of gas consumed, 176,960 kg/hr or 3,185,000 kg per 18-hr day of CO emitted, 10,710 kg/hr or 192,780 kg per 18-hr day of HC emitted, and 3,470 kg/hr or 62,370 kg per 18-hour day of NOₓ emitted as the result of the operation of these signals across the U.S.

If the U.S were to adopt a strict fuel conservation policy, the expected amount of fuel which would be conserved, with respect to current practice, is 17,500(0.846)=14,810 gal/h or 266,490 gallons per 18-hour day. This amounts to approximately 14,100 barrels of crude oil per day or about 0.15% of the 1989 average daily import amount. Again, the savings amount assumes that each intersection has
only two incoming streams of traffic and that all fixed-cycle intersections are moderately heavy trafficked.

![Graph showing efficient frontier for delay and fuel performance measures (MODEL3).](image)

Figure 5.7 Efficient frontier for the delay and fuel performance measures (MODEL3).

The potential amount of fuel which can be conserved at isolated fixed-cycle intersections is at least 7.4% with respect to current practice. Again providing evidence that the current practice of optimizing only with respect to delay should be rethought.

5.5.6 Pareto Optimal Set for the C Values

The cycle lengths obtained under this model are realistic and so it is meaningful to obtain the Pareto optimal set of the three performance measures with respect to cycle length. The curves depicting the performance measures as a function of the cycle length have been rescaled and are drawn in Figure 5.8. The corresponding Pareto optimal set is \( C*\epsilon[.6379, 1.4119] \), where \( C \) is in minutes. Figure 5.9 shows the efficient frontier for \( \overline{d}_3 \) and \( \overline{f}_3 \) as a function of the cycle length. The efficient frontiers for the other pairs of measures can be found in Appendix C.
Table 5.5 summarizes the values of each of the performance measures for the $C$ values which minimizes one of them. Using these values, an analysis similar to the ones performed for the $\pi_1$ values can be performed.

<table>
<thead>
<tr>
<th>$C$</th>
<th>$\bar{d}_3$</th>
<th>$\bar{f}_3$</th>
<th>$\bar{P}_{CO3}$</th>
<th>$\bar{P}_{HC3}$</th>
<th>$\bar{P}_{NOx3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{Q}^*$</td>
<td>0.1999 min</td>
<td>10.032 gal/hr</td>
<td>7.544 kg/hr</td>
<td>0.473 kg/hr</td>
<td>0.183 kg/hr</td>
</tr>
<tr>
<td>$C_{F}^*$</td>
<td>0.2310 min</td>
<td>9.558 gal/hr</td>
<td>7.533 kg/hr</td>
<td>0.467 kg/hr</td>
<td>0.172 kg/hr</td>
</tr>
<tr>
<td>$C_{PCO3}^*$</td>
<td>0.2079 min</td>
<td>9.672 gal/hr</td>
<td>7.400 kg/hr</td>
<td>0.462 kg/hr</td>
<td>0.175 kg/hr</td>
</tr>
<tr>
<td>$C_{PNC3}^*$</td>
<td>0.9009 min</td>
<td>9.642 gal/hr</td>
<td>7.403 kg/hr</td>
<td>0.462 kg/hr</td>
<td>0.175 kg/hr</td>
</tr>
<tr>
<td>$C_{PNOx3}^*$</td>
<td>1.4119 min</td>
<td>9.600 gal/hr</td>
<td>7.735 kg/hr</td>
<td>0.477 kg/hr</td>
<td>0.171 kg/hr</td>
</tr>
</tbody>
</table>

Table 5.5 Summary of the MODEL3 performance measure values for the $C$ values which minimizes one of them.
Efficient Frontier for Fuel and Delay Measure

Figure 5.9 Efficient frontier for the delay and fuel performance measures (MODEL3) as a function of the cycle length, $C$.

Assuming a total of 17,500 isolated fixed-cycle intersections in the United States (Section 3.2) and assuming each intersection is moderately heavy trafficked, the expected amounts of the performance measures which correspond to current practice (minimizing delay) can be calculated. There would be an average of 69,970 veh-hr/hr or 1,259,000 veh-hr per 18-hr day of delay incurred, 175,560 gal/h or 3,160,000 gal per 18-hr day of gas consumed, 132,020 kg/hr or 2,376,000 kg per 18-hr day of CO emitted, 8,280 kg/hr or 149,100 kg per 18-hr day of HC emitted, and 3,210 kg/hr or 57,750 kg per 18-hr day of NO$_x$ emitted as the result of the operation of these signals across the U.S.

If the U.S. were to adopt a strict fuel conservation policy, the expected amount of fuel which would be conserved, with respect to current practice, is $17,500(0.474)=8,300$ gal/h or 149,310 gallons per 18-hr day. Thus the potential amount of fuel which can be conserved at isolated moderately trafficked fixed-cycle
intersections by optimizing the cycle length is approximately 4.7% for a given green-red split. This savings compares with the approximately 7.4% which can be realized by optimizing over the red-green splits for a given cycle length.

For this particular example, it appears as though there is a greater potential for fuel savings through optimizing the red-green splits than the cycle lengths. As mentioned before, individuals have informally explored the cycle lengths with regard to delay and fuel measures or pollution measures (Section 4.1).

SECTION 5.6 Summary and Conclusion

To close this chapter, I briefly summarize some of the findings and then draw some relevant conclusions.

5.6.1 Summary

In this chapter I formalize the tradeoffs between the five performance measures of interest to me for a given signal technology. As a by-product, I discovered that such an analysis for the fixed-cycle intersection in operation today may be appropriate due to the substantial amount of potential savings in fuel consumption and air pollution.

In order to better illustrate the tradeoffs which exist among the measures, I use the simplest type of signal control strategy: fixed-cycle control. In my analysis, I use two "average" intersections: a lightly trafficked intersection and a moderately trafficked intersection (defined in Section 5.2.3). Though the analysis is done for a fixed-cycle signal, it could be done for any type of new and existing control strategy.

For the two different traffic conditions considered, there is a given model which best describes the operation of the intersection. In the case of the lightly trafficked, I present two models. The first and simplest model is used to illustrate how one calculates the quantities of interest and finds the Pareto optimal set of timing plans. The second model (Section 5.4) provides a more realistic description of the operation of the intersection and hence more accurate equations for the key measures. Basically, the two lightly trafficked models presented produce the same general results but with slightly different values. Because MODEL2 accounts for the delay incurred in queue, it produces higher values for each of the performance measures but the basic relationships between them remain the same. The third model is for a
CHAPTER FIVE. Pareto Optimal Sets for Fixed-Cycle Signal Control

moderately heavy to heavily trafficked intersection. It accounts for queues of vehicles which may not completely clear the intersection during their allotted green interval.

For each type of intersection, the Pareto optimal set for the five performance measures is found. For the red-green splits, the Pareto optimal set for $\pi_1$ are those values which lie between the ratio which minimizes the expected delay per randomly arriving vehicle and the ratio which minimizes the NO$_x$ emissions.

Using the more realistic model for a lightly trafficked intersection, it appears that the potential amount of fuel which could be conserved through the operation of an isolated intersection, over common practice, is approximately 6.8%. If the full savings were realized, this would result in a daily savings of 2,670 barrels of crude oil. Though this represents less than 0.1% of the daily imports of crude oil, the savings in absolute terms appears to be significant.

For the lightly trafficked intersection, if one measure had to be chosen over which to optimize, it appears as though CO emissions would be the most reasonable measure to choose. Minimizing CO emissions would result in a 9.8% savings in CO emissions with respect to current practice and 3.8% savings in fuel consumption all at the cost of 3.6% in delay. To gain the full 6.8% of the potential fuel savings, the cost in terms of delay would be an increase of 29.3% or an increase of 127,510 veh-hr per 18-hr day. On the other hand, to gain a 3.8% in fuel savings, the cost is an increase in the daily amount of delay of only 15,620 veh-hr per 18-hour day when one minimizes CO emissions.

The results for the moderately trafficked intersection are similar. The potential amount of fuel which can be conserved at such an isolated intersection is 7.4%. If all fixed-cycle intersections experienced moderately heavy traffic for 18 hours of the day, then, the potential fuel savings could be 14,100 barrels of crude oil per day which is approximately 0.15% of the U.S. daily import amount.

Not all the 17,500 isolated are lightly trafficked throughout the entire day, nor do they experience moderately heavy traffic for the entire day. Realizing this, the potential amount of total daily savings in terms of crude oil is somewhere between 2,600 barrels and 14,100 barrels. These amounts seem significant enough to make explicit consideration of the tradeoffs involved among the measures when developing timing plans for fixed-cycle intersections.

Authors in the past have informally examined the relationship between the delay measure and fuel consumption measure with respect to cycle time (Section 4.1). In
Section 5.5.6, I determine the Pareto optimal set for the cycle lengths for a given $\pi_1$ value. My result supports those found in the past, longer cycle lengths reduce fuel consumption (and pollution emissions) while increasing delay. For a given $\pi_1$ value, I discovered for a moderately trafficked intersection, the potential fuel savings which can be realized by optimizing the cycle length is approximately 4.7%. Thus, it appears more beneficial, in terms of potential tradeoffs, to choose a reasonable cycle length and then to determine the split which produces the "optimal" tradeoff.

5.6.2 Conclusion

The main purpose of this chapter was to explore the tradeoffs among the various performance measures of interest. Figure 5.10 is a pictorial summary of my findings with regard to delay and fuel consumption. This picture could be the situation in which a cycle time is given and the splits must be determined, or the reverse situation, or the situation in which the best cycle length and split pair are determined. Such a diagram could be drawn for any pair of performance measures of interest.

Point 1 corresponds to a fixed-cycle timing plan which is optimal with respect to the commonly accepted practice of timing signals to minimize delay. Point 2 corresponds to the most likely status of the timing plans in practice. This could represent both a situation where the long term trend of traffic has changed making the current plan obsolete and non-optimal with respect to all measures or it could correspond to a situation where there is a short lived traffic fluctuation and so the plan is not optimal with respect to the immediate horizon. Point 3 obviously corresponds to an optimal timing plan for an extreme fuel conservation policy.

The most desirable point for a fixed-cycle signal in terms of both delay and fuel consumption corresponds to point 4 on the graph. As was shown in Section 5.3.4, neither extreme policy is desirable and so this implies some policy which weights the performance measures is best. If one is restricted to optimizing one measure it may be more reasonable to minimize CO emissions.

Point 5 corresponds the optimal timing plan with respect to the delay and fuel measures for a "smart" signal technology. Notice that this point provides improvements in both measures above all the fixed-cycle timing plans which lie on the Pareto optimal frontier. The only way to move in this direction is through more traffic-responsive technologies. An indirect observation is that under smart technologies, it is easier to maintain a point on the Pareto optimal frontier because these strategies are traffic responsive and hence are able to adapt to changing traffic conditions.
Table 5.10. Diagram depicting hypothetical Pareto delay-fuel optimal sets for different technologies.
Chapter Six

Values of the Performance Measures Under Perfect Information

In this chapter, I use two different methods to evaluate the value of perfect information for each performance measure. The purpose of obtaining values under perfect information is twofold. First, it establishes an ideal standard for each measure associated with the operation of an isolated traffic signal. Secondly, it establishes whether or not substantial differences exist between the optimal strategies for each of the performance measures. Significant differences between the optimal policies validates explicit consideration of all five measures in the development of future real-time adaptive control strategies for isolated intersections (and for coordinated signal networks).

Two approaches for obtaining the values of the five performance measures under perfect information are investigated. The first method, presented in Section 6.1, is an extension of an approach used by Gartner [1982] which represents time as discrete intervals and determines the optimal value of total delay using dynamic programming. I extend his algorithm to include all five performance measures of interest. In Section 6.2, a different approach is taken to devise a new continuous-time approximation of the value of perfect information which does not use dynamic programming. The following section compares the characteristics of the two different techniques. Section 6.4 closes the chapter with an example which prices the expected value of perfect
information for a fixed-cycle intersection. It also contains a discussion of the potential nationwide savings possible under perfect information.

SECTION 6.1 Optimal Policy Adaptive Control (OPAC-1) Strategy

If it were known precisely when vehicles were to arrive to the intersection, it would be a relatively easy matter to determine the optimal operation of the traffic signal in order to optimize some performance index. Each instant in time can be viewed as a decision point where the decisions are (1) maintain the current status, of the signal or (2) switch the signal status and award the green indication to a different set of approaches. Partitioning time into discrete intervals and making the signal status decision at the beginning of each interval, naturally suggests the use of dynamic programming to obtain an optimal solution.

Thus, I begin by describing an extension to the discrete-time dynamic programming OPAC-1 strategy developed by Gartner [1982]. In the following subsection, I present the expected values of the performance measures using the OPAC-1 strategy.

6.1.1 OPAC-1 Strategy

What follows is the Dynamic Programming (DP) formulation for determining the optimal control strategy for an intersection with two approaches. Note that the strategy could be easily generalized to allow for any number of approaches.

The formulation is essentially the one developed by Gartner with some modifications which allow any of the five performance indices to be used. In general the dynamic programming approach utilizes the fact that the decision, whether or not to switch the current status of the signal, made at the beginning of each interval only depends on the queue lengths of each approach at the beginning of the interval, the status of the signal at the beginning of the interval, and the number of vehicles which will arrive on each approach during the interval. It is assumed that there are \( N \) discrete time intervals each of which are of five-second durations.

The stages of the DP are \( N \) five-second time intervals. At a given stage \( k \), the state of the algorithm is defined by \((Q_k, Y_k)\) where \( Q_k = [q_k, q_k^2] \) and \( Y_k = [y_k, y_k^2] \) and \( k=1,2,\ldots,N \). \( q_k^i \) is the length of the queue on approach \( i \) and \( y_k^i \) is the status of the

\(^7\) OPAC is an acronym for Optimal Policy Adaptive Control.
signal for approach \( i, i = 1 \) or 2, which is either green or red. At the final stage, an operating decision has been specified for each stage \( k (k = 1, 2, \ldots, N) \) such that the performance measure has been minimized over the entire time interval of interest (i.e. over \([0, T]\) where \( T = N \cdot 5 \) seconds).

It is assumed that an intersection of two single lane approaches is controlled by a signal which alternately assigns the green indication to approach 1 and 2; there is no yellow period in this formulation\(^8\). If the optimal decision for stage \( k \) is to switch the status of the signal, both approaches are assigned a red indication for the entire five-second interval. When an approach has the green, the maximum discharge rate is assumed to be two vehicles per interval\(^9\).

The algorithm uses dynamic programming to find the optimal signal control strategy which is characterized by a sequence of switch or no-switch decisions. The method used is backward recursion and it proceeds by the stages corresponding to discrete intervals of five-seconds. At each stage \( k \), the best signal operating decision is made such that a specified performance index is minimized. Basically, the optimal sequence of decisions \( (y^1_0, d_1, \ldots, d_N) \), where \( y^1_0 \) is the initial status of the signal for approach 1 at the start of the time horizon under consideration (the status of approach 1 implies the status of approach 2), is computed. At a stage \( k \), the optimal sequence of decisions has been computed for the succeeding \( k+1, \ldots, N \) stages (from the DP principle of optimality). Hence, it is necessary to find the best decision for state \((Q_k, Y_k)\) which produces the least cost sequence from \((Q_k, Y_k)\) to \((Q_N, Y_N)\).

Before beginning a more specific discussion of the algorithm, I define the key variables of the DP below:

\begin{itemize}
    \item N, the total number of stages in the DP.
    \item j, the index for the performance measure. In this section, d represents the delay measure, f the fuel measure, and p_l the pollution measure (where \( l = \) CO, HC or NO\(_x\)).
    \item \( q^i_k \), a state variable describing the the length of the queue on approach \( i, i = 1, 2 \).
\end{itemize}

\(^8\)The yellow period can be considered as being included in the all-red interval which occurs between status changes for the signal.

\(^9\)This is the approximate flow rate for a single lane of traffic. This could easily be modified to approximate flow rates for different intersection configurations.
• $y^i_k$, a state variable describing the status of the signal for approach $i$, $i=1,2$. A value of 0 corresponds to green and 1 to red.

• $A_k=[a^1_k, a^2_k]$, the given input vector containing the number of arrivals on approach 1 and approach 2 during stage $k$.

• $d_k$, the decision variable associated with stage $k$. A value of 0 corresponds to the decision of no-switch and 1 corresponds to the decision to switch. $d_k^*={D_k^*}(Q_k, Y_k)$ is the optimal decision associated with state $(Q_k, Y_k)$.

• $P_l^j(Q_k, Y_k)$, the optimal value of performance measure $j$ from stage $k$ onwards.

• $C(Q_k, Y_k, A_k, d_k)$, the cost incurred at stage $k$ as the result of decision $d_k$.

Assuming a maximum discharge rate of two vehicles per interval, for a given state $(Q_k, Y_k)$ associated with stage $k$, the decision $d_k$ implies the succeeding state is

$$Q_{k+1} = [q^1_{k+1}, q^2_{k+1}]$$

where

$$q^i_{k+1} = \max(0, q^i_k + a^i_k - 2(1-y^i_k)(1-(y^i_k + d_k)_{MOD2})) \quad \text{for } i=1,2,$$

and

$$Y_{k+1} = [y^1_{k+1}, y^2_{k+1}]$$

where

$$y^i_{k+1} = (y^i_k + d_k)_{MOD2} \quad \text{for } i=1,2.$$

The cost function associated with decision $d_k$ is

$$C(Q_k, Y_k, A_k, d_k) = \begin{cases} 
1 \cdot \psi_j(q^1_k + q^2_{k+1} + \frac{1}{2}(a^1_k + a^2_k)) + \omega_j(a^1_k + a^2_k) & \text{for } d_k=1 \\
1 \cdot \psi_j(q^1_k + \delta^i_k + \frac{1}{2}(a^1_k + \max(0, a^2_k - \delta^i_k))) + \omega_j(a^1_k + \max(0, a^2_k - \delta^i_k)) & \text{for } d_k=0 \text{ and } y^j_k=1 \\
1 \cdot \psi_j(q^1_k + \delta^i_k + \frac{1}{2}(a^1_k + \max(0, a^2_k - \delta^i_k))) + \omega_j(a^1_k + \max(0, a^2_k - \delta^i_k)) & \text{for } d_k=0 \text{ and } y^j_k=0 
\end{cases}$$

$$\text{where } I \equiv \text{length of the discrete time intervals} = 5 \text{ seconds },$$

and

$$\delta^i_k = \begin{cases} 
2-q^i_k & \text{if } 2-q^i_k \geq 0 \\
0 & \text{if } 2-q^i_k < 0 
\end{cases} \quad \text{for } i=1,2 \text{ and all } j.$$

The cost function for stage $k$ is comprised of a cost associated with the delay of the vehicles at the signal, reflected by the $\psi_j$, plus the cost associated with the stop/start
CHAPTER SIX. Values of the Performance Measures Under Perfect Information

maneuver of a vehicle, reflected by the \( \omega_j \). For \( j = d \), the value of \( \psi_j \) is 1 (one second of delay is accrued for each vehicle second) and \( \omega_j = 0 \) because no additional delay penalty is associated with a stop/start maneuver. For \( j = f \) and \( p_l \), the values are \( \psi_f = \alpha \) and \( \omega_f = \beta \), and \( \psi_{p_l} = \gamma_{p_l} \) and \( \omega_{p_l} = \delta_{p_l} \) (for \( l = \text{CO}, \text{HC}, \text{NO}_X \)), respectively. Values for these coefficients can be found in Chapter Four.

Following the DP principle of optimality, at each stage \( k \), the best decision \( d_k \) is the one which minimizes the performance measure \( j \) from stage \( k \) onwards to stage \( k = N \). This is given by

\[
P^*(Q_k, Y_k) = \min_{d_k=0,1} C(Q_k, Y_k, A_k, d_k) + P^*(Q_{k+1}, Y_{k+1}) \quad (6.7)
\]

The first term is the cost of the best decision for stage \( k \) and the second term is the optimal value of performance measure \( j \) for all decisions made from \( k + 1 \), the succeeding stage, to \( N \), the final stage. If the optimal decision is \( d_k^* \), then

\[
P^*(Q_k, Y_k) = C(Q_k, Y_k, A_k, d_k^*) + P^*(Q_{k+1}, Y_{k+1}) \quad (6.8)
\]

Since the initial queue lengths \( q^*_0 (i=1,2) \) and initial signal status, \( y^*_0 \), is known, it is possible to retrace the optimal policy by taking a forward pass through the stored values of \( D_k^*, k=1,\ldots,N \). The policy consists of the optimal sequence of switching decisions \{\( d_k^*, i=1,\ldots,N \)} for all stages of the optimization process.

### 6.1.2 OPAC-1 Experimental Results

The modified OPAC-1 algorithm described in the previous subsection was implemented on a Macintosh II. I coded the algorithm in the "C" language. As done for the fixed-cycle analysis in Chapter Five, I obtain results for both a typical lightly trafficked intersection and for a typical moderately trafficked intersection. In total, there are five different performance measures which can be minimized (three of which are pollution measures): total delay per time unit, total fuel consumption per time unit, total CO emissions per time unit, total HC emissions per time unit, and total NO\(_X\) emissions per time unit.

As in the previous chapter, the lightly trafficked intersection assumes average arrival rates of 3 veh/min and 5 veh/min for approaches 1 and 2, respectively. I obtained five sets of data, of 100 runs each, using the algorithm. Each set of 100 runs corresponds to the optimization of one of the five performance measures. A run determines the optimal signal operating schedule and value for a designated performance index over a 10 minute period of time for which the Poisson arrival data for each approach is known entirely in advance. It also gives the values of the other
four measures which were not optimized. For each set of 100 runs, I calculated the sample means for each performance measure along with the 95% confidence bounds for the values. These are summarized in Table 6.1. The values in the highlighted cells along the diagonal are the minimum values of the corresponding performance measure under perfect information.

Table 6.2 contains results for a moderately heavy trafficked intersection analogous to those in Table 6.1. The average Poisson arrival rates for this intersection are assumed to be 8 veh/min and 12 veh/min for approaches 1 and 2, respectively.

<table>
<thead>
<tr>
<th>Minimized Performance Measure</th>
<th>E[delay] (sec/10-min)</th>
<th>E[fuel consumption] (gal/10-min)</th>
<th>E[CO emissions] (g/10-min)</th>
<th>E[HC emissions] (g/10-min)</th>
<th>E[NOX emissions] (g/10-min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay</td>
<td>200.55 (187.5, 213.6)</td>
<td>0.241 (0.229, 0.253)</td>
<td>163.95 (155.4, 172.5)</td>
<td>10.51 (9.96, 11.06)</td>
<td>4.513 (4.283, 4.743)</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>292.93 (272.8, 313.0)</td>
<td>0.198 (0.188, 0.209)</td>
<td>150.09 (141.8, 158.4)</td>
<td>9.39 (8.87, 9.91)</td>
<td>3.602 (3.418, 3.808)</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>245.53 (230.4, 260.6)</td>
<td>0.203 (0.191, 0.214)</td>
<td>146.99 (139.0, 155.0)</td>
<td>9.29 (8.78, 9.79)</td>
<td>3.780 (3.559, 4.001)</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>254.28 (238.6, 270.0)</td>
<td>0.201 (0.190, 0.212)</td>
<td>147.66 (139.6, 155.7)</td>
<td>9.27 (8.77, 9.78)</td>
<td>3.695 (3.494, 3.896)</td>
</tr>
<tr>
<td>NOX Emissions</td>
<td>371.53 (293.8, 341.3)</td>
<td>0.199 (0.188, 0.209)</td>
<td>153.23 (144.5, 161.9)</td>
<td>9.55 (9.02, 10.09)</td>
<td>3.60 (3.407, 3.794)</td>
</tr>
</tbody>
</table>

Table 6.1 Summary of the expected values of the performance measures obtained using OPAC-1 for the lightly trafficked intersection averaged across each set of 100 runs per measure minimized. The 95% confidence interval is given below each number.

The results in Tables 6.1 and 6.2 indicate that under perfect information, there are significant differences between the values of the performance measures under a policy which optimizes the delay measure and a policy which optimizes any one of the other four measures. The existence of such differences under perfect information suggests that it is desirable to explicitly consider the tradeoffs between delay and the other performance measures when developing new, more real-time adaptive signal control strategies.
### Table 6.2
Summary of the expected values of the performance measures obtained using OPAC-1 for the moderately heavy trafficked intersection averaged across each set of 100 runs per measure minimized. The 95% confidence interval is given below each number.

<table>
<thead>
<tr>
<th>Minimized Performance Measure</th>
<th>E[delay] (sec/10-min)</th>
<th>E[fuel consumption] (gal/10-min)</th>
<th>E[CO emissions] (g/10-min)</th>
<th>E[HC emissions] (g/10-min)</th>
<th>E[NOx emissions] (g/10-min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay</td>
<td>3381.3 (3087.8, 3674.8)</td>
<td>1.527 (1.470, 1.585)</td>
<td>1289.2 (1221.7, 1356.8)</td>
<td>78.87 (74.98, 82.76)</td>
<td>26.93 (26.04, 27.82)</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>3885.8 (3589.2, 4182.4)</td>
<td>1.434 (1.374, 1.493)</td>
<td>1294.3 (1225.1, 1363.5)</td>
<td>78.15 (74.16, 82.15)</td>
<td>24.70 (23.77, 25.63)</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>3484.0 (3193.4, 3774.6)</td>
<td>1.466 (1.406, 1.526)</td>
<td>1265.5 (1196.9, 1334.0)</td>
<td>77.07 (73.10, 81.04)</td>
<td>25.65 (24.71, 26.58)</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>3535.7 (3245.0, 3826.3)</td>
<td>1.455 (1.395, 1.515)</td>
<td>1265.5 (1196.9, 1334.0)</td>
<td>76.96 (73.00, 80.93)</td>
<td>25.50 (24.54, 26.46)</td>
</tr>
<tr>
<td>NOx Emissions</td>
<td>4198.7 (3898.6, 4498.7)</td>
<td>1.440 (1.380, 1.500)</td>
<td>1335.1 (1265.5, 1404.7)</td>
<td>80.22 (76.20, 84.24)</td>
<td>24.57 (23.65, 25.50)</td>
</tr>
</tbody>
</table>

If it is necessary to select a timing policy which only optimizes one of the five performance measures, the CO emissions measure is arguably a better choice than the delay measure. Optimizing the delay measure produces values for the other four measures which are significantly greater than their minimum values. On the other hand, when CO emissions are minimized, the values for fuel consumption, and the two other pollution measures do not differ significantly from their respective minimum values. Though the value of the delay measure does differ significantly from its minimum value, minimizing CO emissions produces a delay value closer to its minimum value than would policies which minimize fuel consumption or any of the other two pollution measures. Hence when CO emissions are minimized, only the resulting delay value is significantly different from its minimum value compared to when delay is minimized, the values of all the other performances measures differ significantly from their minimum values.

The best policy is probably not the selection of one of the measures to optimize in isolation of the other measures but rather some weighted function of all five measures. The difficulty lies in determining the function which best reflects how society currently values each of the five different quantities.
SECTION 6.2 A Continuous Time Approximation for the Values of the Key Measures under Perfect Information

The dynamic programming based OPAC-1 strategy presented in the previous section is time consuming. To determine the optimal value of a given performance measure for ten minutes of arrival data requires approximately seven minutes on a Macintosh II fx. Also, the OPAC-1 strategy does not suggest a back-of-the-envelope calculation which could be used to obtain bounds for the values of the performance measures under perfect information.

Another drawback of the OPAC-1 strategy is that useful arrival information is lost when time is partitioned into discrete intervals. Using the exact vehicle arrival times allows one to determine the precise values of the performance measures. Furthermore, using discrete time intervals places a restriction upon the signal changes which unnecessarily increases the performance measure values. Consider the following extreme example: in a 10-minute period, there is one arrival on each approach during the same five-second interval. OPAC-1 allocates the green status to one of the approaches and then requires five seconds to change the signal status. In total, there is an average of 7.5 seconds of delay incurred (in my version of OPAC-1, I assume the delayed vehicle was equally likely to have arrived at any point in its arrival interval which implies an average of 2.5 seconds of delay). In reality, the vehicle on the approach with the green could have arrived immediately after the start of its arrival interval and vehicle encountering the red signal could have arrived immediately before the end of its arrival interval. Knowing this information, the light could have been optimally scheduled to change immediately after the first vehicle cleared the intersection thus causing no more than 2 seconds of delay to be incurred.

The fixed-cycle optimal deviations (F-COD) algorithm considered in this section attempts to address some of the drawbacks associated with OPAC-1 strategy. Basically, this approach starts from a fixed-cycle strategy and determines "optimal" deviations from this schedule and so, it is not dynamic programming based, and it uses a continuous representation of time.

In the next subsection, I present the method for determining the optimal deviations from a fixed-cycle control schedule for a specified performance measure. Section 6.2.1 provides an analysis of F-COD in addition to bounds for the expected values of the
performance measures produced by the algorithm. Finally, I conclude this section with some experimental results obtained from using F-COD.

6.2.1 Fixed-Cycle Optimal Deviations (F-COD) Algorithm

As for the OPAC-1 strategy, it is assumed that the arrival data over given length of time is known in advance for both approaches. (Though the algorithm is given for two approaches, it can be easily modified to accommodate any number of approaches.) The algorithm superimposes a fixed-cycle control schedule over the arrival data. It then examines each individual signal change and determines the point between it's two adjacent signal changes at which there is the maximum improvement in the specified performance measure. In this manner, the optimal deviations from the original fixed-cycle schedule is sought.

Bearing this brief explanation in mind, I present the steps of the algorithm below followed by an example and a discussion.

F-COD ALGORITHM

Step 0: Choose the initial status of the signal and an initial cycle length C.

Step 1: Starting at time $t=0$, determine the initial set of signal changes for the interval $[0, T]$, where $T$ is the length of time under consideration. The time of signal change time $i$ is $i \cdot \frac{C}{2}$ for $i=0, \ldots, \lfloor \frac{T}{C/2} \rfloor - 1$. Calculate the value of the performance measure under the fixed-cycle schedule and denote it CURRENT_VALUE.

Step 2: For each signal change $i$, $i=1,2,\ldots,N-1$, determine the point $x$ which lies between signal change $i-1$ and signal change $i+1$ such that the specified performance measure is minimized over this interval. Set signal change $i$ equal to $x$.

Step 3: Determine the new value of the performance measure over the entire interval, denote this as NEW_VALUE.

Step 4: If $|NEW\_VALUE - CURRENT\_VALUE| < \varepsilon$ where $\varepsilon$ is a predetermined level of precision (usually on the order of $10^{-6}$) then stop, otherwise repeat from Step 2.

In the discussion of the individual steps of the algorithm, I use a concrete example. Consider the situation depicted in Figure 6.1. Assume that $C$ is equal to 60 seconds which implies that the length of the individual red and green periods are 30 seconds each. Consider the first signal change, denoted signal change 1. We want to select a point between signal change 0 and 2 such that the delay measure over the entire
interval from 0 to 2 is minimized. Note that the value of the delay measure over this interval is currently 124 seconds (incurred by arrivals during the red: \(a_2, a_3, a_5, a_8, a_{10}\)).

![Diagram of signal changes and arrivals]

Figure 6.1   Example of initial fixed-cycle assignment (step 1 of F-COD algorithm) superimposed over the arrival data.

Now we would like to determine the point of optimal deviation for signal change \(\oplus\) from its current position. The only points which can be considered are those between signal change \(\ominus\) and signal change \(\otimes\). After a little thought, it can been seen that this point will occur at an arrival point and furthermore, as is argued below, only arrival points on the approach which has the green signal immediately before the signal change under examination need be considered (if they exist).

From Figure 6.1, note that if signal change \(\oplus\) is moved to the left, there is continuous improvement in the delay measure (i.e. it is decreasing) until one reaches arrival \(a_4\) (a green arrival point). If the signal change is moved just beyond \(a_4\), there is an jump increase in the delay measure of 18+30=48 because \(a_4\) now becomes a red arrival. It is only worthwhile to move the signal change beyond \(a_4\) if the additional decrease in delay that is obtained exceeds the initial increase in delay. At each green arrival point in this direction, there will be a similar jump increase. If the signal change is moved beyond a green arrival point, it will be moved back to the next green arrival point because the delay measure decreases until that point. Note if there were no
green arrivals to the left of the signal change, the optimal move to the left is to move the signal change all the way back to the red arrival furthest away from the original signal change position\(^{10}\) (which is \(a_2\) in this case).

If the signal change were moved to the right, there is an immediate increase in the value of the delay measure. The delay value will continue to increase until one reaches arrival \(a_7\). If the signal change were moved just beyond \(a_7\), there is a jump decrease in total delay because \(a_7\) now becomes a green arrival. This implies that if the signal change were moved in this direction, it would be moved to a red arrival (which is an arrival on the approach which has the green signal immediately before the original signal change position). If there are no red arrivals in this direction before signal change \(\oplus\), then signal change \(\circ\) would not be moved in this direction.

Figure 6.2 displays the behavior of the delay measure over the interval between signal change 0 and signal change \(\oplus\). From this figure, it can be seen that the optimal place to move signal change \(\circ\) is to \(a_4\). The new delay level for this interval is 65.

---

\(^{10}\) If there are no arrivals, either green or red, then the signal change will not be moved under the algorithm.
This process is continued in a similar manner. Signal change $\odot$ is considered next. Only the arrival points on approach 2 between signal change $\odot$ and the new position of signal change $\odot$ need to be considered as potential positions for signal change $\odot$.

A few observations and comments regarding the general mechanics of the algorithm can be made. First, since the initial status of the signal only affects the number of red and green intervals per approach by one, the initial status of the signal has no effect on the expected value of the performance measures produced by the algorithm over many runs. Even for a single realization of the Poisson arrivals, the differences between the performance measures for the two different initial signal status assignments is not significant.

Secondly, the algorithm, as given, assumes that equal proportions of the cycle length, $C$, are initially assigned as red and green periods for each approach. This assumption has an insignificant effect on the values of the performance measures produced by the algorithm. The reasons for the non-effect are that the initial cycles are all of the same length and because a signal change can only be moved to a new position bounded by its adjacent signal changes. A parameter which has a first-order effect on the final performance value is the number of signal changes per minute which is determined by the initial cycle length, $C$.

In the next subsection, I provide a brief analysis of F-COD.

### 6.2.2 Analysis of the Fixed-Cycle Optimal Deviations (F-COD) Algorithm

In this section, I furnish bounds for the final performance measure values produced by the algorithm. These bounds provide a quick way to estimate a lower bound for the expected value of perfect information for any intersection of interest.

Assume that an intersection has two approaches, 1 and 2. As usual, let $\lambda_i$ denote the average Poisson arrival rate for approach $i$ ($i = 1, 2$). Then the overall average arrival rate for the intersection, $\lambda$, is $\lambda_1 + \lambda_2$. As before, $C$ is the length of the initial cycle time in minutes and so the length of the green period and red period for each approach is $\frac{C}{2}$. Let $w(\lambda, C)$ be the average wait of a random vehicle. This quantity is required to calculate the values of the performance measures.

The value of $w(\lambda, C)$ can be easily determined for the initial fixed-cycle schedule of signal changes. For a given approach $i$, the expected wait of a vehicle which arrives in a green interval is 0 and the expected wait of a vehicle which arrives during a red
period is \( \frac{C}{4} \). The probability the vehicle arrives in a red (green) interval is \( \frac{1}{2} \) and so \( w(\lambda, C) = \frac{C}{8} \).

The expected total amount of delay per cycle under the initial fixed-cycle assignment, is \( \frac{\lambda C^2}{8} \) because \( \lambda C \) is the expected number of arrivals per cycle. Finally, to simplify the algebra, assume that \( T \), the length of the entire time period under inspection, is a multiple of \( C \). This implies that \( \frac{T}{C} \) is the number of cycles in a period of length \( T \) and so the total expected delay value, \( D(\lambda, C, T) \) over the interval of length \( T \) is

\[
D(\lambda, C, T) = \frac{\lambda T C}{8}.
\]  

(6.9)

The expected number of stops which result from the initial fixed-cycle schedule is just the expected number of stops per cycle times the number of cycles in time \( T \) or \( \lambda T \). Using this we obtain the values of \( F(\lambda, C, T) \) and \( P_i(\lambda, C, T) \) for \( i = \text{CO, HC or NO}_x \), the initial expected values of the fuel and pollution measures, respectively.

\[
F(\lambda, C, T) = \alpha e^{\frac{\lambda TC}{8}} + \beta e^{\lambda T}.
\]  

(6.10)

\[
P_i(\lambda, C, T) = \gamma_i e^{\frac{\lambda TC}{8}} + \delta_i e^{\lambda T} \quad \text{for} \quad i = \text{CO, HC, NO}_x.
\]  

(6.11)

Note that the initial value of all the measures are linear in \( C \).

Let \( w'(\lambda, C) \) be the expected wait per random vehicle at the completion of the algorithm. Before developing the bounds for the average values of the performance measures produced by the algorithm, let's first examine the value of \( w'(\lambda, C) \) for some limiting cases.

If the arrival rates for both approaches increase without bound, then \( \lim_{\lambda \to \infty} w'(\lambda, C) = w(\lambda, C) = \frac{C}{8} \). This says that if the number of arrivals per interval increase without bound for each approach, shifting will have no effect on the expected delay per vehicle because any proposed shift will yield at best a net change of zero in the average wait. On the other hand, as the arrival rates for the two approaches nears zero, then \( w'(\lambda, C) \to 0 \). In other words, as \( \lambda_1 \to 0 \) and \( \lambda_2 \to 0 \), arrivals become rare with respect to signal changes and so it is possible to arrange a green signal for each arrival thus eliminating all delay.

Now vary the parameter \( C \). If \( C \) approaches zero, this implies that the signal will change infinitely often in a finite period of time. Hence (because the algorithm does not explicitly model clearing times), it is possible to arrange a green for each arriving
vehicle and thus reduce \( w(\lambda, C) \) to zero. If \( C \to \infty \), then the cycle length becomes very long and so the number of signal changes approaches zero. This implies shifting has little or no effect on the average delay per random vehicle because there are no signal changes to shift and so \( \lim_{C \to \infty} w(\lambda, C) = \lim_{C \to \infty} w(\lambda, C) \to \infty \).

To summarize, as \( \lambda \to \infty \) or \( C \to \infty \), shifting the signal changes has no effect on \( w(\lambda, C) \) and hence \( w(\lambda, C) = w(\lambda, C) \). As \( \lambda \to 0 \) or \( C \to 0 \) it becomes possible to arrange a green period for each arrival and so \( w(\lambda, C) \to 0 \).

Next, I calculate bounds for the expected values of the performance measures produced by the algorithm. A trivial lower bound for the expected value of the performance measures calculated by the algorithm is zero. To determine a upper bound for the expected value of the measures, define the critical approach to be the approach which has the green indication immediately prior to the signal change under scrutiny. Upon examination of Figures 6.1 and 6.2, it can be seen that if the signal change is moved to the first critical approach arrival to the left (i.e. the signal status is switched earlier than originally scheduled), if it exists, there is always a non-negative improvement in all the performance measures. If there is no critical approach arrival to the left of the original signal change position, then the signal change can be moved to the first non-critical approach arrival after the previous adjacent signal change (if it exists) to obtain a non-negative improvement in all the performance measures. On the other hand, if the signal change is moved immediately to the right of its current position, there is always a non-positive increase in the performance measure. These facts furnish a method for determining an upper bound for the expected value of the total delay over \([0, T]\) yielded by F-COD.

To place an upper bound on the final values of the performance measures, I define a new algorithm A. The steps of the algorithm are given below.

**A Algorithm**

**Step 0:** Choose the initial status of the signal and an initial cycle length \( C \).

**Step 1:** Starting at time \( t = 0 \), determine the initial set of signal changes for the interval \([0, T]\), where \( T \) is the length of time under consideration. The signal change time

\[
i = i \cdot \frac{C}{2} \text{ for } i = 0, \ldots, \lceil \frac{T}{C/2} \rceil = N.
\]

**Step 2:** For each signal change \( i \), \( i = 1, 2, \ldots, N-1 \), if there is at least one critical approach arrival then move the signal change back to the closest critical approach arrival else if there are no critical approach arrivals and there is at least one non-critical approach arrival then move the signal change back to the first non-
critical approach arrival after signal change \( i-1 \) otherwise, do not move the signal change.

Obviously, the expected improvement in the performance measures will be less using algorithm A than obtained by using F-COD.

Let I-A denote the improvement in the total delay per signal change as a result of using algorithm A. To determine \( E[I-A] \) there are two cases per cycle. Case one is the situation in which approach 1 is the critical approach (denote the improvement in total delay for this case as \( A_1 \)). If we condition on \( g_1 \), the number of arrivals on approach 1 during the green interval, and \( r_2 \), the number of Poisson arrivals on approach 2 during a red interval, and \( x \), the amount of distance the signal change is shifted to the left, we get

\[
E(I-A_1 | r_2, g_1>0, x) = x^2 \left( \frac{1-x}{C} \right) .
\]

(6.11)

Unconditioning on \( r_2 \) and then \( x \) produces

\[
E(I-A_1 | g_1>0, x) = \frac{\lambda_2}{2} \left( 2C - xC - x^2 \right)
\]

(6.12)

and

\[
E(I-A_1 | g_1>0) = \frac{1}{1-e^{-\lambda_2 C}} \left( \frac{\lambda_2 C^2 e^{-\lambda_1 C}}{2\lambda_1^2} + \frac{\lambda_2}{2\lambda_1} + \frac{\lambda_2 e^{-\lambda_1 C}}{\lambda_1^2} - \frac{A_2}{\lambda_1^2} \right)
\]

(6.13)

respectively. The expected value of I-A_1 given that \( g_1=0 \) is \( \frac{\lambda_2 C^2}{8} \). Appropriately unconditioning on \( g_1 \) and combining the resulting expected values produces

\[
E(I-A_1) = \frac{\lambda_2}{\lambda_1^2} \left( C - \frac{1}{\lambda_1} \left( 1 - e^{-\lambda_1 C} \right) \right)
\]

(6.14)

An analogous argument for case 2 where the critical approach is approach 1 yields

\[
E(I-A_2) = \frac{\lambda_1}{\lambda_2^2} \left( C - \frac{1}{\lambda_2} \left( 1 - e^{-\lambda_2 C} \right) \right)
\]

(6.15)

Recognizing that the total expected improvement per cycle is just \( E[1-A_1] + E[1-A_2] \) yields

\[
E(I-A) = \frac{\lambda_1}{\lambda_2^2} \left( C - \frac{1}{\lambda_2} \left( 1 - e^{-\lambda_2 C} \right) \right) + \frac{\lambda_2}{\lambda_1^2} \left( C - \frac{1}{\lambda_1} \left( 1 - e^{-\lambda_1 C} \right) \right)
\]

(6.16)

Using equation (6.16), it is possible to calculate the expected improvement over the entire time interval \([0, T]\) for the total expected delay as the result of algorithm A.

\[
E\left( \text{improvement in total delay over [0,T] using A} \right) = \frac{C}{T} \left( \frac{\lambda_1}{\lambda_2} \left( C - \frac{1}{\lambda_2} \left( 1 - e^{-\lambda_2 C} \right) \right) + \frac{\lambda_2}{\lambda_1} \left( C - \frac{1}{\lambda_1} \left( 1 - e^{-\lambda_1 C} \right) \right) \right)
\]

(6.17)
Subtracting equation (6.17) from equation (6.9) yields an upper bound for \(D'(\lambda, C, T)\), the expected delay value produced by the F-COD algorithm.

\[
D'(\lambda, C, T) \leq \frac{\lambda_1 C}{8} \cdot \left( \frac{1}{\lambda_2} \left[ 1 - \frac{1}{\lambda_2} \left( 1 - e^{-\frac{\lambda_2 C}{2}} \right) \right] + \frac{1}{\lambda_1} \left[ 1 - e^{-\frac{\lambda_1 C}{2}} \right] \right) \quad (6.18)
\]

In order to find the similar values for the fuel consumption and pollution emissions measures, the expected number of stops saved by algorithm A must be found. Again, start with case 1: the critical approach is approach 1 (type 1 signal change). If we condition on the event that there was at least one green arrival on the critical approach during the cycle, \(g_1 > 0\), and on the amount the signal change is shifted, \(x\), we obtain,

\[
\frac{\text{number of stops saved per type 1 signal change using A}}{g_1 > 0, x} = \lambda_2 \cdot x \quad . \quad (6.19)
\]

Unconditioning on \(x\) produces

\[
\frac{\text{number of stops saved per type 1 signal change using A}}{g_1 > 0} = \frac{\lambda_2}{1 - e^{-\frac{\lambda_2 C}{2}}} \left( C \frac{\lambda_1 C}{2} - 1 - e^{-\frac{\lambda_1 C}{2}} + 1 \right) \quad . \quad (6.20)
\]

Realizing that the expected number of stops saved per a type 1 signal change when \(g_1\) equals zero is just \(\frac{\lambda_2 C}{2}\) and using the appropriate probabilities to combine the two conditional results yields

\[
\frac{\text{number of stops saved per type 1 signal change using A}}{g_1} = \frac{\lambda_2}{\lambda_1} \left( 1 - e^{-\frac{\lambda_1 C}{2}} \right) \quad . \quad (6.21)
\]

The result for a case 2 signal change (the critical approach is approach 2) is analogous. Adding the results for case 1 and case 2 produces the expected number of stops saved per cycle.

\[
\frac{\text{number of stops saved per cycle using A}}{g_1} = \frac{\lambda_2}{\lambda_1} \left( 1 - e^{-\frac{\lambda_1 C}{2}} \right) + \frac{\lambda_2}{\lambda_1} \left( 1 - e^{-\frac{\lambda_1 C}{2}} \right) \quad . \quad (6.22)
\]

Manipulating equations (6.10), (6.16) and (6.22) we can compute a bound for \(F'(\lambda, C, T)\), expected value of the fuel consumption over the entire interval \([0, T]\) produced by the F-COD algorithm.

\[
F'(\lambda, C, T) \leq \alpha \cdot \left( \frac{\lambda_1 C}{8} \cdot \left[ 1 - e^{-\frac{\lambda_2 C}{2}} \right] \right) + \beta \cdot \left( \frac{\lambda_2 C}{8} \cdot \left[ 1 - e^{-\frac{\lambda_1 C}{2}} \right] \right) \quad . \quad (6.23)
\]

Similarly, using equations (6.11), (6.16) and (6.22) we can compute the analogous bound for the expected values of the pollution measures.
\[ P_i(\lambda, C, T) \leq \gamma \cdot \left( \frac{ATC}{8} \cdot C \cdot \left( \frac{\lambda_1}{\lambda_2} \left[ \frac{C - \lambda_1}{\lambda_2} \left( 1 - e^{\frac{-\lambda_1 C}{2}} \right) + \frac{\lambda_2}{\lambda_1} \left( 1 - e^{\frac{-\lambda_2 C}{2}} \right) \right] \right) \right. \\
+ \delta \cdot \left( \frac{ATC}{8} \cdot C \cdot \left( \frac{\lambda_1}{\lambda_2} \left( 1 - e^{\frac{-\lambda_1 C}{2}} \right) + \frac{\lambda_2}{\lambda_1} \left( 1 - e^{\frac{-\lambda_2 C}{2}} \right) \right) \right) \)
\\
for \( i = \text{CO, HC, NOx} \). \hspace{1cm} (6.24)

These bounds can be used to provide a quick, back-of-the-envelope method for computing the expected value of perfect information for any intersection. The next subsection presents some experimental results from using the algorithm and compares the bounds computed above with the actual values obtained from running the algorithm.

### 6.2.3 F-COD Experimental Results

The F-COD algorithm was implemented on a Macintosh II. I coded the algorithm in the "C" language. As for the OPAC-1 strategy, I obtain results for both a lightly trafficked intersection and for a moderately heavy trafficked intersection. For each of the two different types of intersections, I performed 5 sets of 100 separate runs with each set corresponding to minimizing the value of each of the five performance measures, three of which are pollution measures. As before, each run was done for 10 minutes of generated Poisson arrival data. Each run provides a summary of all the performance measure values.

The F-COD results depend on the initially chosen cycle length \( C \). Clearly, if the initial \( C \) is chosen to be small, F-COD will yield smaller performance measures because there are a larger number of signal changes and this provides more opportunity to arrange green periods for a larger number of vehicles. Figures 6.3 and 6.4 depict the relationship between the initial total delay experienced by the vehicles and the length of the initial chosen \( C \) for the lightly trafficked and moderately trafficked intersections, respectively.

As reflected by equation (6.9), the experimental value of \( D(\lambda, C, T) \) is linear in \( C \). From Figures 6.3 and 6.4, it can be observed that as \( C \) becomes larger, the improvement between the \( D(\lambda, C, T) \) and \( D'(\lambda, C, T) \) decreases as is expected from the limiting case discussion in previous section. The same discussion also predicts that as \( \lambda \) increases, the improvement between the initial and final values of the expected delay decreases which is again supported by the results in Figures 6.3 and 6.4. Note that the same holds true for a comparison between the upper bound for \( D'(\lambda, C, T) \) and the observed value of \( D'(\lambda, C, T) \).
CHAPTER SIX. Values of the Performance Measures Under Perfect Information

![Comparison of Initial, the Upper Bound and F-COD E(total delay over [0,T]) Values Vs. Cycle Length](image1)

Figure 6.3 Comparison of $D(\lambda, C, T)$, the calculated upper bound for $D'(\lambda, C, T)$, and the experimental value for $D'(\lambda, C, T)$ using F-COD for a lightly trafficked intersection.

![Comparison of Initial, Upper Bound and F-COD $\varepsilon$(total delay over [0,T]) Values Vs. Cycle Length](image2)

Figure 6.4 Comparison of $D(\lambda, C, T)$, the calculated upper bound for $D'(\lambda, C, T)$, and the experimental value for $D'(\lambda, C, T)$ using F-COD for a moderately heavy trafficked intersection.
In general, for the lightly trafficked intersection, the percentage improvement in the expected delay is between 55% and 90% using F-COD. For the moderately trafficked intersection this improvement is between 30% and 60%. The percentage differences between the upper bound on the expected improvement obtained from using F-COD and the observed value is between 40% and 70%, and 20% and 40% for the lightly trafficked and moderately trafficked intersections, respectively.

As noted, when $C$ increases the algorithm produces smaller expected values for the performance measures in roughly a linear relationship. If $C$ is too small, the times between signal changes becomes unrealistically small. For this reason, I have chosen a cycle length of 60 seconds for the remainder of my results. This value is within the range of usually cycle lengths commonly used for fixed-cycle intersections. It is also towards the smaller end of the range which permits F-COD to produce better values.

Tables 6.3 and 6.4 are analogous to tables 6.1 and 6.2. They contain the expected values of the performance measures, along with their 95% confidence intervals, observed over 100 runs of F-COD for the lightly and moderately trafficked intersections, respectively.

<table>
<thead>
<tr>
<th>Minimized Performance Measure</th>
<th>E[Delay] (sec/10-min)</th>
<th>E[fuel consumption] (gal/10-min)</th>
<th>E[CO emissions] (g/10-min)</th>
<th>E[HC emissions] (g/10-min)</th>
<th>E[NOx emissions] (g/10-min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay</td>
<td>138.29 (128.6, 148.0)</td>
<td>0.117 (0.110, 0.123)</td>
<td>84.20 (79.17, 89.23)</td>
<td>5.32 (5.01, 5.64)</td>
<td>2.148 (2.021, 2.275)</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>191.19 (178.6, 203.8)</td>
<td>0.100 (0.095, 0.106)</td>
<td>80.965 (76.49, 85.44)</td>
<td>4.99 (4.72, 5.27)</td>
<td>1.788 (1.690, 1.887)</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>163.05 (152.5, 173.6)</td>
<td>0.101 (0.095, 0.117)</td>
<td>77.929 (73.52, 82.33)</td>
<td>4.85 (4.58, 5.13)</td>
<td>1.821 (1.715, 1.928)</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>169.88 (159.5, 180.3)</td>
<td>0.100 (0.095, 0.106)</td>
<td>78.427 (74.18, 82.67)</td>
<td>4.87 (4.61, 5.13)</td>
<td>1.805 (1.703, 1.908)</td>
</tr>
<tr>
<td>NOx Emissions</td>
<td>199.23 (186.6, 211.9)</td>
<td>0.101 (0.096, 0.107)</td>
<td>82.424 (77.95, 86.90)</td>
<td>5.08 (4.80, 5.35)</td>
<td>1.799 (1.700, 1.897)</td>
</tr>
</tbody>
</table>

Table 6.3 Summary of the expected values of the performance measures obtained using F-COD for the lightly trafficked intersection averaged across each set of 100 runs per measure minimized. The 95% confidence interval is given below each number.

One observation pertaining to the F-COD results in Tables 6.3 and 6.4 is that it is possible to have the minimum value for a given performance measure occur when that
particular measure is not being optimized. For example, in Table 6.3, the minimum expected value of the HC emissions measure occurs when the CO emissions are being minimized. An explanation for this phenomenon is that the F-COD algorithm performs a local optimization for over short intervals (i.e., between the two adjacent changes of the signal change under consideration) which does not guarantee that the final solution is globally optimal over the entire interval \([0, T]\).

<table>
<thead>
<tr>
<th>Minimized Performance Measure</th>
<th>(E[\text{delay}]) (sec/10-min)</th>
<th>(E[\text{fuel consumption}]) (gal/10-min)</th>
<th>(E[\text{CO emissions}]) (g/10-min)</th>
<th>(E[\text{HC emissions}]) (g/10-min)</th>
<th>(E[\text{NO}_x \text{ emissions}]) (g/10-min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay</td>
<td>675.37</td>
<td>0.485</td>
<td>362.09</td>
<td>22.72</td>
<td>8.854</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>892.73</td>
<td>0.424</td>
<td>352.61</td>
<td>21.63</td>
<td>7.503</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>769.23</td>
<td>0.422</td>
<td>336.88</td>
<td>20.84</td>
<td>7.566</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>790.71</td>
<td>0.420</td>
<td>338.08</td>
<td>20.87</td>
<td>7.504</td>
</tr>
<tr>
<td>(\text{NO}_x \text{ Emissions})</td>
<td>952.84</td>
<td>0.430</td>
<td>363.00</td>
<td>22.19</td>
<td>7.563</td>
</tr>
</tbody>
</table>

Table 6.4 Summary of the expected values of the performance measures obtained using F-COD for the moderately heavy trafficked intersection averaged across each set of 100 runs per measure minimized. The 95% confidence interval is given below each number.

The F-COD results also exhibit a significant difference between the performance measure values produced when using total delay as the optimization criterion and using any one of the other four measures as the optimization criterion. Again, if it is necessary to select a timing policy which only optimizes over one of the five performance measures, the CO emissions measure is arguably a better choice than the delay measure for the same reasons given in Section 6.1.2.

SECTION 6.3 Comparison of OPAC-1 and F-COD

In this section I briefly compare the characteristics of the two approaches for obtaining the expected values of the key measures under perfect information.
A major difference between F-COD and OPAC-1 is that F-COD is an heuristic while the OPAC-1 strategy is guaranteed to produce an optimal solution. As mentioned above, F-COD is not guaranteed to produce an optimal solution for a given number of cycle changes because it only performs a local search for the optimal improvement in the performance measure. As is true in most cases of heuristic versus exact solution methods, the loss of an optimality assurance is compensated for by an increase in speed; the F-COD algorithm is much faster than the OPAC-1 strategy. The time required to run F-COD on a Macintosh II fx for 10 minutes of arrival data is under 10 seconds while for the same amount of data, OPAC-1 requires about seven minutes to run. However, it should be noted that other versions of the OPAC-1 dynamic programming algorithm which, run faster than the results reported here, have been developed.

Besides running faster, F-COD has a set of easily computable upper bounds for the expected values of the performance measures it yields. Hence, it is possible to quickly obtain a rough idea for magnitude of the values of the measures which F-COD will produce without having to run a program. Unfortunately, there are no analogous bounds for the OPAC-1 strategy.

A comparison of the OPAC-1 experimental results (Tables 6.1 and 6.2) and F-COD results (Tables 6.3 and 6.4) immediately reveals that F-COD produces smaller values for the performance measures. There are several reasons for this difference. First, F-COD results depend upon the initial number of signals changes allowed: the more signal changes allowed, the smaller the values of the performance measures. Note that the F-COD cannot increase the number of changes above the initial amount which is determined by the chosen cycle length. On the other hand, OPAC-1 has no restriction on the number of signals changes it uses since for each interval, it determines whether or not to switch the signal. Given the F-COD dependence on the initial cycle length and no corresponding dependence in the OPAC-1 strategy, it is unclear which initial $C$ value is the most meaningful when comparing F-COD and OPAC-1 results.

The most obvious reason why F-COD produces smaller values is that it implicitly assumes that the entire queue on each approach instantaneously clears the intersection during a given green period. Therefore, F-COD does not include the delay experienced by vehicles in queue waiting for preceding vehicles to clear the intersection in the evaluation of the performance measures. Since OPAC-1 explicitly
provides a realistic model of vehicles clearing the intersection, it naturally produces larger values for the performance measures.

Finally, some of the decrease in F-COD values from the corresponding OPAC-1 values are a result of F-COD's continuous representation of time as discussed earlier.

An interesting phenomenon which is found in both the OPAC-1 and F-COD results is that both methods of evaluating the performance measures under perfect information show significant differences between the values yielded as a result of current practice, which is to minimize delay, and the values produced when any other performance index is optimized.

As a final consideration in comparing the two different approaches is that the cycle-based nature of the F-COD shows promise of becoming a foundation for a real-time adaptive control strategy for signal networks. The underlying cycle structure could be used as the entire network default cycle from which ebbs and flows are allowed to occur in localized areas. The use of a base cycle for the entire network would greatly reduce the risk of local gridlocks and other congestion problems which may arise when uncoordinated real-time control, such as a control strategy based upon OPAC-1, is used at the individual signals.

The next section briefly describes the usefulness of the ideal values of the performance measures. It also provides an upper bound on the maximum amount of savings that could be realized nationwide by implementing real-time adaptive control at all isolated intersections.

SECTION 6.4 National Implication of Perfect Information Results for Fixed-Cycle Intersections

Before beginning a detailed analysis of a new adaptive control strategy, the values of the performance measures under perfect information can be used to determine an upper bound on the amount the proposed control is worth. The expected value of perfect information, which is equal to the value of the performance measure of interest under perfect information minus the value of the measure under the current control scheme, can be used to screen proposed control strategies. If a proposed strategy costs more than the savings obtained from knowing perfect information over a reasonable amount of time, it can be eliminated from further consideration.
A question that can be asked is exactly how much are real-time adaptive control strategies really worth? In this section, I answer this questions in two different ways for fixed-cycle control. In the next subsection, I determine the expected value of perfect information for a single fixed-cycle intersection. I close the section and chapter with a bound on the absolute potential yearly savings that perfect information can offer across all isolated fixed-cycle intersections.

6.4.1 Expected Value of Perfect Information for a Lightly Trafficked Fixed-Cycle Intersection

The calculations which follow evaluate the worth of perfect information for an isolated intersection operating under fixed-cycle control. It is assumed that the arrival rates for approach 1 and 2 are 3 veh/min and 5 veh/min, respectively. To calculate the expected value of perfect information, the expected cost of operating the intersection under fixed-cycle control is calculated assuming the signal is timed to minimize delay. This quantity is then subtracted from the expected cost of operating the intersection assuming perfect information and that the signal is timed to minimize delay.

To evaluate the performance measures under fixed-cycle control, I use the results of MODEL2, the more realistic light traffic model, found in Section 5.4. Because the OPAC-1 strategy models the queue clearing aspect of the intersection and so more accurately models the operation of the intersection, I use it for the expected value of the performance measures under perfect information. It should also be noted that F-COD can produce unacceptably short intervals between signal changes given the current vehicular technology which makes it a unrealistic control strategy even under perfect information. Until crash avoidance systems and other such advances are made in vehicular technology, there is a necessary minimum duration between signals changes which must be observed for safety reasons. OPAC-1 is better able to model such restrictions.

Other assumptions I make are the same as those made in section 5.3.4: the cost of gas is $1.25 per gallon, an individual's time is worth $4.25, one-half the average hourly rate, and the average vehicle occupancy is 1.25 adults. My calculation presumes that the arrival rate to the intersection is constant over the entire day and the entire year. Though this is clearly an unrealistic assumption, it is reasonable for a first-order approximation if the average hourly arrival rate to the intersection is equal to the indicated rates. Given the above assumptions, a conservative estimate for the value of perfect information is $58,800 per year for an isolated, lightly trafficked fixed-cycle intersection. Hence, if a community were only interested in new signal control
strategies which had a payback horizon of one year or less, it should consider projects for this intersection which cost less than $58,800. Projects which cost considerably more than $58,000 can be ignored. Obviously, if the community's payback period is longer, then further calculations would be required.

The above calculation assumes a lightly trafficked intersection and ignores the lifetime annual benefits which would be accrue. This implies that for each fixed-cycle intersection, any adaptive real-time control strategy which costs less than $60,000 definitely merits further investigation if the desired payback horizon is of one year or less.

6.4.2 National Savings Implications

From the values of the performance measures for fixed-cycle control in Section 5.4, it is possible to estimate the daily potential savings realizable under perfect information with regard to these intersections. Again, because I assume that all these intersections are lightly trafficked and because I ignore the savings in the pollution measures, all the monetary values computed below should be viewed as lower bounds.

Using MODEL2 of Section 5.4, the expected daily amounts (assuming 24-hr days) of the key performance measures associated with the operation of the approximately 17,500 fixed-cycle isolated intersections are given in Table 6.5. This table also includes the ideal standards associated with the intersections under perfect information. The first two columns correspond to the current practice of minimizing delay. The last column contains the ideal values of the performance measures associated with minimizing CO emissions, an arguably more reasonable optimization criterion of the five performance measures of interest.

From Table 6.5, it can be seen that each performance measure can be improved drastically above the fixed-cycle control by knowing perfect information. Perfect information would reduce delay at least 75%, fuel at least 39%, CO emissions at least 54%, HC emissions at least 48%, and NOx emissions at least 35%.

Absolute savings under perfect information for delay is at least 440,000 veh-hr daily which implies a savings of $1.9 million dollars per day, assuming one adult per vehicle. Similarly, the potential savings in fuel under perfect information is at least 383,880 gal/day which translates to $0.5 M per day. A savings of 383,880 gallons of gasoline per day is equivalent to 20,300 barrels of crude oil per day which is about 0.2% of the U.S. daily import amount.
<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Fixed-Cycle Control (min delay)</th>
<th>Perfect Information (min delay)</th>
<th>Perfect Information (min CO emissions)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay (veh-hr/day)</td>
<td>580,720</td>
<td>140,390</td>
<td>171,870</td>
</tr>
<tr>
<td>Fuel Consumption (gal/day)</td>
<td>991,200</td>
<td>607,320</td>
<td>511,560</td>
</tr>
<tr>
<td>CO Emissions (kg/day)</td>
<td>901,040</td>
<td>413,150</td>
<td>370,420</td>
</tr>
<tr>
<td>HC Emissions (kg/day)</td>
<td>50,630</td>
<td>26,480</td>
<td>23,420</td>
</tr>
<tr>
<td>NO\textsubscript{x} Emissions (kg/day)</td>
<td>17,560</td>
<td>11,380</td>
<td>9,520</td>
</tr>
</tbody>
</table>

Table 6.5 Daily operating costs of a lightly trafficked intersection.

Lastly, examining the last two columns indicates that there exist substantial differences between a policy designed to minimize delay and one which minimizes CO emissions. If CO emissions were minimized under perfect information instead of delay, this would result in a 22% increase in delay and decreases in the other four measures ranging from 10% to 16%. This is not "proof" that CO emissions should be minimized instead of delay but rather an indication that it would be beneficial to weigh the tradeoffs between the five measures when developing new signal timing methods.

Today, the technology does not exist to realize the full savings in the performance measures offered by the knowledge of perfect information though at some point in the future, intelligent vehicle/highway systems may make these savings feasible. Given that the technology for perfect information is currently lacking, the next step is to evaluate the worth of the most real-time adaptive control strategies available today. This is what I investigate in the remainder of this thesis.
Chapter Seven

Two Smart Traffic Control Strategies

In this chapter I describe two smart traffic control strategies which can be implemented with current technology. Both systems possess a real-time adaptive control aspect through the use of detectors. The first is a variation of the most widely utilized isolated signal control strategy today: vehicle-actuated control. What distinguishes this as a smarter technique than the usual form of vehicle-actuation currently in use is that it uses two vehicle detectors per approach in its gap seeking logic. Note that this strategy still does not explicitly optimize the delay measure or any other performance measure. The second strategy does explicitly optimize a particular performance measure over a finite time horizon. This control technique is developed from the dynamic programming based OPAC-1 strategy; modifications have been made to OPAC-1 to create a more practical technique.

I begin with a description of the logic underlying vehicle-actuated control using two detectors per approach which was investigated by Wu [1990]. Next, I present some experimental results for a lightly and moderately heavy trafficked intersection using a simulator which she designed.

In Section 7.2, I describe the OPAC-1 based strategy developed by Gartner [1982]. This strategy was developed to minimize delay at an intersection and I modify it slightly so that any of the five performance measures of interest can be optimized.
This section also contains experimental results for the rolling horizon OPAC model, ROPAC.

I close the chapter with a brief quantitative and qualitative comparison of the two strategies.

SECTION 7.1 Two Detector Vehicle-Actuated Control

Using a traffic signal simulator which she designed for an isolated intersection, Wu [1990] compared four control techniques: fixed-cycle control, vehicle-actuated control with one detector per approach, vehicle-actuated control with two detectors per approach, and vehicle-actuated control with three detectors per approach. Her experimental results indicate that a vehicle-actuated strategy which utilizes two detectors per approach is the most effective control of the four tested. For a one week simulation of a hypothetical intersection, her results suggest that each of the five performance measures are improved between 20% and 35% over a fixed-cycle controller and approximately 15% to 25% over a conventional vehicle-actuated controller which essentially utilizes one detector per approach. It should be noted that the analysis done by Wu essentially only "optimizes" over the delay measure because vehicle-actuated control is mainly a gap seeking strategy.

7.1.1 Description of Vehicle-Actuated Control with Two Detectors

Under a vehicle-actuated strategy which utilizes one detector per approach in its control logic, the initial green interval is set so that a queue which spans the distance from the stop line to the detector has time to clear the intersection. This implies that the minimum length of any green phase is one initial green interval. Note that the further the detector is placed from the stop line, the longer the initial interval must be. To minimize the delay for vehicles on the approach with a red signal, the detector should be placed as close to the stop line as feasible. On the other hand, the detector is also used to extend the existing green phase so that more vehicles can pass through the intersection without interruption. In order to satisfy this objective, the detector should be placed a significant distance from the stop line. Clearly, there are tradeoffs among the performance measures associated with the location of the detector.
CHAPTER SEVEN. Two Smart Traffic Control Strategies

This naturally suggests the use of two detectors per approach as an attempt to
advantageously minimize these tradeoffs. Even though many vehicle-actuated
intersections do use two detectors per approach, the data from both detectors are
usually not explicitly incorporated into the control logic. Commonly, one detector is
placed at, or very near, the stop line to ensure that motorists do not become stranded
at a red signal. So in essence, these intersections only use data gained from the other
detector in their control logic and hence behave like vehicle-actuated control with one
detector per approach. Vehicle-actuation with two detectors per approach uses the
detector closest to the stop line to minimize the initial green interval and to possibly
shorten the green extension interval. With two detectors, it is possible to more
accurately ascertain if and when a vehicle clears the intersection. The detector
furthest from the stop line is primarily used to detect vehicles at an earlier point
allowing the significant gaps in the traffic stream to be identified.

It should be noted that the length of the minimum green interval is also constrained
by safety factors. Today, there is a commonly accepted range of the initial green
interval which is considered to provide adequate safety. As detectors become more
reliable and as vehicular technology advances, it is conceivable that this initial interval
can be shorten considerably. So even though Wu did not explicitly consider safety
margins when determining the minimum green interval, her results are still meaningful
because soon the technology may be available to allow the same safety standards to
be met with shorter initial green intervals.

Some advantages of vehicle-actuated control using two detectors per approach
identified by Wu are (1) the initial and vehicle-extension periods are minimal, (2) if no
vehicles in the direction with the green are approaching, a vehicle in the cross direction
can be detected earlier and thus receive a green indication earlier, and (3) a vehicle in
the green direction is more likely to be detected and allowed to pass through the
intersection without delay and without having to stop.

Wu found that it was best to place one detector near the stop line (within
approximately 20 to 40 feet) and the second a significant distance from the stop line
(within 100 to 300 feet). The actual optimal locations depend upon the arrival rates of
vehicles to the intersection and on the speed of the approaching vehicles. These
dependencies highlight a deficiency in this control technique which is discussed later.
7.1.2 Experimental Results

Even though Wu did produce results for all five performance measures, her work does not really optimize over any individual measure. Vehicle-actuated control does not attempt any explicit optimization but merely seeks to identify gaps in the traffic streams. The idea being that if a significant gap in a traffic stream can be recognized, this information can be used to shorten the current green in order to allow the vehicles on the approach with the red signal to use the intersection which would otherwise be unused. Again, there is no attempt to optimize over any specific measure. When determining the best maximum allowable green period and the best location for the detector(s), Wu only considered the delay measure. She selected the best detector locations and maximum green times based on the results of simulation runs using different values for these parameters and then she calculated the values of the other performance measures using the best delay settings.

Using the method described above, Wu discovered that vehicle-actuated strategies produce significant savings in the performance measures relative to fixed-cycle control. She found that the savings were more significant when the average arrival rates on the approaches are small. This agrees with previous work [Grace et al. 1964, Nip 1975]. Of the three vehicle-actuated schemes she tested, vehicle-actuation with two detectors per approach is the most effective controller for a large number of average arrival rates. The vehicle-actuated controller with three detectors did produce slightly better results in all performance measures but not enough to justify the additional cost.

For a lightly trafficked intersection, Wu's results indicate that a savings between 30% and 40% can be realized in all five performance measures by using vehicle-actuated control with two detectors per approach rather than one detector per approach. The savings achieved over fixed-cycle control are in the range of 40% to 50%. For a moderately trafficked intersection, the comparable savings are not as impressive. Wu found that there is a savings between 1% and 6% realizable with vehicle-actuation using two detectors per approach versus one detector per approach. The savings in the measures using vehicle-actuated with two detectors instead of fixed-cycle control are between 6% and 14%. The reason the savings are not as significant for more heavily trafficked intersections is that the controller behaves as a fixed-cycle controller a large portion of the time due to the maximum green constraint for each approach.
SECTION 7.2  Rolling Horizon OPAC Strategy (ROPAC)

7.2.1  Background

ROPAC\textsuperscript{11} was developed by Gartner [1982] as a new demand-responsive decentralized technique for isolated intersection control. It is based upon a modification of the OPAC-1 strategy. Although OPAC-1 produces a globally optimal result for the specified performance measure, it is clearly impractical as an on-line control strategy for several reasons. First and foremost, it requires an infeasible amount of future arrival information. Secondly, the extensive computational requirement makes it much too slow. Given the limitations of OPAC-1, Gartner developed a new strategy which he denotes OPAC-2.

OPAC-2 basically divides the optimization period into stages which are roughly of the same length of a typical cycle for fixed-cycle control. It is assumed that the arrival data for a given stage is known entirely in advance. Each stage is decomposed into $K$ discrete time units. For each of the $K$ subunits of a stage, a decision is made whether or not to switch the signal status. If a switch is made, all approaches have a red for the duration of the stage subunit. Within each stage, it is required that at least one signal change occurs and up to three changes are allowed. The algorithm determines the best such signal switching strategy for each stage under the specified performance measure. Though Gartner focuses on minimizing delay, the strategy can obviously be generalized to allow any of the other four measures to be used. Results for the delay measure produced by this algorithm are very close to optimal (OPAC-1 values) with a greatly reduced running time (see [Gartner 1982]).

Though the OPAC-2 strategy does significantly reduce the amount of required input data and the running time, it is still impractical as an on-line control strategy. As stated, the procedure assumes that the arrival history for an entire stage is known in advance. In reality, it is extremely difficult to obtain this amount of information with a high degree of accuracy. To circumvent this problem, Gartner developed the ROPAC strategy which uses OPAC-2 as a basic building block. This procedure is described next.

\textsuperscript{11} More recently, the ROPAC name has been changed. This strategy is now referred to as OPAC.
7.2.2 Description of ROPAC

The information required for the ROPAC strategy is also obtained through detectors. The detectors must be located at sites which ensure that the necessary amount of future information which is required can be obtained. In field tests performed by Farradyne Systems, Inc. under the auspices of the Federal Highway Administration, two detectors per lane were used. One detector was placed at the stop line and the other detector was placed a significant distance upstream (roughly 600 feet from the stop line) [Gartner, Tarnoff and Andrews 1991].

ROPAC is an acronym for rolling horizon OPAC. The rolling horizon concept is commonly used for production and inventory control problems. The logic underlying ROPAC is that the $K$ subintervals of each stage are categorized as two distinct types. The first $r$ subintervals are designated as the head and the remaining $K-r$ as the tail. $r$ is referred to as the roll period. It is assumed that it is possible to accurately determine the arrival pattern for the $r$ subintervals which comprise the head while the data for the tail of the stage is predicted by a chosen model. The principal idea is to determine the optimal switching sequence for the entire stage, as for OPAC-2, but only implement the policy for the subintervals which are in the head. Then actual data is obtained for the next $r$ periods following the last interval in the current head period and the procedure is repeated for the new resulting stage. The projection horizon has been rolled or shifted by $r$ subintervals to obtain a new stage. This entire process is repeated.

Now, all that is required is accurate data for the $r$ subintervals which comprised the head of the stage rather than the entire stage. The head is chosen to be short enough so that it is possible to obtain the required arrival information from detectors.

7.2.3 ROPAC Experimental Results

The ROPAC algorithm was implemented on a Macintosh II. I coded the algorithm in the "C" language. As usual, I obtain results for both a typical lightly trafficked intersection (Poisson average arrival rates of 3 veh/min and 5 veh/min for approach 1 and approach 2, respectively) and for a typical moderately heavy trafficked intersection (Poisson average arrival rates of 8 veh/min and 12 veh/min). For each of the two different types of intersections, I performed 5 sets of 100 separate runs. Each set of runs "optimized" over a different performance measure. Tables 7.1 and 7.2 contain the average values of the performance measures for the lightly and moderately heavy trafficked intersections, respectively. The highlighted boxes contain the value of the
performance measure over which the "optimization" is performed. For both types of intersections, I use a Poisson model, with the same average arrival rates as used to generate the actual arrivals, to predict the arrivals on each approach for the tail of the stage.

<table>
<thead>
<tr>
<th>Minimized Performance Measure</th>
<th>E[delay] (sec/10-min)</th>
<th>E[fuel consumption] (gal/10-min)</th>
<th>E[CO emissions] (g/10-min)</th>
<th>E[HC emissions] (g/10-min)</th>
<th>E[NO\textsubscript{x} emissions] (g/10-min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay</td>
<td>273.53 (256.5, 290.6)</td>
<td>0.273 (0.259, 0.286)</td>
<td>190.98 (181.3, 200.7)</td>
<td>12.16 (11.54, 12.77)</td>
<td>5.064 (4.816, 5.311)</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>397.30 (369.5, 425.06)</td>
<td>0.253 (0.240, 0.267)</td>
<td>194.51 (183.9, 205.1)</td>
<td>12.13 (11.48, 12.78)</td>
<td>4.592 (4.357, 4.826)</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>327.78 (307.3, 348.2)</td>
<td>0.253 (0.241, 0.265)</td>
<td>185.92 (176.7, 195.2)</td>
<td>11.70 (11.13, 12.28)</td>
<td>4.637 (4.415, 4.858)</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>339.38 (317.9, 360.8)</td>
<td>0.253 (0.241, 0.265)</td>
<td>187.83 (177.8, 196.5)</td>
<td>11.76 (11.18, 12.34)</td>
<td>4.621 (4.401, 4.842)</td>
</tr>
<tr>
<td>NO\textsubscript{x} Emissions</td>
<td>423.98 (393.5, 454.5)</td>
<td>0.255 (0.241, 0.268)</td>
<td>198.4 (187.4, 209.3)</td>
<td>12.33 (11.66, 13.00)</td>
<td>4.359 (4.359, 4.829)</td>
</tr>
</tbody>
</table>

Table 7.1  Summary of the expected values of the performance measures obtained using ROPAC for the lightly trafficked intersection averaged across each set of 100 runs per measure minimized. The 95% confidence interval is given below each number.

<table>
<thead>
<tr>
<th>Minimized Performance Measure</th>
<th>E[delay] (sec/10-min)</th>
<th>E[fuel consumption] (gal/10-min)</th>
<th>E[CO emissions] (g/10-min)</th>
<th>E[HC emissions] (g/10-min)</th>
<th>E[NO\textsubscript{x} emissions] (g/10-min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delay</td>
<td>3718.1 (3429.9, 4006.3)</td>
<td>1.539 (1.482, 1.596)</td>
<td>1335.8 (1269.0, 1402.5)</td>
<td>81.22 (77.37, 85.06)</td>
<td>26.82 (25.94, 27.70)</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>8970.98 (8168.7, 9773.2)</td>
<td>2.000 (1.899, 2.102)</td>
<td>2225.5 (2072.6, 2378.4)</td>
<td>120.96 (121.0, 138.0)</td>
<td>31.47 (30.13, 32.80)</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>4626.25 (4264.1, 4988.4)</td>
<td>1.619 (1.556, 1.682)</td>
<td>1489.6 (1410.8, 1568.4)</td>
<td>89.56 (85.07, 94.06)</td>
<td>27.62 (26.69, 28.56)</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>4991.78 (4591.6, 5391.9)</td>
<td>1.646 (1.582, 1.711)</td>
<td>1548.9 (1464.9, 1632.8)</td>
<td>92.74 (87.98, 97.50)</td>
<td>27.86 (26.93, 28.79)</td>
</tr>
<tr>
<td>NO\textsubscript{x} Emissions</td>
<td>13847.6 (12771, 14924)</td>
<td>2.430 (2.312, 2.549)</td>
<td>3052.6 (2857.4, 3247.8)</td>
<td>174.34 (163.6, 185.1)</td>
<td>15.82 (13.37, 17.26)</td>
</tr>
</tbody>
</table>

Table 7.2  Summary of the expected values of the performance measures obtained using ROPAC for the moderately heavy trafficked intersection averaged across each set of 100 runs per measure minimized. The 95% confidence interval is given below each number.
In addition to modifying the algorithm so that it is possible to optimize any one of the five performance measures, I made a few other changes. The algorithm I use does not have a maximum allowable green time for any of the approaches. In other words, the signal is capable of awarding the green to a given approach for any amount of time. Although the algorithm does require at least one signal change per stage, it is possible to have arbitrarily long green intervals because only the signal schedule for the head of the stage is implemented and there is no explicit maximum constraint for the time between signal changes. In practical applications, it would be necessary to have a maximum green interval because this implies a maximum red interval.

A first observation that can be made from the results in Table 7.1 is that ROPAC is not guaranteed to produce optimal answers. For example, when HC emissions are minimized, the mean value produced is not the minimum produced among all the performance measures. The fact that ROPAC does not produce the optimal values is not unexpected since there is no formal mechanism to ensure that an optimal solution is found. The algorithm optimizes over short local time intervals which is not a sufficient condition for global optimality.

A second interesting observation is analogous to one made in the previous chapter. It again appears that if one were allowed to choose only a single performance measure over which to optimize, choosing CO emissions seems to be a more reasonable choice than delay. As before, when CO emissions are minimized, the values produced for all the performance measures except delay are near their optimal values. Also, minimizing CO emissions produces a delay value closer to its optimum than would minimizing over any other measure except delay itself. On the other hand, when delay is minimized, the values of the other four measures differ significantly, in most cases, from their minimum values.

7.2.4 Unexpected Results for the Moderately Heavy Trafficked Intersection

The results for the moderately heavy trafficked intersection (Table 7.2) are initially surprising. The numbers clearly indicate that in terms of all the performance measures, it is preferable to optimize the delay measure. This is contrary to all the results obtained thus far.

A possible explanation for this phenomenon is that all other four measures associate an explicit cost with each vehicle which is forced to stop which is absent in the delay measure. As a result of the local optimization, these measures create
longer queues because over the short-term optimization horizon, they try to minimize the number of vehicles which must stop in order to minimize the value of the performance measure. However, in the global experience, the cost incurred by the longer queues outweigh the short term benefits of stopping fewer vehicles. This effect is seen more pronouncedly in the moderately heavy trafficked intersection because there are fewer gaps in the traffic streams. Gaps in the traffic streams allow the algorithm to empty out the queues and so the optimization over the short-term horizon does not have such a detrimental cumulative effect for the lightly trafficked intersection.

If I would have kept the original ROPAC feature of enforcing maximum green intervals, the aforementioned phenomenon would probably not be so pronounced. This also implies that in practice, this occurrence would not be so prominent.

This underscores an interesting point. In the future, when developing new adaptive control strategies, it seems important that all performance measures of interest are kept in mind. In this particular case, ROPAC was developed to minimize delay and when applied to the other performance measures in the moderately heavy trafficked situation, it does not perform well. When developing a real-time adaptive control which involves the five performance measures above, it probably would be a good idea to explicitly reflect the tradeoff between minimizing the number of stops in the short-term against and the length of the delay incurred in the global perspective.

SECTION 7.3 Comparison of ROPAC and Two Detector Vehicle-Actuated Control

In this section I briefly compare the characteristics of the two smart traffic control strategies. The first comparison is of a quantitative nature and it compares the savings obtainable under both approaches relative to fixed-cycle control. The second comparison is of a more qualitative nature; I compare the underlying assumptions and the operating characteristics of each type of control strategy.

7.3.1 Quantitative Comparison of the Results

When the optimal ROPAC results for each performance measure for a lightly trafficked intersection are compared to the optimal fixed-cycle results (MODEL2 of Chapter Five), it is seen that there is a savings of approximately 67% in total delay, 31% in fuel consumption and NOx emissions, and 41% for HC and CO emissions.
These are slightly better than the comparable savings achievable in these measures for vehicle-actuated control using two detectors per approach. The delay savings are of the same order of magnitude as those reported by Gartner [1982].

For a moderately trafficked intersection, the parallel savings for ROPAC above fixed-cycle control are 53% in total delay, 14% in HC and CO emissions, and 12% in fuel consumption and NOx emissions. This can be contrasted with savings of between 6% and 14% under vehicle-actuated control which uses two detectors per approach. Clearly, for more heavily trafficked intersections, the ROPAC strategy is superior to the vehicle-actuation strategy.

For both strategies, the achievable savings in the performance measures are greater for lightly trafficked intersections. This makes intuitive sense because there is a larger number of gaps in the traffic stream and the gaps tend to be longer. Both these gap characteristics can be advantageously used for lighter trafficked intersections. In general though, the difference between the ROPAC performance and the vehicle-actuated performance is greater for higher trafficked intersections. This observation agrees with the results of a field-test of the ROPAC strategy versus vehicle-actuated control [Gartner, Tarnoff and Andrews 1991].

7.3.2 Qualitative Comparison of the Results

Overall, a ROPAC controller has two clear advantages over a vehicle-actuated controller using two detectors per approach: ROPAC settings are not dependent upon the average arrival rates of the vehicular traffic and ROPAC attempts to optimize the specified performance measure.

The first advantage is important because traffic patterns for most intersections differ by time-of-day. For the vehicle-actuated controller using two detectors per approach, the optimal placement of the detector furthest away from the stop line depends on the average vehicular arrival rate on the given approach. Under extremely light traffic, Wu found that the placement of the detector should be approximately 320 feet from the stop line. Under heavy traffic conditions, the placement of the detector should be approximately 100 feet from the stop line. Hence, at some times throughout the day, the controller is not operating at maximum efficiency because the detector is not optimally located. On the other hand, ROPAC does not have this problem. All that is required for ROPAC is accurate data for the head of the stage (on the order of 15 seconds worth of future information). This also implies that as the long term traffic
patterns for the intersection shift, ROPAC is more capable of adjusting than is vehicle-actuated control.

Secondly, ROPAC attempts to use some optimization based logic in its switching decision. This appears to be better use of the real-time information than is made by vehicle-actuation. Vehicle-actuation can identify significant gaps in the traffic streams which is definitely beneficial in terms of improved performance, but information as to the size and timing of the gaps is lost. Also, ROPAC is able to distinguish a single arrival in one traffic stream from many arrivals in the conflicting traffic stream. In other words, through its optimization logic, ROPAC does not assign the same worth to a single vehicular arrival as it assigns to many vehicular arrivals over a common time horizon. On the other hand, a vehicle-actuated controller does assign the same worth to one arrival as to many arrivals over the same time period.

Finally, Gartner [1982] has suggested that maybe the greatest contribution of ROPAC is its flow model. ROPAC considers the entire projection horizon, a stage, in the optimization process which seems to make it more suitable for a demand-responsive decentralized flexibly-coordinated system than vehicle-actuated control. The ROPAC strategy may possess the capability to structure the flows in the network so that coordination can be preserved on one hand while taking advantage of the ever present variations in flows on the other. It is not clear whether this could be accomplished with vehicle-actuated controllers. Under vehicle-actuation, a stream of traffic which is released early on one approach may lose all the benefits at the next downstream signal.

In the next chapter, I determine the national cost-effectiveness of the two smart traffic control strategies discussed in this chapter.
Chapter Eight

Cost-Effectiveness of Smart Traffic Signal Control Strategies

Drawing upon the results of previous chapters, I establish first-order approximations for the cost-effectiveness of the two real-time adaptive control strategies for isolated intersections which are described in the previous chapter. I use two different evaluation techniques. The first approach provides first-order approximations of the cost-effectiveness in terms of the two monetarily quantifiable measures: delay and fuel. The other approach is to compare the benefits obtainable relative to the existing control strategy to real and hypothetical savings which have been documented for other vehicular-related policies such as the corporate fuel economy laws (CAFE), right-turn-on-red (RTOR), and the emissions standards for automobiles mandated by the Clean Air Act.

My primary goal is to ascertain whether or not smart traffic signals are cost-effective and if so, what is the approximate magnitude of the national savings. Because I lack the necessary data, I do not provide a highly accurate cost analysis but rather attempt to bound the actual cost-effectiveness from below. I then furnish a good conservative guess for the actual cost-effectiveness of the two smart technologies.

In Section 8.1, I briefly state the assumptions and techniques used to quantify the benefits from smart traffic signal technologies. The next section contains estimates for the potential cost-effectiveness of a vehicle-actuated controller with two detectors
per approach (smart V-A) versus each of the other existing technologies. Naturally, the magnitude of these savings depends upon the alternative existing technology and upon the traffic volume which passes through the intersection. I also perform an analysis of the cost-effectiveness of the ROPAC strategy which is presented in Section 8.3. For both smart technologies, I provide the savings associated with each of the five performance measures in addition to performing a monetary cost-benefit analysis.

Section 8.4 contains a comparison of the potential savings realizable from the use of smart controllers on a national scale to the magnitude of the savings resulting from past policy decisions which were designed to reduce fuel consumption, pollution emissions and delay.

Using the results of the expected value of perfect information techniques which were presented in Chapter Six, I bound the worth of future real-time traffic-responsive technologies. Obviously, this bound provides a guideline for the maximum price society would be willing to pay for "smarter" signal technologies.

In Section 8.6, I present some results pertaining to signal retiming programs which have been established in some areas of the country. The focus of these programs is to devise optimal timing schemes for coordinated signal networks within the current technology. This section also contains some speculations regarding more traffic responsive strategies for coordinated signal networks.

Finally, I close this chapter with a summary of my results for isolated intersections.

SECTION 8.1 Assumptions

For each cost-effectiveness analyses performed below, I make use of some general assumptions and techniques. These are described here.

From the survey results presented in Chapter Three, roughly 31.4% of the nation's 240,000 traffic signals, about 75,500 signals, are operated in isolation. With respect to developing timing plans for these signals, 85% of the respondents indicated that they explicitly use a delay performance measure. In addition, the majority of the timing plans for isolated intersections were developed manually. Communities which use computer packages to aid in designing timing plans use packages which primarily generate plans which minimize delay or maximize intersection capacity. In light of this
information, when establishing the bounds and estimates which follow, I assume that all current timing plans for isolated intersections were designed to minimize delay.

A second assumption I make is that each signal operates 18 hours per day. During the night, many communities operate signals in a flashing mode during periods when there is virtually no traffic, say from 12 am to 6 am. Whether or not a signal is operated in a flashing mode during these hours is not important because the sparseness of the traffic during these hours means the operation of the signals has little impact on the total operating cost.

For both types of smart traffic controllers evaluated below, I determine the cost-effectiveness for an intersection which only has two approaches. If the strategy appears to be cost-effective for this intersection, it will also be cost-effective for an intersection which has four or more approaches because the installation cost I use assumes at least four approaches into the intersection.

As usual, when I refer to a lightly trafficked intersection, I assume the average Poisson arrival rates are 5 veh/min for one approach and 3 veh/min for the other. For the moderately heavy trafficked intersection, I assume the average arrival rates for the two approaches are 12 veh/min and 8 veh/min.

In some of the analyses, I assume that a "typical" intersection operates under some combination of lightly trafficked periods and moderately heavy trafficked periods. Because I lack accurate traffic flow information, it seems reasonable to approximate the operation of a typical intersection as a combination of these two types of intersections which have been used throughout my thesis.

Lastly, whenever the the monetary value of time and fuel are evaluated, I assume that time is valued at $4.25 per hour (one-half the average hourly wage rate, see Chapter Five) and a gallon of gas costs $1.25. In the evaluation of time savings, I only assume one adult per vehicle which provides a conservative estimate for the real value assuming it is correct to evaluate time at $4.25 per hour.

SECTION 8.2 Cost-Effectiveness of Vehicle-Actuation Using Two Detectors per Approach

In the analysis which follows, I estimate the cost of a fixed-cycle controller to be approximately twenty thousand dollars [Homburger and Kell 1988]. A vehicle-actuated controller with one detector per approach costs approximately twice as much
and a vehicle-actuated controller with two detectors per approach costs approximately sixty thousand dollars. These costs cover the *entire* installation including the controller cabinet, poles, signal heads, and wiring. They also assume a simple intersection configuration with at least four approaches.

### 8.2.1 Cost-Effectiveness Relative to a Fixed-Cycle Controller for a Lightly Trafficked Intersection

Using the results from MODEL2, the values of each of the five performance measures are given in Table 8.1 for a single intersection. These values assume that the intersection is lightly trafficked for the entire 18-hour day and the usual practice of minimizing delay.

<table>
<thead>
<tr>
<th>PERFORMANCE MEASURE</th>
<th>DAILY RATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Delay</td>
<td>24.9 veh-hr/18-hr day</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>42.5 gal/18-hr day</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>38.6 kg/18-hr day</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>2.2 kg/18-hr day</td>
</tr>
<tr>
<td>NO\textsubscript{x} Emissions</td>
<td>0.8 kg/18-hr day</td>
</tr>
</tbody>
</table>

Table 8.1 The daily amounts of each of the five performance measures under fixed-cycle control at a lightly trafficked intersection.

The two measures which are relatively easy to price are the delay measure and the fuel measure. The daily cost of operating a fixed-cycle signal at an isolated, lightly trafficked intersection in terms of delay is $105.80. The daily cost associated with the operation of the intersection in terms of the fuel consumed is $53.10. Thus, the total direct cost of operating this type of intersection is at least $158.90 per 18-hour day. It should be emphasized that this amount does not attempt to price the indirect costs associated with the effects of the pollution generated.

To determine the comparable costs of operating a smart V-A signal, I use results obtained by Wu [1990]. Using the simulator she designed, she simulated an intersection under both vehicle-actuated control and fixed-cycle control. Comparing the two values, she discovered a savings between 6% and 60% for each of the five measures. The higher percentage savings are associated with lighter trafficked intersections. Given this, I assume that under vehicle-actuation with two detectors, there is a savings in each of the measures of 30% for this lightly trafficked scenario. This implies the cost in terms of delay associated with the operation of the
intersection is $74.10 per day and terms of fuel it is $37.20 per day. The total direct
cost of operating the intersection under vehicle-actuated control with two detectors
per intersection is $111.30 per 18-hour day.

Hence, a daily savings of $47.60 can be gained from using smart V-A control
instead of fixed-cycle control. Assuming that the intersection is lightly trafficked 365
days of the year, the extra cost associated with the implementation of the smart V-A
control would be recouped in approximately 28 months or 2.3 years, assuming that
wages and the price of gas remain constant over this period.

8.2.2 Cost-Effectiveness Relative to a Fixed-Cycle Controller for a Moderately Heavy
Trafficked Intersection

Using the results from MODEL3, the values of each of the five performance
measures are given in Table 8.2 for a single intersection. These values assume that
the intersection experiences moderately heavy traffic for the entire 18-hour day.

<table>
<thead>
<tr>
<th>PERFORMANCE MEASURE</th>
<th>DAILY RATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Delay</td>
<td>237.2 veh-hr/18-hr day</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>204.4 gal/18-hr day</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>182.0 kg/18-hr day</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>11.0 kg/18-hr day</td>
</tr>
<tr>
<td>NO\textsubscript{X} Emissions</td>
<td>3.6 kg/18-hr day</td>
</tr>
</tbody>
</table>

Table 8.2 The daily amounts of each of the five performance measures under fixed-cycle control at
a moderately heavy trafficked intersection.

The total direct cost of operating this type of intersection is $1263.60 per 18-hour day.

To determine the comparable costs of operating a smart V-A signal, I assume that
the savings in the performance measures for the moderately heavy trafficked
intersection are approximately 12.5% based on Wu's results. This implies the total
cost associated with the operation of the intersection under smart V-A is $1105.70 per
18 hour day.

Thus the potential daily savings which can be gained from using smart V-A is on
the order of $157.90 per 18-hour day. Assuming that the traffic at the intersection is
moderately heavy 365 days of the year, the extra cost associated with the
implementation of the smart V-A control would be recouped in approximately 8
months.
8.2.3 Cost-Effectiveness Relative to a Fixed-Cycle Controller for a More Realistic Intersection

Obviously, neither of the previous scenarios are completely realistic. Most intersections experience a variation in traffic flow which is dependent on the time of day and the day of the week. In an attempt to capture this aspect of the intersection, I use a combination of lightly trafficked periods and moderately heavy trafficked periods.

A more typical intersection is hypothesized to experience two daily rush periods which last two hours each. To determine the values of the performance measures during these periods, I use the moderately heavy traffic scenario. For the remainder of the day, I assume that the traffic flow is light. Obviously, there are some isolated intersections which do not experience rush periods at any point in the day however, these intersections probably do experience higher average arrival rates than the lightly trafficked intersection during certain hours of the day. The operating costs of these intersections are probably closer to the typical intersection I propose here than either of the two extremes of the previous two subsections. Using the assumptions for this typical intersection and the results from MODEL2 and MODEL3, I compute the direct operating costs of both fixed-cycle control and the smart vehicle-actuated control.

In addition to assuming that the traffic volumes at the intersection vary by time of day, I also assume that they vary by day of week. Only weekdays are assumed to experience the four hours of moderately heavy traffic. On the weekends, I assume that the intersections only experience light traffic.

Under these assumptions, the direct cost of operating the signal under fixed-cycle control is $404.40 per 18-hour weekday and $158.90 per 18-hour weekend day. Similarly the costs associated with the operation of the smart vehicle-actuated controller is $332.20 per weekday and $111.30 per weekend day.

This implies that the total savings per weekday is $72.20 and for a day on the weekend it is $47.60. Assuming there are 260 weekdays per year, the yearly savings is approximately $23,770 which implies the extra cost associated with the installation of the smart V-A controller would be recouped in about 20 months.

8.2.4 Cost-Effectiveness Relative to a Vehicle-Actuated Controller with One Detector for the Three Scenarios

As was done for the fixed-cycle controller, vehicle-actuated control using one detector per approach can be compared to the smart V-A control.
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Under the scenario that the intersection is lightly trafficked the entire 18-hour day, there is approximately a 20% savings in the performance measures from using two detectors per approach instead of one [Wu 1990]. Hence, the daily savings from using two detectors is $27.80. The difference in the initial costs of the two control strategies is $20,000 which implies a payback period of less than 2 years or 24 months assuming that the price of an individual's time and the price of gas remain constant.

Assuming the intersection experiences moderately heavy traffic for the entire 18-hour day, there is approximately a 7% savings in the performance measures from using two detectors per approach versus one [Wu 1990]. For this scenario, the daily savings from using two detectors is $83.20. The yearly savings total to $30,370 which implies a payback period for the excess initial cost of the two detector strategy of less than 8 months.

Finally, for the more realistic intersection described in previous subsection, the daily savings per 18-hour weekday are $40.10 and $27.80 per 18-hour weekend day. Thus the total yearly savings are $13,350 which implies a payback period of less than 1.5 years or 18 months.

8.2.5 Implications of the Cost-Effectiveness Analysis

For the analysis above, I have assumed benefits for only two approaches and costs to equip four approaches and so all the initial payback times are upper bounds. Given this, there is a strong indication that the implementation of the vehicle-actuated control using two detectors per approach is cost-effective. In the worst case scenario, which is to assume that the intersection is lightly trafficked over the entire 18-hour day, it would take under 2.3 years to recoup the additional installation costs over fixed-cycle control and 2 years for vehicle-actuated control using one detector per approach. In addition to this, if the lifetime of the controller is predicted to be 10 years, there would be an additional 7.5 years of accrued benefits. For the more realistic scenario described in subsection 8.2.3, the time required to recoup the initial excess cost is only 18 months.

SECTION 8.3 Cost-Effectiveness of the ROPAC Strategy

Excluding the one time cost of developing a standard ROPAC software applicable to real intersections, the cost of equipping an individual intersection with a new
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controller required by the ROPAC strategy would be slightly higher than the controllers currently used for vehicle-actuated control because of the additional RAM required to run the ROPAC code. However, the most significant additional cost associated with the ROPAC strategy is that of the detector installations [Gartner, Tarnoff and Andrews 1991]. In the ROPAC field test, the conventional loop detectors which were used accounted for the majority of the ROPAC installation cost. The authors estimate that the excess initial cost to implement ROPAC versus a conventional vehicle-actuated controller using one detector per approach is approximately $30,000. This implies that the total projected cost to implement the ROPAC control strategy at an isolated intersection is approximately $70,000.

8.3.1 Cost-Effectiveness Relative to a Fixed-Cycle Controller for a Lightly Trafficked Intersection

Using the results from Table 7.1, the values of each of the key performance measures are summarized in Table 8.3 for a single intersection operated under the ROPAC strategy. These values assume that the intersection is lightly trafficked for the entire 18-hour day.

<table>
<thead>
<tr>
<th>PERFORMANCE MEASURE</th>
<th>DAILY RATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Delay</td>
<td>8.2 veh-hr/18-hr day</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>29.5 gal/18-hr day</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>20.6 kg/18-hr day</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>1.3 kg/18-hr day</td>
</tr>
<tr>
<td>NOX Emissions</td>
<td>0.5 kg/18-hr day</td>
</tr>
</tbody>
</table>

Table 8.3 The daily amounts of each of the five performance measures under ROPAC control at a lightly trafficked intersection.

The two measures which are relatively easy to price are the delay measure and the fuel measure. The daily cost of operating an isolated lightly trafficked intersection in terms of delay is $34.90. The daily cost associated with the operation the intersection in terms of the fuel consumed is $36.90. Thus the total direct cost of operating this type of intersection is at least $71.80 per 18-hour day. This amount ignores the indirect costs associated with the effects of the pollution generated.

Hence there is a daily possible savings of $87.10 to be gained from using ROPAC instead of fixed-cycle control. Assuming that the intersection is lightly trafficked 365 days of the year, the extra cost associated with the implementation of the ROPAC
control would be recouped in approximately 19 months, assuming that both the price of gas and wages remain constant over this period.

8.3.2 Cost-Effectiveness Relative to a Fixed-Cycle Controller for a Moderately Heavy Trafficked Intersection

Using the results from Table 7.2, the values of each of the five performance measures were computed and are presented in Table 8.4 for a single intersection operated under the ROPAC strategy. These values assume that the intersection experiences moderately heavy traffic over the entire 18-hour day.

<table>
<thead>
<tr>
<th>PERFORMANCE MEASURE</th>
<th>DAILY RATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Delay</td>
<td>111.5 veh-hr/18-hr day</td>
</tr>
<tr>
<td>Fuel Consumption</td>
<td>166.2 gal/18-hr day</td>
</tr>
<tr>
<td>CO Emissions</td>
<td>144.3 kg/18-hr day</td>
</tr>
<tr>
<td>HC Emissions</td>
<td>8.8 kg/18-hr day</td>
</tr>
<tr>
<td>NO\textsubscript{x} Emissions</td>
<td>2.9 kg/18-hr day</td>
</tr>
</tbody>
</table>

Table 8.4 The daily amounts of each of the five performance measures under ROPAC control at a moderately heavy trafficked intersection.

The direct cost of operating ROPAC at this type of intersection is $681.60 per 18-hour day.

Thus there is a daily savings of $582.00 which can be gained from using ROPAC instead of fixed-cycle control. Assuming that the intersection experiences moderately heavy traffic each of the 365 days of the year, the extra cost associated with the implementation of the ROPAC would be recouped in approximately 3 months.

8.3.3 Cost-Effectiveness Relative to a Fixed-Cycle Controller for a More Realistic Intersection

Using the intersection scenario developed in subsection 3.2.3, I determine a more realistic cost-effectiveness measure for ROPAC based upon a typical isolated intersection. Again, this intersection is assumed to operate under moderately heavy traffic conditions for 4 hours of the 18-hour weekday and under lightly trafficked conditions for the remainder of the time. For weekend days, I assume that the intersection experiences only lightly trafficked conditions.

Under these assumptions, the daily direct cost of operating the signal under ROPAC is $207.30 for an 18-hour weekday and $71.80 per 18-hour weekend day. The comparable costs associated with the operation of the fixed-cycle controller are $404.40 and $158.90, respectively.
This implies that the total savings per weekday is $197.10 and for a day on the weekend it is $87.10. Assuming there are 260 weekdays per year, the yearly savings is approximately $60,320 which implies the extra cost associated with the installation of the ROPAC controller would be recouped in about 10 months.

8.3.4 Cost-Effectiveness Relative to a Vehicle-Actuated Controller with One Detector for the Three Scenarios

As was done for the fixed-cycle controller, operating costs of vehicle-actuated control using one detector per approach are compared to those incurred under ROPAC control.

Under the scenario that the intersection is lightly trafficked the entire 18-hour day, using vehicle-actuation with one detector per approach yields a savings in the performance measures of approximately 30% compared to fixed-cycle control (subsection 8.2.1). Hence, the daily amount saved from using ROPAC instead of conventional vehicle-actuation is $39.50. The difference in the initial costs of the two control strategies is $30,000 which implies a payback period of less than 25 months assuming that the price of an individual's time and the price of gas remain constant.

Assuming the intersection experiences moderately heavy traffic for the entire 18-hour day, there is approximately a 12.5% savings in the performance measures from using conventional vehicle-actuation versus fixed-cycle control [Wu 1990]. For this scenario, the daily savings from using ROPAC is $424.10. The total yearly savings are $155,000 which implies a payback period for the excess initial cost of ROPAC strategy of less than 2 months.

Finally, for the more realistic intersection described in previous subsection, the daily savings per 18 hour weekday are $125.00 and $39.50 per 18 hour weekend day. Thus the total yearly savings are $36,865 which implies a payback period of approximately 10 months.

8.3.5 Implications of the Cost-Effectiveness Analysis

As in the previous section, the analysis above assumes benefits for only two approaches and costs for equipping four approaches and so all the initial payback times are conservative. Given this, there is a strong indication that the implementation of ROPAC is cost-effective. In the worst case scenario, which is to assume that the intersection is lightly trafficked over the entire 18-hour day, it would take less than approximately 20 months to recoup the additional installation costs over fixed-cycle control and 25 months for vehicle-actuated control using one detector
per approach. These are just the initial payback periods and they do not account for the additional years of benefits over the remaining lifetime of the controller. For the more realistic scenario intersection, the time required to recoup the initial excess costs is approximately 10 months relative to either type of existing technology.

SECTION 8.4 Projected Savings on the National Scale

In this section, I project the potential savings for each performance measure which are realizable under the two smart signal technologies for isolated intersections on the national scale using the calculations in the previous section. First, I estimate the possible monetary savings, and then I take a different approach and compare the potential improvements achievable under smart signal technologies to those achieved with federally legislated or adopted policies. Specifically, I compare the improvements gained in each measure from smart signals to those from right-turn-on-red (RTOR), the 55 mph speed limit, the corporate average fuel economy (CAFE) laws, and emissions controls for automobiles mandated by the Clean Air Act.

8.4.1 Estimate of National Savings for Each Performance Measure Under Smart Controllers

Based upon the survey results in Chapter Three, there are approximately 75,500 isolated intersections in the United States. Of these, roughly 17,500 are operated under fixed-cycle control and the other 58,000 are either semi-actuated or fully-actuated. For the purpose of my first-order approximations, I assume that all 58,000 actuated signals are conventional vehicle-actuated signals with one detector per approach.

To develop the expected values for each of the five measures, I use as a typical intersection the one introduced in subsection 8.2.3. Again, this intersection has two approaches and experiences moderately heavy traffic flow four hours out of an 18-hour day. The remainder of the day, it is assumed to experience light traffic conditions. These assumptions probably serve to underestimate the true values of the performance measures on a national scale.

Table 8.5 contains the daily amounts of each performance measure for fixed-cycle control, conventional vehicle-actuated (V-A) control, vehicle-actuated control using two detectors per approach (smart V-A), and the ROPAC strategy assuming lightly
trafficked conditions at the intersection. The analogous results are contained in Table 8.6 for a moderately heavy trafficked intersection.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Fixed-Cycle</th>
<th>Conventional V-A</th>
<th>Smart V-A</th>
<th>ROPAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>delay (veh-hr/18-hr day)</td>
<td>24.9</td>
<td>21.8</td>
<td>17.4</td>
<td>8.2</td>
</tr>
<tr>
<td>fuel (gal/18-hr day)</td>
<td>42.5</td>
<td>37.3</td>
<td>29.8</td>
<td>29.5</td>
</tr>
<tr>
<td>CO (kg/18-hr day)</td>
<td>38.6</td>
<td>33.8</td>
<td>27.0</td>
<td>20.6</td>
</tr>
<tr>
<td>HC (kg/18-hr day)</td>
<td>2.2</td>
<td>1.9</td>
<td>1.5</td>
<td>1.3</td>
</tr>
<tr>
<td>NOₓ (kg/18-hr day)</td>
<td>0.8</td>
<td>0.8</td>
<td>0.6</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 8.5  Daily amounts of each performance measure for the indicated control strategies at a lightly trafficked intersection.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Fixed-Cycle</th>
<th>Conventional V-A</th>
<th>Smart V-A</th>
<th>ROPAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>delay (veh-hr/18-hr day)</td>
<td>237.2</td>
<td>223.2</td>
<td>207.6</td>
<td>111.5</td>
</tr>
<tr>
<td>fuel (gal/18-hr day)</td>
<td>204.4</td>
<td>192.4</td>
<td>178.9</td>
<td>166.2</td>
</tr>
<tr>
<td>CO (kg/18-hr day)</td>
<td>182.0</td>
<td>171.3</td>
<td>159.3</td>
<td>144.3</td>
</tr>
<tr>
<td>HC (kg/18-hr day)</td>
<td>11.0</td>
<td>10.3</td>
<td>9.6</td>
<td>8.8</td>
</tr>
<tr>
<td>NOₓ (kg/18-hr day)</td>
<td>3.5</td>
<td>3.3</td>
<td>3.1</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Table 8.6  Daily amounts of each performance measure for the indicated control strategies at a moderately heavy trafficked intersection.

 Appropriately combining the results of Tables 8.5 and 8.6 produces the values in Table 8.7 which are the daily operational costs for a typical intersection in terms of each of the key performance measures.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Fixed-Cycle</th>
<th>Conventional V-A</th>
<th>Smart V-A</th>
<th>ROPAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>delay (veh-hr/18-hr day)</td>
<td>72.1</td>
<td>66.6</td>
<td>59.7</td>
<td>31.2</td>
</tr>
<tr>
<td>fuel (gal/18-hr day)</td>
<td>78.6</td>
<td>71.8</td>
<td>62.9</td>
<td>59.9</td>
</tr>
<tr>
<td>CO (kg/18-hr day)</td>
<td>70.5</td>
<td>64.4</td>
<td>56.4</td>
<td>48.1</td>
</tr>
<tr>
<td>HC (kg/18-hr day)</td>
<td>4.2</td>
<td>3.8</td>
<td>3.3</td>
<td>3.0</td>
</tr>
<tr>
<td>NOₓ (kg/18-hr day)</td>
<td>1.4</td>
<td>1.4</td>
<td>1.2</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 8.7  Daily amounts of each performance measure for the indicated control strategies at the "typical" intersection.
Finally, Table 8.8 contains the projected national daily savings in each performance measure under each of the two smart signal technologies for a "typical" intersection.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>V-A Two Detectors</th>
<th>ROPAC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Amt of Decrease</td>
<td>% Decrease</td>
</tr>
<tr>
<td>delay (veh-hr/18-hr day)</td>
<td>617,200</td>
<td>12.0</td>
</tr>
<tr>
<td>fuel (gal/18-hr day)</td>
<td>791,000</td>
<td>14.3</td>
</tr>
<tr>
<td>CO (kg/18-hr day)</td>
<td>710,800</td>
<td>14.3</td>
</tr>
<tr>
<td>HC (kg/18-hr day)</td>
<td>44,800</td>
<td>15.2</td>
</tr>
<tr>
<td>NOX (kg/18-hr day)</td>
<td>15,100</td>
<td>14.3</td>
</tr>
</tbody>
</table>

Table 8.8  National mean daily savings in each performance measure for the indicated smart control strategies based on the "typical" intersection.

It is of interest to note that the results in the above table show that ROPAC does not offer the same magnitude of percentage improvement for the other performance measures as it appears to for the delay measure. This seems to support the field observation that the ROPAC strategy may cause an increase in the number of stops experienced at an intersection [Gartner, Tarnoff and Andrews 1991].

8.4.2 Monetary Evaluation of the National Savings in Fuel and Delay

To calculate the dollar savings possible under each of the two smart signal strategies, I use the results in Table 8.8. The monetary value of the potential daily savings for an 18-hour weekday under smart V-A control is $3.6 million. The comparable dollar savings possible under ROPAC is $13.0 million. It has been determined in the previous section that the payback period of the excess initial costs for the ROPAC strategy is under one year and for smart V-A is under 18 months.

Both smart traffic signal technologies result in a significant amount of fuel savings. From Table 8.8, it is estimated that ROPAC could save approximately one million gallons of gas daily for an 18-hour weekday. This translates to a savings of 53,000 barrels of crude oil daily which is approximately 1% of the daily amount of crude oil imported to the United States [API 1990].

8.4.3 Comparison with RTOR and the 55 MPH Speed Limit

The first national energy crisis in 1973 produced at least two nationally accepted policies designed to alleviate the situation: RTOR and the federally legislated 55 mph speed limit.
The average reduction in stopped delay per right turn vehicle was 5.2 seconds per vehicle under RTOR [Parker 1976, Chang et al. 1977]. This compares to a potential savings of approximately 6.5 seconds per vehicle using ROPAC and 3.1 seconds using smart V-A relative to a lightly trafficked intersection operated under fixed-cycle control. Similarly, the comparable amounts for a moderately heavy trafficked intersection operated under fixed-cycle control are 20 seconds and 4.9 seconds for ROPAC and smart V-A, respectively. With respect to conventional V-A control, the savings for a lightly trafficked intersection are 5.2 seconds per vehicle under ROPAC and 1.8 seconds per vehicle under smart V-A. For the moderately heavy trafficked intersection the comparable savings are 18.6 seconds per vehicle and 2.6 seconds per vehicle under ROPAC and smart V-A, respectively. The magnitude of these savings compare favorably with those for RTOR.

Studies concerning the impact of RTOR have documented savings of 4% to 8.9% in fuel [Parker 1976, Chang et al. 1977]. Wagner [1980] estimates savings of 4.38 gallons per signalized intersection per weekday. From Table 8.7, the average daily potential savings in fuel at a typically trafficked fixed-cycle intersection is 18.7 gallons under ROPAC and 15.7 gallons under smart V-A. The comparable amounts for a typical trafficked conventional V-A intersection are 11.9 gallons under ROPAC and 8.9 gallons using smart V-A.

The potential savings in delay and fuel under ROPAC are substantially greater than those estimated for RTOR. Possible fuel savings are greater for smart V-A control than RTOR and although the delay savings per vehicle were less, there is a direct benefit to all vehicles under smart V-A. It is not surprising that the fuel savings are more significant under smart signal control strategies than for RTOR. RTOR merely reduces idling time and yields no direct reduction in the number of vehicles which must stop.

Another result of the 1973 energy crisis was the federally legislated 55 mile per hour speed limit. The primary motivating force for this law was fuel conservation. A secondary benefit of a 55 mph speed limit are the lives it saves. At the time the 55 mph law was enacted, the projected savings in fuel was 8.4 million gallons per day. This compares to the projected savings of 0.8 million and one million gallons of gasoline saved per day using smart vehicle-actuation and ROPAC, respectively.

It should be noted that the two aforementioned comparisons are not entirely valid. The direct implementation cost of both RTOR and the 55 mph speed limit were
negligible. On the other hand, a policy designed to encourage the implementation of smart traffic signals, which could yield savings in fuel and delay that are superior to those realized under RTOR and about an eighth of the fuel savings realized by the 55 mph speed limit, has a considerably large implementation cost associated with it.

8.4.4 Comparison with CAFE Laws

The corporate-average fuel economy laws (CAFE) are fuel economy standards for new cars mandated by the Federal Energy Policy and Conservation Act of 1975. The standards currently require that manufacturers maintain an average fuel efficiency of 27.5 miles per gallon in their foreign and domestic fleets. A federal task force estimated the fuel economy standards would save 1.5 million barrels of gasoline daily by 1985, or about 8% of the total daily oil consumption for all purposes. Since its passage, the average fuel efficiency of a new vehicle has doubled [Wright 1990]. Clearly, this policy has a substantially larger effect on fuel conservation than smart signal technology though, there are also some disadvantages associated with this energy policy.

One disadvantage of the CAFE law, which may not be as substantial for the smart signal technologies, is that it has not significantly lowered total fuel use; in recent years, even though fuel use per vehicle has gone down, total fuel use has risen. This in part can be explained by the fact that the average number of miles travelled per vehicle has increased. For example, in 1975 the average passenger car's gasoline mileage was 13.74 mpg and in 1989, it was 20.62. On the other hand, in 1975 the average number of miles travelled per vehicle was 9,406 and in 1989 it was 10,141 [API 1991]. One plausible explanation for this phenomenon is that the CAFE law lowers the cost of driving which encourages people to drive more miles. This, in part, offsets the impact of higher fuel economy.

A lower cost of driving also aggravates the pollution problem. Because the EPA has chosen to place a ceiling on the number of grams of pollutants that autos may emit per mile driven instead of per gallon of fuel burned, saving energy does not necessarily stem emissions [Khazzoom 1988]. As fuel efficiency of the car increases, there is no engineering principle that assures emissions are reduced. If better mpg is a result of making a car lighter, then lower emissions will probably occur. But economies in fuel can be achieved in other manners: type of fuel used, the chemistry of combustion, the engineering of the combustion chamber, pollution-control devises used, and the like. These other fuel efficiency techniques do not result in lowered
emissions. Currently, when fuel economy results in better emissions, it is possible for
the manufacturer to economize by relaxing emissions controls (down to the level set
by the standards) and thus increase its profits.

Hence, individuals who buy more fuel efficient cars may have a more technically
advanced car but it probably emits the same pollution per mile. Thus, if the owner's
behavior does not change, she may use less fuel but still emit the same amount of
pollutants. Worse, if the owner now drives more because it is cheaper for her to do
so, she may still use less gas but will produce more pollutants.

Another disadvantage, compared to smart signal technologies, is CAFE standards
affect a smaller segment of the energy market. Basically, they only affect new cars,
and more recently new light-weight trucks, which account for less than 10 percent of
gasoline consumption.

Finally, manufacturers may be forced to adopt more expensive technologies to meet
CAFE standards, driving up the price of new cars which provides an incentive for
drivers to keep their older, less fuel efficient models. The costs of smart signal
technologies, on the other hand, are indirect costs to motorists. All motorists would
be effected by both the costs and benefits of these technologies.

Hence, even though smart traffic signal technologies for isolated intersections are
unable to produce the same magnitude of fuel savings as CAFE standards, they
probably would not have their impacts offset as significantly as CAFE standards
have. Through adoption of smarter signal technologies, the travel time of a motorist
may be reduced overall for a given trip. This decrease in travel would probably not be
noticeable to motorists and so would not encourage motorists to drive more.
Furthermore, if the signals were timed to optimize some measure other than delay,
within a given signal technology, the average delay experienced per vehicle would
increase and perhaps add to driver frustration and thus discourage optional auto trips.
Finally smart signal technologies reduce both fuel consumption and pollution
emissions which arguably is not necessarily true for the CAFE law.

8.4.5 Comparison with Vehicle Emissions Standards

Up to this point, I have ignored the costs associated with air pollutants. It is known
that air pollution adversely affects soils, water, crops, vegetation, human-made
materials, buildings, wildlife, weather and climate, as well as reduces economic
values, personal comfort and well being [Elsom 1987]. The exact assessment of
these costs is extremely difficult.
As an attempt to gauge the relative effect that smart technologies have on reducing air pollutant emissions, I compare the projected savings from smart signals to those targeted under the Clean Air Act of 1970. In 1970, federal car emissions standards were set to begin in 1975 and 1976 for new cars, without the knowledge of whether the technology would be available to meet these standards. After some deadline extensions and standards modifications, an 80-94% decrease in pollution emitted by automobiles has been achieved today [Elsom 1987, Wright 1990].

Clearly, this reduction is substantially larger than that projected through the use of smart signals (Table 8.8). However, unlike RTOR and 55 mph speed limit, these gains were essentially paid for by motorists through the price of a new automobile. There are two costs associated with automobile emissions control: the price of the hardware required to meet the standards and a decrease in fuel efficiency.

The Committee on Motor Vehicle Emissions (CMVE) was established to report on the technological feasibility of achieving emission control standards established by the Clean Air Act. The CMVE interpreted feasible to mean emission control systems that meet standards at a reasonable cost. Based upon a 1974 CMVE report and on the Cost Benefit Committee report, Dewees [1978] shows that the increase in lifetime costs of a controlled automobile compared to an uncontrolled automobile is $844 in 1974 dollars which was deemed to be reasonable. This projected amount would pay for an average 90% decrease over all three pollutants which is roughly an average lifetime cost of $9 per percentage abatement and a marginal cost of $30 per percent decrease. He also found that the marginal cost increases as a function of pollution abatement.

A comparable quantity for the smart signals technologies can be calculated from the numbers in Table 8.8. As done in the study cited above, assume that equivalent savings in the different gasses can be treated equally. Under vehicle-actuated control the price per percentage decrease in pollution emissions is $1489 and for ROPAC, it is $1168. This is a cost per intersection and would be spread across all motorists living in a community. For example, if all isolated signals in a community which has 1000 signals and 1,000,000 drivers were replaced by ROPAC controlled signals versus the current technology at the intersection, the excess installation cost would be 10.9 million dollars. The average amount of pollution decrease which would be obtained is approximately 30% per pollutant. This would mean an average indirect cost per motorist of $10.88 to cover the excess replacement cost. To cover the entire
installation cost of ROPAC at all isolated intersections would mean an indirect cost of $21.70 per motorist. Notice that these costs also provide motorists with other benefits including a reduction in delay and fuel consumption, unlike the vehicle emissions standards.

Basically, the costs and benefits of pollution reduction achievable under smart signal technologies affects all motorists. The percentage reduction in emissions would be less than those produced by automobile pollution control devices. On the other hand, though the benefits of reduced emissions yielded by the emissions standards are enjoyed by all, the direct cost is borne by the motorists who purchase new automobiles.

SECTION 8.5 Bounds for Future "Smarter" Technologies

Given existing signal technologies and the available feasible smart technologies, one can develop bounds for the expected value of perfect information using each of these as a base. These bounds are useful as a quick means of evaluating whether or not proposed technologies warrant further investigation based on the maximum price that a community is willing or able to pay for additional gains yielded by real-time adaptive control.

In this section, I use the results of Chapter Six to develop bounds for the cost of new technologies for 1) the systems which are currently in place and 2) assuming that smart technologies have been implemented at all intersections.

8.5.1 Bounds Assuming Current Technologies

Table 8.9 contains the weekday average values of the performance values of OPAC-1 and F-COD based on a "typical" intersection. For this example, I have presented numbers for the current practice of minimizing delay. I have also included the values produced under perfect information when optimizing CO emissions, the arguably more reasonable single measure.

Using the values presented Table 8.9, those found in Tables 6.1 and 8.5, and assuming all isolated intersections are lightly trafficked on weekend days, I calculate the average annual savings in each of the performance measures assuming perfect information. The values found in Table 8.10 assume that the delay measure is being optimized under perfect information while the values in Table 8.11 assume that CO
emissions are optimized under perfect information. When evaluating the existing technologies, I use the values obtained when the delay measure is optimized because this is what is currently practiced. I provide Table 8.11 merely for comparison purposes.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Minimize Delay</th>
<th>Minimize CO Emissions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed-Cycle</td>
<td>V-A</td>
</tr>
<tr>
<td>delay (veh-hr/18-hr day)</td>
<td>72.1</td>
<td>66.6</td>
</tr>
<tr>
<td>fuel (gal/18-hr day)</td>
<td>78.6</td>
<td>71.8</td>
</tr>
<tr>
<td>CO (kg/18-hr day)</td>
<td>70.5</td>
<td>64.4</td>
</tr>
<tr>
<td>HC (kg/18-hr day)</td>
<td>4.2</td>
<td>3.8</td>
</tr>
<tr>
<td>NOX (kg/18-hr day)</td>
<td>1.4</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Table 8.9 Average national daily amounts of each performance measure for the indicated control strategies based on the "typical" intersection.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>OPAC-1 Savings</th>
<th>F-COD Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed-Cycle</td>
<td>V-A</td>
</tr>
<tr>
<td>delay (veh-hr/year)</td>
<td>13,650</td>
<td>11,900</td>
</tr>
<tr>
<td>fuel (gal/year)</td>
<td>7,370</td>
<td>5,060</td>
</tr>
<tr>
<td>CO (kg/year)</td>
<td>8,900</td>
<td>6,810</td>
</tr>
<tr>
<td>HC (kg/year)</td>
<td>475</td>
<td>340</td>
</tr>
<tr>
<td>NOX (kg/year)</td>
<td>135</td>
<td>135</td>
</tr>
</tbody>
</table>

Table 8.10 Average annual savings per intersection over existing technologies assuming perfect information and optimizing over the delay measure.

The values in the aforementioned tables can be utilized to determine whether or not a new technology is too expensive. For example, if a community values an hour of time at $4.25 per hour, gas at $1.25 per gallon, ignored pollution costs, and wanted a payback period for the entire installation cost of a year or less, then for an intersection which is currently operated under fixed-cycle control, the community would not need to consider any technologies which cost more than $103,000 under F-COD. If vehicular technology is along the same lines as current technology, i.e. drivers are in complete control of their vehicle and no crash avoidance systems are used, then the community need only consider projects priced under $67,000 (OPAC-1). These numbers are only
good as an initial screening test. If a project costs $65,000, this does not imply that it should be automatically accepted. It merely suggests that a more careful analysis is warranted before a final decision is made.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>OPAC-1 Savings</th>
<th>F-COD Savings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed-Cycle</td>
<td>V-A</td>
</tr>
<tr>
<td>delay (veh-hr/18-hr day)</td>
<td>13,050</td>
<td>11,290</td>
</tr>
<tr>
<td>fuel (gal/18-hr day)</td>
<td>9,030</td>
<td>6,710</td>
</tr>
<tr>
<td>CO (kg/18-hr day)</td>
<td>9,610</td>
<td>7,520</td>
</tr>
<tr>
<td>HC (kg/18-hr day)</td>
<td>540</td>
<td>410</td>
</tr>
<tr>
<td>NOx (kg/18-hr day)</td>
<td>170</td>
<td>170</td>
</tr>
</tbody>
</table>

Table 8.11 Average annual savings per intersection over existing technologies assuming perfect information and optimizing over CO emissions.

The same analysis for a vehicle-actuated intersection indicates the upper bound on the cost of the technologies worthy of further investigation are $57,000 for the current generation vehicular technology (OPAC-1) and $92,000 for the next generation vehicular and control technologies (F-COD).

Note that the costs of the two smart technologies considered in this thesis are less than the rough upper bounds determined in this subsection.

8.5.2 Bounds Assuming "Smart" Technologies

As was done in the previous section, it is possible to gauge the worth of new technologies using the currently feasible smart technologies as a base. Tables comparable to 8.9-8.11 can easily be generated for these technologies. Using only the delay and fuel consumption measures, the highest price for which a new technology merits further consideration is $44,000 for comparable vehicular technology (OPAC-1) and $80,000 for the next generation vehicular technology (F-COD) assuming smart V-A. The comparable numbers for ROPAC are $6850 and $42,500.

SECTION 8.6 Traffic-Responsive Coordinated Signal Networks

From the survey results in Chapter Three, there are an estimated 164,500 signals operated in coordination with other signals. This suggests that the greatest potential
CHAPTER EIGHT. Cost-Effectiveness of Smart Traffic Signal Control Strategies

for savings in each of the performance measures lies in developing "smart" or real-time traffic-responsive control algorithms for coordinated signal networks.

Savings in the performance measures can also be realized through optimization of the existing signal timing plans. Proper signal timing optimization requires a highly skilled traffic operations staff's repeated efforts. There is a great variation among jurisdictions in the quantity and quality of signal timing efforts. In too many places day-to-day pressures imposed on a limited traffic engineering staff often reduces the signal timing policy to one of "set it and forget it" [Wagner 1980].

This section first summarizes the results of some known signal retiming programs implemented by several states. The remainder of the section contains a brief discussion of future traffic-responsive control strategies for a system of coordinated signals.

8.6.1 Retiming Results

Traffic signal timing plan optimization is a very cost-effective policy. Wagner [1980] estimates its annual cost is between $300 to $400 per intersection which translates to only 3.5 to 4.7 cents of project cost for each gallon of fuel saved (evaluated in current fuel prices). Each dollar spent for optimization produces 21 to 29 gallons of fuel savings.

It has been estimated by the Federal Highway Administration (FHWA) that of the approximately 240,000 urban signalized intersections in the United States about 148,000 need upgrades in physical equipment and signal timing optimization while an additional 30,000 require timing optimization only [Wallis 1990]. As a result of this assessment, the National Signal Timing Optimization Project (NSTOP) was initiated by the FHWA in 1980. Its objective was to encourage states and municipalities to undertake signal retiming projects to improve the quality of urban driving and to thereby reduce fuel consumption. The FHWA's specific goal was for cities and states to develop and maintain signal plans for all coordinated and most uncoordinated signals in the U.S. The role the FHWA in this effort was to provide the tools and technical assistance that would enable cities and states to achieve this goal.

The results of the 11 NSTOP project cities showed impressive improvement in traffic performance from using TRANSYT-7F to develop optimal traffic signal timing plans. Conservative before and after estimates from TRANSYT-7F showed annual savings for the average intersection in the project to be 4500 gallons of fuel and 15,400 vehicle-hours of delay. Considering fuel savings alone, the average benefit/cost ratio
from the eleven cities was 10 to 1. When the value of time saved and stops eliminated were also considered, the benefit/cost ratio increased to 20 to 1 [Euler 1983, Wilbur 1985].

As of February 1989, there were at least ten states conducting signal timing optimization activities. These programs spanned an entire spectrum of state agency involvement. One end of the spectrum includes states which provide signal timing training programs and computing facilities for municipalities undertaking signal retiming projects. The actual work of devising and implementing new timing strategies is the responsibility of the locality. On the other end of the spectrum, the state’s transportation agency itself is responsible for the entire retiming program from the planning stages through implementation [Arnold 1989].

Four states have documented the fuel savings, the time savings, and costs of their programs. These states are California, Florida, Missouri and North Carolina.

The state agency that administers the California signal timing project is Caltrans. Caltrans has published statistics regarding the savings obtained from retiming. The numbers cited below are from a three-year experience report 1983-85 [Caltrans 1985] and from the 1991 California grant application manual for the FETSIM (Fuel-Efficient Traffic Signal Management) program [Caltrans 1990].

During the three year period from January 1983 through December 1985, California had retimed a total of 3,172 signals. These improved timings have reduced vehicular delay by 15 percent, decreased stops by 16 percent, and reduced overall travel times through the target systems by 7.2 percent. Fuel use has been cut by 8.6 percent. The reduction in fuel expenditures alone for the three-year experience was $72.3 million. Overall, the experience has an indicated benefit-cost ratio of 50:1. As a result of 8-years of experience (83-90), there have been 8,000 signals retimed with documented benefit-cost ratios again greater than 50 to 1. An average annual fuel savings of 4,000-6,000 gallons per intersection has been achieved. Additional benefits have included reduced air pollutant emissions and improved safety due to smoother traffic flow and fewer stops.

One specific city in California that has enjoyed large benefits from signal retiming is Los Angeles. Several weeks prior to the 1984 Olympic Games, an automated traffic surveillance and control system (ATSAC) was put into operation at 118 intersections. It has since been expanded to 450 intersections and by 1992 will include 1600 intersections [Rowe 1991]. According to the Los Angeles Transportation
Commission, the USC-Coliseum project saves motorists 13 percent of travel time, increases their average speed 15 percent, reduces fuel consumption 13 percent, and reduces air-polluting vehicle emissions 10 percent [Wallis 1990, Rowe 1991]. In a 1987 ATSAC evaluation report [LA DOT 1987], the average annual benefit per intersection was reported to be $66,500 with an annualized cost per intersection of $6,800 implying a savings of $9.80 for each dollar spent. The time required to recover the costs in terms of benefits to motorists is just 8.6 months of operation. Furthermore, annual operating costs of the system were reported to be recovered in the first week of operation per year.

Under the Missouri program run by the Missouri Division of Highway Safety, 161 signals have been retimed. Through simulation runs, it has been estimated that the average reduction in time delay is 21 percent, the average reduction in fuel consumption is 12 percent, and the reduction in the number of stops was 16 percent. This reflects an annual savings of $2.7 million implying a benefit-cost ratio of 32 to 1 [Missouri 1989].

In North Carolina, the signal retiming program is administered by the North Carolina Department of Transportation (NCDOT). For the thirty-two month period from October, 1987 to June, 1990, the benefit-cost ratio of the project was 28.2 to 1. Under the program, 1,755 signals had been retimed. The retiming produced a reduction in fuel consumption of 8.3 million gallons and an overall operating cost savings of $29.8 million. It is estimated that there still remains 1,125 signals to be retimed under the program [NCDOT 1990].

The retiming program in Florida involved 83 intersections. It has been calculated that the new timing schedules has resulted in 8,000 gallons of fuel saved annually with retiming costs per intersection of $900 to $1200 [Florida DOT 1990].

Other retiming programs which I was made aware of through additional literature provided to me by communities responding to my survey include those conducted by the state of New Hampshire, Beloit, Wisconsin, and Amarillo, Texas. In a 1982 study [Highway Traffic Consultants 1982] performed for the state of New Hampshire, a statewide savings of 2,215 veh-hours/day along with fuel savings of 1680 gallons per day were projected for 234 signalized intersections. A more recent report [Holden 1990] involves 136 major signalized intersections on the state highway system. It is estimated that a total yearly fuel savings of 463,000 gallons can be achieved. The cost
of the program is $68,750. Pricing gas at $1.00 per gallon clearly shows the cost-effectiveness of the program.

Beloit, Wisconsin received money from the state of Wisconsin to upgrade its signal hardware and to develop new signal timing plans. It was found that the approximate $240,000 total project cost produced an estimated $557,340 in annual benefits through the reduction of delays, stops and fuel consumption. The fuel consumption savings alone are estimated to be 79,500 gallons annually [Beloit 1990].

Finally, the city of Amarillo, Texas undertook the retiming of six arterial networks. In particular, the city performed a before and after coordination study for a specific artery. The reduction in stops and delay corresponds to an annual fuel savings of $92,170 assuming gas costs $1.30 per gallon. The initial cost of the program was $136,060 implying a payback period of 1.48 years. In terms of time savings, it was found that the new system could save motorists 35,930 hours per year. A significant savings no matter what value is placed on an individual's time [Amarillo 1990].

All the documented benefit-cost ratios are exceptional. Each of the programs cited above used off-the-shelf technologies, often not involving any additional hardware to be purchased. None of the strategies implemented are real-time traffic-responsive. Most of the studies priced gas at around $1.00 per gallon and valued an individual's time between $2 per hour and $8 per hour. The aforementioned studies all serve to show the importance of developing "optimal" timing plans within a given technology.

8.6.2 A Discussion of Past Real-Time Traffic-Responsive Strategies for Coordinated Signal Networks

Currently, the most successful and the most widely used traffic-responsive strategies for signal networks in the United States are those which select plans generated by 1-GC software based upon recorded traffic flow. These plans are chosen from a library of stored timing plans on the basis of the best match between the current traffic flow conditions and those of the stored plans.

There is only one established traffic-responsive control strategy world-wide today, SCOOT [Hunt et al. 1982, Chandler 1981, Robertson and Bretherton 1991]. Traffic surveys performed in Glasgow and Coventry show that, compared to a high quality fixed-time control strategy (TRANSYT), SCOOT reduced delay by an average of 12% during the working day. Because SCOOT plans do not "age" in the way typical of fixed-cycle plans, it follows that SCOOT should achieve savings in many practical situations of 20% or more depending upon the quality and age of the previous fixed-
cycle plan [Robertson and Bretherton 1991]. During hours which experience low traffic flows, SCOOT caused fewer stops than fixed-cycle control. It was demonstrated that SCOOT rapidly adapts to unusual variations in demand. Finally, an equally important benefit of SCOOT is that there is no need to periodically prepare new fixed-cycle plans because the signal timings are automatically kept up-to-date.

In the past, the lack of success of vehicle-responsive systems has been investigated and summarized [Gartner 1982, Hunt et al. 1982]. Some of the problems which have been identified include frequent plan changing, inadequate prediction, slow response, and effects of poor decisions. Most attempts of traffic-responsive control (2-GC and 3-GC) require that new plans be calculated online and implemented as soon as possible. Even the best methods of transition from one timing plan to another cause significant delay. Hunt et al. [1982] suggest that a new plan must operate for more than ten minutes to achieve overall benefit.

Inadequate prediction is a real problem. Gartner [1982] investigates the accuracy of predictor algorithms which have been used in 2-GC and 3-GC strategies. He shows that the discrepancies between predicted and measured volumes are substantial for normal operating days. They can often exceed the measured value by 30% to 50%. Obviously, the random variation in traffic makes it difficult to derive meaningful traffic responsive data through the use of inaccurate predictor algorithms.

Past traffic-responsive strategies have an inherent slow response time. The choice or generation of a new plan is based upon past events and not accurate information regarding future traffic conditions.

Finally, unexpected events, faulty detector data and/or inaccurate predicted traffic volumes may cause poor plans to be implemented which cannot be corrected until the next plan update.

8.6.3 A Discussion of Possible Future Real-Time Traffic-Responsive Strategies for Coordinated Signal Networks

Based upon the experience of the past studies described briefly in the previous subsection, Gartner [1985] constructed some guidelines for the development of traffic-responsive strategies:

1. The system must be designed to provide better performance than offline methods. This obvious requirement has been superseded in the development of some strategies of the past by less relevant criteria such as main street platoon progression or variable cycle time.
2. Development of new concepts is needed and not merely the extension of existing concepts. Effective responsiveness is not achieved by implementing offline methods at an increased frequency [see Section 2.3.6].

3. The method must be truly demand-responsive meaning it should adapt to actual traffic conditions and not to historical or predicted values.

4. It should not be arbitrarily restricted to control periods of specified length but should be capable of updating plans as frequently as necessary.

The SCOOT system has met most of these guidelines and as a consequence has met with success.

There is a fifth guideline that my work indicates should be added to the above list.

5. The system should be designed with explicit consideration of the tradeoffs which exist among the five performances measures: delay, fuel consumption, CO emissions, HC emissions and NO axe emissions.

Based upon my results for the timing plans for isolated intersections and given the need for fuel conservation and pollution reduction policies, it seems necessary to explicitly express the tradeoffs among the performance measures. As is seen with the ROPAC strategy (Section 7.2.4) which was designed to minimize delay, it can yield a worse value in a performance measure (other than delay) when asked to optimize it than the value produced when delay is optimized.

There have been other attempts made to satisfy the four guidelines above which may lay the groundwork for the development of usable traffic-responsive strategies. Gartner, Kaltenbach and Miyamoto [1983] studied the possibility of a decentralized, on-line traffic control for a signal network. The authors found that it is possible with such a system to achieve better performance than the best current fixed-cycle control methods.

Hall [1991] used a simulator, based on Wu's [1990] work for an isolated intersection, to investigate decentralized control strategies for one, two and four signalized intersection networks. The strategies he investigated include fixed-cycle, V-A with one detector per approach (conventional V-A), and V-A with two detectors per approach (smart V-A). For the three different network scenarios, Hall found that
smart V-A was the most effective type of control in terms of all the performance measures. She found that compared to a fixed-cycle control strategy for the networks, smart V-A could reduce the performance measures for the entire network by between 4% and 9%.

In future research related to expanding the demand-responsive aspect to signal networks, there may be several strategies which are best used in combination to match the characteristic of the network. Many different approaches should be investigated to develop simplified strategies that approach the optimal performance as closely as possible. Obviously, these strategies must be feasible in terms of real-time implementation and the amount of information regarding current and future traffic conditions required. Possible approaches that have been identified by Larson and Gartner for a UTC grant include:

1. Alternative demand-responsive strategies such as network dynamic programming, optimal deviations from a default pattern, hierarchical control and bi-level optimization.

2. "Stand alone" smart controllers that interact only from present time through to the tail of a rolling horizon.

3. Use of a structure which includes a pattern of "anchor" nodes in the network. These nodes have a default fixed-cycle length and force the remaining nodes into synchronization through a dynamic optimization process.

4. Use of dynamic subnetwork configurations to simplify the optimization. Possible configurations include linear subnetworks which correspond to arterials, independent signal operations, and various types of grid geometries.

5. A view from a network perspective, as possible alternatives or in combination, of isolated intersection control, arterial progression schemes, fixed-cycle network control and on-line optimization with a common or variable cycle.

The F-COD algorithm developed in Chapter Six may provide a basic building block for a new type of traffic-responsive control for a network and merits further research. It meets the five guidelines for the development of traffic-responsive strategies. It calculates the optimal deviations under perfect information relatively fast and because it is cycle-based, it naturally provides the basic coordination characteristics required in a network.
CHAPTER EIGHT. Cost-Effectiveness of Smart Traffic Signal Control Strategies

It is clear that traffic-responsive strategies have the potential to provide improvements in all the performance measures at least as good as those achieved for isolated intersections. They also have the potential to provide greater improvements in each of the measures than have been reported for the network signal retiming programs which have been undertaken by various local governments because these programs only make use of the current technology.

SECTION 8.7 Summary

In this chapter I examine the cost-effectiveness of two smart traffic control strategies for isolated intersections: smart V-A (vehicle-actuation which utilizes two detectors per approach) and ROPAC. My findings indicate that both strategies are cost-effective. All my calculations assume 18-hour days. Time is valued at $4.25 per hour and gas is priced at $1.25 per gallon. The calculations also assume that the intersections only have two traffic streams which makes my estimates conservative.

Tables 8.12-8.14 summarize the payback periods (Section 8.2) for the excess cost of installing each of the smart strategies versus the existing control techniques currently in place for each of the indicated traffic conditions which are assumed to exist for the entire year. The calculations also assume that the intersection has only two traffic streams in terms of the benefits but the cost is for equipping four traffic streams. Note that these are payback periods for excess initial installation costs between the indicated strategies and are upper bounds for the time required to recoup the cost of upgrading an existing control strategy to one of the smart strategies.

<table>
<thead>
<tr>
<th>Smart Technologies</th>
<th>Current Technologies</th>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Fixed-Cycle</td>
<td>Conventional V-A</td>
</tr>
<tr>
<td>Smart V-A</td>
<td>28 months</td>
<td>24 months</td>
</tr>
<tr>
<td>ROPAC</td>
<td>19 months</td>
<td>25 months</td>
</tr>
</tbody>
</table>

Table 8.12 Payback periods for the excess cost of installing the smart technology versus the current technology assuming light traffic conditions.
### Table 8.13  
Payback periods for the excess cost of installing the smart technology versus the current technology assuming moderately heavy traffic conditions.

<table>
<thead>
<tr>
<th>Smart Technologies</th>
<th>Current Technologies</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed-Cycle</td>
<td>Conventional V-A</td>
</tr>
<tr>
<td>Smart V-A</td>
<td>8 months</td>
<td>8 months</td>
</tr>
<tr>
<td>ROPAC</td>
<td>3 months</td>
<td>2 months</td>
</tr>
</tbody>
</table>

### Table 8.14  
Payback periods for the excess cost of installing the smart technology versus the current technology assuming "typical" traffic conditions.

<table>
<thead>
<tr>
<th>Smart Technologies</th>
<th>Current Technologies</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed-Cycle</td>
<td>Conventional V-A</td>
</tr>
<tr>
<td>Smart V-A</td>
<td>20 months</td>
<td>18 months</td>
</tr>
<tr>
<td>ROPAC</td>
<td>10 months</td>
<td>10 months</td>
</tr>
</tbody>
</table>

Assuming the most pessimistic situation, the payback period for both smart control technologies compared to either of the current technologies is under 2.5 years as a conservative estimate. For a more realistic situation, ROPAC's payback period is under 1 year given any type of current technology and smart V-A has a payback period of approximately 1.6 years.

The expected monetary savings (Section 8.4.2) which would result if all intersections were equipped with smart V-A is $3.6 million per day and for ROPAC, $13.0 million per day. Using the ROPAC strategy could save approximately 1 million gallons of gas daily and using smart V-A could result in an expected daily gas savings of 0.8 million gallons. These values account for roughly 1% of the U.S daily import amount of crude oil.

The nationally supported RTOR (Section 8.4.3) policy is estimated to save 4.38 gallons of gas per signalized intersection per weekday. The possible average daily savings in fuel at a "typical" fixed-cycle intersection is 18.7 gallons under ROPAC and 15.7 gallons under smart V-A and the comparable amounts relative to conventional V-A are 11.9 gallons and 8.9 gallons, respectively. The average delay savings realized under RTOR are estimated to be 5.2 seconds per right turn vehicle. This compares to potential savings of between 5.2 and 20 seconds per vehicle for ROPAC and 1.8 to 4.9 seconds per vehicle under smart V-A. Overall, the smart signal
strategies seems capable of delivering savings in fuel and delay greater than those realized under RTOR.

Fuel savings from the 55 mph speed limit (Section 8.4.3) have been estimated to be 8.4 million gallons per day. This compares to the projected savings of 0.8 million and one million gallons per day from smart V-A and ROPAC, respectively.

The projected fuel savings resulting from the CAFE standards (Section 8.4.4) are 1.5 million barrels of gasoline daily or about 8% of the total daily consumption for all purposes. This is substantially more than is realizable from the smart signal technologies investigated here. However, the CAFE standards have had some of these projected benefits offset because they have in effect lowered the cost of driving which may encourage people to drive more, and may even be the cause of more pollution. Smart technologies would probably not encourage individuals to drive more and smart signal technologies lower both fuel consumption and pollutant emissions simultaneously.

It is estimated that the pollution emissions standards (Section 8.4.5) decrease the amount of each pollutant emitted by 90% at a cost to the buyer of a new automobile of $844 (1974 dollars). This translates to an average lifetime cost of $9 per percentage abatement. Installation of smart technologies versus the current signal technology would have an excess cost of approximately $1489 and $1168 per percentage decrease in pollutant emitted per intersection under smart V-A and ROPAC, respectively. For the ROPAC strategy, assuming a community of size 1,000,000 with 1000 total signals, this translates to an indirect cost of $10.88 per motorist.

It is also possible to determine the maximum worth of smart signal technologies (Section 8.5) using the expected values of the performance measures under perfect information. If a technology costs more than this amount, it need not be considered. If a community desires a payback period of the excess installation cost of a smart signal technology less than a year, it need not consider technologies which cost more than $103,000 assuming an improvement in vehicular technology (such as crash avoidance systems) or which cost more than $67,000 assuming no change in vehicular technology for an intersection which is operated under fixed-cycle control. For an intersection which currently uses conventional vehicle-actuated control, the comparable numbers are $92,000 and $57,000.

Finally, Section 8.6 contains results that have been documented for signal retiming programs which have been implemented in some areas of the United States. The
benefit/cost ratios of these projects have been impressive. Fuel savings have been documented to be between 4,000-6,000 gallons per intersection, and overall travel times through the target systems have been reduced by 7% to 21%. Obviously, the savings depend upon the status of the systems before the retiming.

Clearly, traffic-responsive strategies for coordinated signal networks have the potential to provide improvements in all performance measures at least as large as those for isolated intersections and definitely as much as signal retiming programs have. Furthermore, strategies for coordinated network systems which are truly traffic-responsive would not require yearly retiming efforts which are expensive to perform in terms of time and money.
Chapter Nine

Conclusion

This thesis examines the cost-effectiveness of two smart traffic signals. These are signals which are made intelligent by sensors reporting real-time traffic conditions in the vicinity of the intersection and by their ability to adapt to these traffic conditions. The study is motivated by the substantial savings in delay, fuel consumption, and air pollution which can be obtained through the use of such smart signals. Hence, I develop expressions for five measures of interest as a function of the operation of an isolated traffic signal: expected delay, expected fuel consumption, expected CO emissions, expected HC emissions and expected NOx emissions. Using the performance measures and the results of a national survey of traffic engineers, I estimate the cost-effectiveness of two smart traffic signals and project the potential savings on a national scale.

As a secondary objective, I explore the tradeoffs among the performance measures of interest within a given technology. Since it is not possible to determine a signal timing plan which minimizes all five measures simultaneously, I construct a Pareto optimal set of timing plans in order to explore the tradeoffs among the measures for fixed-cycle control.

Also included in this thesis are two techniques for determining the values of the performance measures under perfect information: a discrete-time dynamic-programming based algorithm (OPAC-1), and a continuous-time heuristic which is not based on dynamic programming (F-COD). Such techniques are useful for calculating
the expected value of perfect information which can be used to bound the worth of new traffic-responsive strategies.

In this chapter, I highlight my major findings and suggest areas for future research. In the first section, I present a summary of my major results. Next, I discuss potential extensions of the current work. I close this chapter with some final comments.

SECTION 9.1 Summary of Results

My work can be grouped into three different parts. The first part is the exploration of the tradeoffs among the performance measures of interest within a given technology. The second part is an evaluation of the performance measures under perfect information using two different techniques, one of which is developed in this thesis. The final part is the investigation of the cost-effectiveness of two smart traffic signals. The summary of my results is divided along these lines.

9.1.1 Review of Tradeoff Results

Using the simplest form of traffic control, I illustrate the inherent tradeoffs between the five measures of interest. I developed three different models for the fixed-cycle signal: two models for light to moderate traffic conditions (MODEL1 and MODEL2) and one model for moderate to heavy traffic conditions (MODEL3). With each model, it is possible to determine the expected value of each measure for the indicated traffic conditions. MODEL1 is only used for illustrative purposes.

Using the results produced by the models, I determine the Pareto optimal set of timing plans. For fixed-cycle control, there are two parameters which must be specified by the traffic engineer: the cycle length and the proportion of green time allocated to each approach. Consistent across all models, it was discovered that the approach which experienced a smaller volume of traffic flow should be awarded a smaller fraction of green time to minimize the fuel measure or a pollution measure than the fraction which minimizes the delay measure. Similarly, it was discovered across all three models that for a given red-green split, the cycle length required to optimize the fuel measure or a pollution measure is longer than the cycle length which optimizes the delay measure.

There is a significant difference between the values of the performance measures when the delay measure is optimized compared to the values produced when any of
the other performance measures is optimized. An explanation for this occurrence is that all measures, except for the delay measure, explicitly associate a cost with each stop/start maneuver. Such a significant difference indicates that all performance measures should be explicitly considered when developing timing plans within a given technology.

Also from the fixed-cycle analysis, it appears that the current practice of minimizing delay is not rational from a monetary perspective. On the other hand, a policy designed to minimize fuel consumption does not appear to be rational either. The best monetary policy is probably some weighted function of all the measures. If a single measure must be chosen, the CO emissions measure is arguably a better choice over which to optimize than the delay measure. In general, a monetary evaluation is probably too simplistic because it does not reflect the indirect costs associated with the operation of the intersection.

9.1.2 Review of the Perfect Information Results

Under both of the perfect information techniques used in this thesis (OPAC-1 and F-COD), significant differences were again found between the value of a particular performance measure when it was optimized compared to its value when some other measure was optimized. Again, if a single measure had to be chosen over which to minimize, CO emissions is defendably the most reasonable choice. Such significant tradeoffs among the performance measures under perfect information indicates that all the measures should be explicitly considered in the development of new traffic-responsive technologies.

A comparison between the two perfect information techniques highlights some major differences. First, OPAC-1 is guaranteed to produce an optimal solution because it is a dynamic programming model. On the other hand, F-COD is an heuristic. F-COD performs local optimization which does not guarantee a globally optimal solution.

Although F-COD is not guaranteed to produce an optimal solution, it runs faster than OPAC-1. Additionally, a useful by-product of F-COD are easily computable upper bounds for the expected values of the performance measures. OPAC-1 does not suggest such straightforward bounds.

A final advantage of F-COD is its potential promise of becoming a foundation for a real-time adaptive control strategy for signal networks. Its underlying cycle structure could be used as the entire network default cycle from which ebbs and flows are
allowed to occur in localized areas. The use of a base cycle for the entire network would greatly reduce the risk of congestion problems which may arise when uncoordinated real-time control, such as one based upon OPAC-1, is used at individual intersections.

A disadvantage of F-COD is that it currently does not have a mechanism to guarantee acceptably safe intervals between signal changes. As it stands, F-COD results do not only assume perfect information but also assume perfect vehicular control. Until advances are made in vehicular technology, such as crash avoidance systems or computer driven vehicles, it is more reasonable to use OPAC-1 to determine the value of perfect information. OPAC-1 does guarantee safe intervals between consecutive signal changes.

The expected value of perfect information can be used in two ways. The first is to determine an upper bound for the worth of a traffic-responsive signal at any particular intersection and the second is to determine an upper bound for the national savings which can be gained through the use of smart signal technology. If either of these bounds are small, it is an indication that it may not be worthwhile to pursue smart signal technology and efforts should be concentrated elsewhere within intelligent vehicle/highway systems (IVHS) research.

As a conservative estimate, I found (using OPAC-1 results) that for a lightly trafficked fixed-cycle intersection, the worth of perfect information is approximately $60,000 per year. This estimate is conservative because it only values time savings and fuel savings, it only assumes benefits from two traffic streams, and it assumes the intersection is lightly trafficked the entire day.

On the national scale, I only determine the potential savings offered by perfect information for the approximately 17,500 fixed-cycle signals currently in existence in the United States. In terms of time savings and fuel savings, a conservative estimate of the monetary amount which could be saved daily from knowing perfect information is $2.4 million. It should be noted that fixed-cycle signals only account for 23% of all isolated signals and 7% of all signals. Clearly, there are substantial savings to be gained from traffic-responsive signal technologies.

Although I only used the results of the perfect information evaluations with respect to fixed-cycle control, such an analysis could also be applied to vehicle-actuated signalized intersections.
9.1.3 Review of the Cost-Effectiveness Results

There are two smart traffic signals strategies investigated in this thesis: vehicle-actuated control using two detectors per approach (smart V-A) and a rolling horizon control technique (ROPAC) derived from a dynamic-programming signal control algorithm.

The results under ROPAC, depend upon the traffic conditions. For a lightly trafficked intersection, the tradeoffs were significant among the measures when each of the the different measures were optimized. (Again, it seems that if a single measure must be chosen over which to optimize, the most reasonable choice is CO emissions.) However, for a moderately heavy trafficked intersection, it was found that when the delay measure was "optimized", it produced the best values for all five performance measures. A possible explanation for this phenomenon is that ROPAC was designed to minimize delay and does not penalize stops as do the other four measures. As a result of the local or short-term optimization horizon, ROPAC attempts to minimize the number of stops for the non-delay measures and in doing so, create longer queues. In the global experience, the cost incurred by the longer queues outweigh the short term benefit of stopping fewer vehicles. This effect is more pronounced in heavier traffic because there are fewer gaps in the traffic which allow the algorithm to empty the queues. This underscores the importance of explicitly considering all performance measures when developing a signal technology.

The other smart signal technology, vehicle-actuation using two detectors per approach, does not actively optimize any measure but merely a seeks significant gaps in the traffic streams. Hence, exploring the tradeoffs among the performance measures is not relevant under this type of control.

In the results presented below, all monetary values are in terms of the delay incurred and fuel consumed. The indirect costs, such as those associated with the generated pollution, are ignored.

One measure of the cost-effectiveness of two strategies is to determine the amount of time required to recoup the excess cost between installing the existing control strategy at the signal and a smart technology. It was found that for "typical" traffic conditions, the excess cost of installing ROPAC versus either fixed-cycle or convention vehicle-actuated control could be recouped in under 10 months. The excess cost of installing smart V-A versus fixed-cycle or conventional vehicle-actuated control could be recouped in under 20 months.
The annual savings associated with ROPAC control are $60,000 and $37,000 over fixed-cycle control and vehicle-actuated control, respectively. The analogous annual savings for smart V-A are $24,000 and $13,500.

Using the results of my national survey of traffic engineers, I project the savings associated with smart traffic signals on a national scale. The potential daily savings associated with the installation of smart signals at isolated intersections across the nation is $3.6 million for smart V-A and $13 million for ROPAC.

Comparisons were also made between the benefits of smart traffic signals and the benefits of four nationally sponsored policies: the right-turn-on-red (RTOR) law, the 55 mph speed limit, the corporate average fuel economy (CAFE) standards, and the vehicle emissions standards mandated under the Clean Air Act. It was found that the implementation of smart isolated traffic signals nationwide would produce savings in delay and fuel consumption greater than those produced under RTOR. Both the 55 mph speed limit and the CAFE standards have substantially larger fuel savings but at increased costs in other measures. The 55 mph speed limit increases travel time and the CAFE standards may inadvertently increase pollution emissions. The savings in fuel consumption as a result of the CAFE standards are partially offset because CAFE standards lower the cost of driving thus providing incentive for motorists to drive more. Finally, pollution emissions standards decrease emissions substantially more than would smart isolated signals but at a direct cost to the motorist through a new car sticker price and at the expense of decreased fuel efficiency. A distinct benefit of smart traffic signals is that improvements can be obtained in all five measures simultaneously at an indirect cost to the motorist.

SECTION 9.2 Extensions of the Current Work

Below I identify three major areas of future research: developing an appropriate performance measure for the evaluation of signal control strategies, developing a traffic-responsive strategy for a coordinated signal network, and identifying steps which would lead to the implementation of smart strategies on a national level.

Given that a monetary evaluation of the different performances measures is probably not the best mode of evaluation, future research could be done to ascertain an appropriate global measure which captures the tradeoffs among the individual measures. Such a measure would likely be in the form of a multivariate utility function.
CHAPTER NINE. Conclusion

Various global measures could be developed and analyzed for their reasonableness with regard to the numerous groups affected by signals. Some such groups include policy makers, commuters, commercial vehicle drivers, companies that rely heavily on road transportation, and environmental groups. In addition to determining which global measures are reasonable for each specific group, potential conflicts of interest between the groups could be identified. A final aspiration would be to determine a measure which seems the most suitable to society as a whole.

Based upon the fact that approximately 60% of the 240,000 signals in the United States are in coordinated networks, and given the impressive benefit-cost ratios realized through signal retiming programs (Section 8.6.1), which merely optimize timing plans for the existing technology, an obvious area for future research is to develop traffic-responsive strategies for coordinated signal networks. The potential savings which can be realized under such systems would be greater than those reported for the retiming programs which have been undertaken in local areas of the country. Today, there is only one type of traffic-responsive network system in operation world-wide. Currently in the United States there is no such system in operation.

When developing a new traffic-responsive system for networks, there are certain guidelines, constructed from mistakes in the past, which should be observed:

1. The system must be designed to provide better performance than offline methods.

2. Development of new concepts is needed and not merely the extension of existing concepts.

3. The method must be truly demand-responsive meaning it should adapt to actual traffic conditions and not to historical or predicted values.

4. It should not be arbitrarily restricted to control periods of specified length but should be capable of updating plans as frequently as necessary.

5. The system should be designed with explicit consideration of safety and the tradeoffs which exist among the key performance measures: delay, fuel consumption, CO emissions, HC emissions and NOx emissions.

In addition to following the above guidelines, the traffic-responsive system should be designed such that it will be used by traffic engineers. A common compliant of
traffic engineers in my survey is that current network signal generation techniques are too data intensive. Many departments do not have the labor nor time necessary to collect all the data required by the current state-of-the-art software package, TRANSYT. Furthermore, there does not exist and may never exist a package which generates signal timing plans that do not require fine-tuning in the field. Many engineers feel that solutions provided by the computer packages are a good starting point and not the final answer. Thus, it might be better to focus on developing techniques which do not require as much data collection and still produce plans which can be used as a good initial starting point. This even suggests that it may be worthwhile to develop a package and/or model designed specifically for what-if scenarios.

As mentioned previously, a potential building block for a new traffic-responsive signal system for a coordinated network is the F-COD algorithm developed in this thesis. This algorithm superimposes a fixed-cycle schedule over a given time horizon. It then attempts to determine the optimal deviations from this initial signal change pattern with respect to a specified performance measure. What makes this approach particularly attractive is that it calculates the optimal deviations relatively fast and because it is cycle-based, it naturally provides the basic coordination characteristics required in a network.

A final area of future research is to identify those organizational, political and procedural steps, on a national level, leading to implementation of recommended traffic-responsive policies. For instance, since the purchases of traffic control devices are usually made at the municipal level, there may be the situation where an extremely cost-effective national policy appears not to be economically feasible on a local level. This is may be particularly true in severely constrained tax-capped environments. Ultimately, if this were shown to be the case, mechanisms to reverse the conflicts of interest between local and national policies would have to be found.

SECTION 9.3 Closing Remarks

There are basically three final points I would like to make. The first is regarding the argument that the savings associated with the implementation of smart isolated signals are too small to matter. The second point stresses the importance of developing traffic-responsive signal technologies. The final remark is with regard to
the other approaches available for obtaining improvements in the key performance measures.

Urban automobile travel alone accounts for more than one-sixth of the total oil consumption of the United States and two-thirds of the vehicle miles travelled, making this sector one of the most, if not the most important in which to seek a higher quality of traffic flow. No traffic efficiency action is too small to matter. By their very nature, improvements in fuel efficiency and pollution abatement are composed of a large number of actions each contributing a small but essential share. Traffic-responsive signals can deliver a portion of this.

It should be remembered that traffic-responsive signals are an essential element of the IVHS of the future. They are also necessary today. It is known that proper signal timing optimization requires a highly skilled traffic operations staff's repeated efforts. From my survey results and the literature [Wagner 1980], in too many places day-to-day pressures and understaffing often reduce the signal timing policy to one of "set it and forget it". Even in jurisdictions with computer systems, a common syndrome is to devote highest priority to upkeeping the computer hardware and software operation and less priority to systematically and frequently updating timing plans to optimize traffic flow. Given that deteriorating timing plans can increase delay by 3% per year (ignoring the other measures), it is important to implement traffic-responsive strategies. These techniques lessen the burden of keeping timing plans updated.

Finally, my work here has stressed only one aspect of improving the chosen performance measures: through an improved quality of traffic flow. Equal emphasis should be given to increasing the use of more efficient travel modes and travel habits. In other words, effort should be directed towards reducing the total quantity of vehicular travel through ridesharing, public transit, self-powered modes, increasing the price of gasoline, and perhaps even the controversial policy of road pricing.
Appendix A

Derivations of the Quantities Used in the Pedestrian Crossing Example

Derivations of the quantities used in Chapter One are contained in this appendix. The notation and equation numbers used in both places are identical.

A.1 Derivation of the Common Quantities

To evaluate the three decision rules from the perspective of a pedestrian and from the perspective of a vehicle, it is necessary to determine, under each rule, the probability density function (pdf) for the length of the steady DON'T WALK signal and the expected time a randomly arriving pedestrian must wait before crossing. Denote the pdf's for the length of the steady DON'T WALK display as \( f_{IA}(t) \), \( f_{IB}(t) \), and \( f_{IC}(t) \) for rules A, B, and C, respectively. These quantities are determined first.

Under rule A, the length of the steady DON'T WALK is the deterministic quantity \( T_A \). This implies that \( f_{IA}(t) \) is an unit impulse function located at \( t = T_A \); the pdf is

\[
\hat{f}_{IA}(t) = \mu(t - T_A).
\]  \hspace{1cm} (1.1)

Rule B equates the length of a steady DON'T WALK to the amount of time up to and including the \( N_B \)th Poisson arrival. This is the \( N_B \)th-order interarrival time for a Poisson process with parameter \( \lambda \), a \( N_B \)th-order Erlang. Hence the pdf for \( t_a \) is
\[ f_N(t) = \frac{\lambda^N e^{-\lambda t} t^{N-1}}{(N-1)!} \quad \text{for } t \geq 0 \quad (1.2) \]

The length of the steady DON'T WALK display specified by rule C, \( t_C \), is equal to the amount of time from the end of the previous dump up to and including the arrival of the first pedestrian from the pooled Poisson process plus the deterministic amount \( T_C \). The time until the first Poisson pedestrian arrival since the end of the previous dump is described by exponential pdf with parameter \( \lambda \). It follows that the pdf for \( t_C \) is an exponential pdf which is shifted to the right by \( T_C \),

\[ f_{t_C}(t) = \lambda e^{-\lambda(t-T_C)} \quad \text{for } t \geq T_C \quad (1.3) \]

Now I determine the quantities \( \bar{w}_A \), \( \bar{w}_B \), and \( \bar{w}_C \), the mean waiting times of a randomly arriving pedestrian under rules A, B, and C, respectively. Under rule A, a steady DON'T WALK lasts \( T_A \) minutes and is followed by a dump of length \( D \) minutes. If a randomly arriving pedestrian must queue, he arrives during a steady DON'T WALK. His actual arrival time in this period is uniformly distributed over \([0, T_A]\) implying his waiting time until the next dump is initiated is also uniformly distributed over \([0, T_A]\). Thus if a pedestrian queues, his mean waiting time is \( \frac{T_A}{2} \). The probability that a randomly arriving pedestrian will have to queue equals the proportion of time in the long run that the signal displays DON'T WALK. For rule A, this is merely \( \frac{T_A}{T_A + D} \).

Unconditioning and simplifying results in the following expression for \( \bar{w}_A \):

\[ \bar{w}_A = 0 \cdot \frac{D}{D + T_A} + \frac{T_A}{2} \cdot \frac{T_A}{D + T_A} = \frac{T_A^2}{2(T_A + D)} \quad (1.4) \]

To analyze rule B the problem again becomes simpler by conditioning on whether or not the pedestrian had to queue. First consider the case where a randomly arriving pedestrian has to queue. This pedestrian is equally likely to have been the first to arrive, the second to arrive, ..., the \( N_B \)th to arrive since the last dump. Because there must be \( N_B \) total pedestrians to initiate the next dump, the probability that the pedestrian is the \( k \)th to queue is equal to \( \frac{1}{N_B} \). Given that she is the \( k \)th pedestrian to arrive, her conditional mean waiting time is an \((N_B-k)\)th-order Erlang with parameter \( \lambda \), implying

\[ E(w_B \mid k \text{th pedestrian to queue}) = \frac{N_B - k}{\lambda} \quad . \]
Now it is necessary to uncondition which is accomplished by multiplying by the probability of being the $k$th arriving pedestrian and summing over all values of $k$ yielding

$$E(w_b \mid \text{pedestrian queues}) = \frac{N_B - 1}{2\lambda}.$$ 

If a pedestrian arrives during a WALK or a flashing DON'T WALK signal, she does not queue and experiences no wait as expressed by

$$E(w_b \mid \text{pedestrian doesn't queue}) = 0.$$ 

All that is needed is the probability that a randomly arriving pedestrian must queue. This probability is equal to the fraction of time in the long run that the signal displays DON'T WALK which is simply the expected length of a DON'T WALK divided by the expected length of a complete cycle,

$$P(\text{pedestrian queues}) = \frac{N_B}{\frac{N_B}{\lambda} + D} = \frac{N_B}{N_B + \lambda D}.$$ 

Now it is possible to calculate $\bar{w}_B$,

$$\bar{w}_B = E(w_b \mid \text{ped doesn't queue}) \cdot P(\text{ped doesn't queue}) + E(w_b \mid \text{ped queues}) \cdot P(\text{ped queues}) = 0 \cdot \frac{\lambda D}{N_B + \lambda D} + \frac{N_B - 1}{2\lambda} \cdot \frac{N_B}{N_B + \lambda D} = \frac{N_B(N_B - 1)}{2\lambda(N_B + \lambda D)}.$$ (1.5)

The final quantity that remains to be calculated is $\bar{w}_C$. Again, it is easiest to condition on whether or not the randomly arriving pedestrian had to queue. Given that a randomly arriving pedestrian must queue, his waiting time further depends on whether he was the first pedestrian to queue since the last dump. The first pedestrian to queue must wait $T_c$ minutes. Any other pedestrian that must queue arrives at a random time in $[0, T_c]$ implying that his expected waiting time is uniform on $[0, T_c]$. Thus a randomly arriving pedestrian that is not the first to queue has an expected waiting time of $\frac{T_c}{2}$ minutes. The probability that a randomly arriving pedestrian is the first to arrive after the last dump is $\frac{1}{k+1}$ because each pedestrian that queues is equally likely to be the first, second, ..., $(k+1)$th given that $k+1$ pedestrians queue. Putting all the pieces together yields,
\[ E(w_c \mid k+1 \text{ peds queue}) = E(w_c \mid 1\text{st ped to queue}) \cdot P(1\text{st ped to queue}) + E(w_c \mid 1\text{st ped to queue}) \cdot P(1\text{st ped to queue}) \]

\[ E(w_c \mid k+1 \text{ peds queue}) = T_C \left( \frac{1}{k+1} + \frac{T_C}{2} \frac{k}{k+1} \right) \]

\[ = \frac{(2+k)T_C}{2(k+1)} \cdot \]

Now, it is necessary to uncondition on \( k \) by multiplying by the probability that a randomly chosen pedestrian crosses the street in a group of \( k+1 \) queued pedestrians and summing over all values of \( k \). Applying the appropriate random incidence result, this probability is

\[ P(k+1 \text{ peds queue}) = \frac{(k+1)^2 \lambda T_C e^{-\lambda T_C}}{k! (1+\lambda T_C)} \cdot \]

It follows that

\[ E(w_c \mid \text{ ped queues}) = \sum_{k=0}^{\infty} E(w_c \mid k+1 \text{ peds queue}) \cdot P(k+1 \text{ peds queue}) \]

\[ = \sum_{k=0}^{\infty} \frac{(2+k)T_C}{2(k+1)} \cdot \frac{(k+1)^2 \lambda T_C e^{-\lambda T_C}}{k! (1+\lambda T_C)} \]

\[ = \frac{T_C}{2} \left( 1 + \frac{1}{1 + \lambda T_C} \right) \cdot \]

Obviously, if the pedestrian does not queue he experiences zero waiting time. Finally, the last term that must be computed is \( P(\text{pedestrian queues}) \) under rule C. As before, this quantity is equal to the fraction of time over a long period of time that the signal displays \textit{DON'T WALK} which is equal to the mean length of a \textit{DON'T WALK} signal divided by the mean length of a cycle,

\[ P(\text{pedestrian queues}) = \frac{1 + T_C}{\lambda + \frac{T_C}{D} + \frac{T_C}{D} + \frac{1 + \lambda T_C}{1+\lambda(D+T_C)}} \cdot \]

Combining all the terms in an appropriate manner produces,

\[ \bar{w}_c = E(w_c \mid \text{ ped doesn't queue}) \cdot P(\text{ped doesn't queue}) + E(w_c \mid \text{ped queues}) \cdot P(\text{ped queues}) \]

\[ = 0 \cdot \frac{\lambda D}{1 + \lambda(D+T_C)} + \frac{T_C}{2} \left( 1 + \frac{1}{1 + \lambda T_C} \right) \left( \frac{1 + \lambda T_C}{1 + \lambda(D+T_C)} \right) \]

\[ = \frac{T_C}{2} \left( \frac{2 + \lambda T_C}{1 + \lambda(D+T_C)} \right) \cdot (1.6) \]
A.2 Derivation of the Expected Values for the Fuel Measure

By definition, \( \bar{\tau}_i = \alpha E(w_i^2)E(s_i) + \beta E(s_i) \) for \( i = A, B, C \). In order to calculate \( \bar{\tau}_i \), I need \( E(w_i^2) \) and \( E(s_i) \). These are derived below and are used to produce (1.9)-(1.11).

From section A.1, it is known that \( w_A \) is uniformly distributed over \([0, T_A] \). Hence, \( E(w_A^2) = \frac{T_A^2}{3} \). Under rule A, the length of a red signal is the deterministic quantity \( T_A \). This implies that \( s_A \) is a Poisson random variable with an average arrival rate \( \lambda \) and expected value \( E(s_A) = \lambda T_A \). Combining all the terms produces

\[
\bar{\tau}_A = \lambda T_A \left( \frac{\alpha T_A^2}{3} + \beta \right).
\]  

(1.9)

Under rule B, the number of queued vehicles is the constant \( N_B \), so \( E(s_B) = N_B \). Given that the vehicle is the \( k \)th to arrive since the previous dump \( (k = 1, 2, \ldots, N_B) \), the time it must wait is an \((N_B-k)\)th order Erlang implying

\[
E(w_B^2|kth \ vehicle \ to \ arrive) = \frac{N_B-k}{\lambda^2}(1+N_B-k) \cdot
\]

As argued in Section A.1, \( P(kth \ vehicle \ to \ arrive) = 1/N_B \). Unconditioning produces

\[
E(w_B^2) = \sum_{k=1}^{N_B} \frac{N_B-k}{\lambda^2}(1+N_B-k)(\frac{1}{N_B}) = \frac{N_B^2-1}{3\lambda^2}.
\]

It follows that

\[
\bar{\tau}_B = N_B \left[ \frac{\alpha(N_B^2-1)}{3\lambda^2} + \beta \right].
\]  

(1.10)

For rule C, the expected number of vehicles that must stop is equal to one plus the expected number of Poisson arrivals in \( T_C \) minutes. This simplifies to \( E(s_C) = 1+\lambda T_C \).

To calculate \( E(w_C^2) \) it is easiest to condition on the number of vehicles in the dump. Given a vehicle is the first to arrive since the last dump, it must wait \( T_C \) minutes, so

\[
E(w_C^2|first \ vehicle \ to \ arrive) = T_C^2.
\]

If the vehicle is not the first to arrive its wait is uniformly distributed over \([0, T_C] \) which means

\[
E(w_C^2|not \ first \ vehicle \ to \ arrive) = \frac{T_C^2}{3}.
\]

It follows that

\[
E(w_C^2|k+1 \ total \ vehicles) = T_C^2(\frac{1}{k+1}) + \frac{T_C^2}{3}(\frac{k}{k+1})
\]
because a randomly arriving vehicle is equally likely to be first, second, ..., (k+1)th. To uncondition on k, it is necessary to multiply by \( P(k+1 \text{ vehicles in dump}) \), which is given in Section A.1, and sum over all values of k:

\[
E(w_C^2) = \sum_{k=0}^{\infty} \left( \frac{T_C^2(3+k)}{3(k+1)} \right) \left( \frac{(k+1)(\lambda T_C)^k e^{-\lambda T_C}}{k!(1+\lambda T_C)} \right) = \frac{T_C^2(3+\lambda T_C)}{3(1+\lambda T_C)} .
\]

To complete the calculation, the above terms are substituted into the formula for \( \bar{f}_C \):

\[
\bar{f}_C = \frac{\alpha T_C^2(3+\lambda T_C)}{3} + \beta(1+\lambda T_C) .
\] (1.11)

### A.3 Derivation of the Expected Values for the Pollution Measure

An air pollution function in terms of \( s \) and \( w \) is defined to be \( p(s,w) = \eta w^2 s + \theta s^2 \). The values for the \( \bar{p}_i \), \( i = A, B, \) and \( C \), are derived in this section.

Under rule A, the length of the red signal is the constant \( T_A \). Hence, \( s_A \) is a Poisson random variable with average arrival rate \( \lambda \). This implies that \( E(s_A^2) = \lambda T_A + (\lambda T_A)^2 \). Using this result plus the appropriate terms above, the expected amount of pollutant emissions per cycle for rule A is

\[
\bar{p}_A = \lambda T_A \left[ T_A \left( \frac{\eta T_A}{3} + \lambda \theta \right) + \theta \right] .
\] (1.14)

Rule B specifies the number of stops is equal to the constant \( N_B \). Thus \( E(s_B^2) = N_B^2 \). Substituting this term and all the appropriate terms that were previously derived into \( \bar{p}_B \) yields

\[
\bar{p}_B = N_B \left[ N_B \left( \frac{\eta N_B}{3 \lambda^2} + \theta - \frac{\eta}{3 \lambda^2} \right) \right] .
\] (1.15)

As argued for the fuel consumption measure, the number of stops per red signal as a result of rule C is a Poisson random variable that is shifted by one stop to the right. This produces the quantity \( E(s_C^2) = \lambda^2 T_C^2 + 3\lambda T_C + 1 \). It follows that

\[
\bar{p}_C = \frac{\eta T_C^2(3+\lambda T_C)}{3} + \theta \lambda^2 T_C^2 + 3\theta \lambda T_C + \theta .
\] (1.16)
Appendix B

Traffic Survey Data

This appendix contains information regarding a survey I mailed to traffic engineers nationwide. The first section contains a copy of the questionnaire which was mailed. Section B.2 contains the compilation of the results for the surveys which were completed and returned.

SECTION B.1 The Questionnaire Form

What follows is a copy of the questionnaire which was mailed to an arbitrarily selected set of urban traffic engineers. The survey was mailed December 15th, 1990 and the engineers were requested to complete it and return it by February 10th, 1991. Accompanying each survey were two additional items: (1) a cover letter which explained the purpose of the survey and encouraged the individual to call me if (s)he had any questions pertaining to the survey questions or the purpose of my research, and (2) a self-addressed return envelop for the completed survey.
MIT NATIONAL TRAFFIC SIGNAL SURVEY

Please complete this survey by February 10th, 1991.

Return completed survey in addressed envelop provided or mail to:
Karla V. Ballman
MIT
Room E40-179
77 Massachusetts Avenue
Cambridge, MA 02139
GENERAL INFORMATION

- Population of community: __________________________
- Total number of signalized intersections under your office's jurisdiction: __________

ISOLATED INTERSECTIONS

Isolated intersections are defined to be signalized intersections which are timed and operated independently of surrounding signalized intersections, i.e. there is no timing coordination with any other signals.

--Fixed-time control: This type of traffic control signal assigns the right of way at an intersection according to a predetermined schedule. The time interval for each signal indication in the time cycle is of fixed length and is predetermined on the basis of historic traffic patterns.

  number of fixed-time signals: __________

--Semi-actuated control: Signals under this type of control are usually located at intersections on a major street. The major street does not have any vehicle detectors and is always awarded a green signal in the absence of side-street demand. When detectors located on a side-street detect demand, it is serviced in accordance with some control logic. The main characteristic of this control method is that one street of the intersection has vehicles detectors and the other does not.

  number of semi-actuated signals: __________

--Fully-actuated control: Signals under this type of control employ vehicle-detectors on all approaches to the intersection. Each phase has a preset initial green interval. The green interval is extended by a preset unit extension for each actuation after the expiration of the initial interval, provided a gap greater than the unit extension does not occur. Total green extension time is limited by a preset maximum amount. Note that this differs from traffic-responsive signals because the decision to switch the right of way is based solely on whether there has been a gap in the traffic stream detected on the favored approach.

  number of fully-actuated signals: __________

--Traffic-responsive control: Signals under this type of control employ upstream vehicle-detectors to determine the traffic situation at the signal for the immediate future. Based upon this information and some optimization criterion, for each small time increment (usually 1-2 seconds) a decision is made whether or not to extend the existing green or to allocate it to some other approach. Note that this differs from fully-actuated control because of the optimization criterion; fully-actuated control switches the right-of-way when a gap in traffic has been detected.

  number of traffic-responsive signals: __________
SIGNAL NETWORKS

These are signalized intersections which are timed and operated in a coordinated fashion. The timing of a given signal depends on the timing of the surrounding signalized intersections.

--Separate network: A separate network is a system composed of signals whose timing schemes only depend upon other signals in the network and is designed to control traffic flow throughout an area. The operation and timing of this network as an entity is independent of all other existing signals.

    total number of separate networks: __________

---

Separate networks can be further identified as either grid networks or arterial networks. Of the total number of separate networks, please specify below how many there are of the two specific types.

--Arterial network: This is a system of signals along a single artery or a system composed solely of arterial streets. The objective of the timing scheme is to provide the maximum bandwidth along the arteries.

    number of arterial networks: __________

    total number of signals in all arterial networks: __________

--Grid network: This is a network of any geometrical form. It may include arterial streets but cannot be entirely composed of arterial streets.

    number of grid networks: __________

    total number of signals in all grid networks: __________
SIGNAL TIMING METHODS

ISOLATED INTERSECTIONS

FIXED-TIME CONTROL

• fraction of fixed-time signals which use more than one timing plan per day: 

• fraction of fixed-time signals with timing plan(s) developed by a manual method:

• briefly describe method(s) used
  (e.g. Webster's formulas, engineering experience, etc.):

  ________________________________________________________________
  ________________________________________________________________
  ________________________________________________________________

• fraction of fixed-time signals with timing plan(s) developed by computer packages:

• briefly describe package(s) used:
  (e.g. name of commercial package, self-designed program, etc.):

  ________________________________________________________________
  ________________________________________________________________
  ________________________________________________________________

SEMI-ACTUATED CONTROL

• fraction of semi-actuated signals with timing plan(s) developed by a manual methods:

• briefly describe method(s) used:

  ________________________________________________________________
  ________________________________________________________________
  ________________________________________________________________

• fraction of semi-actuated signals with timing plan(s) developed by computer packages:

• briefly describe package(s) used:

  ________________________________________________________________
  ________________________________________________________________
FULLY-ACTUATED CONTROL

- fraction of fully-actuated signals with timing plan(s) developed by manual methods:

- briefly describe method(s) used:

- fraction of fully-actuated signals with timing plan(s) developed by computer packages:

- briefly describe package(s) used:

TRAFFIC RESPONSIVE CONTROL

- describe in detail the nature of the traffic-responsive control:

-
NETWORK GRID / ARTERIAL

NETWORK GRID

• fraction of timing plans developed using
  ----manual methods: ________
  briefly describe method(s) used:

  ___________________________________________________________

  ___________________________________________________________

  ___________________________________________________________

  ___________________________________________________________

  ----TRANSYT-7F: ________
  ----SIGOP-II: ________
  ----other computer package(s): ________
  briefly describe package(s) used:

  ___________________________________________________________

  ___________________________________________________________

  ___________________________________________________________

ARTERIAL NETWORK

• fraction of timing plans developed using manual methods: ________

• briefly describe method(s) used

  ___________________________________________________________

  ___________________________________________________________

  ___________________________________________________________

• fraction of timing plans developed using computer packages: ________

• briefly describe package(s) used:

  ___________________________________________________________

  ___________________________________________________________

  ___________________________________________________________
MISCELLANEOUS INFORMATION

What percentage of your isolated fixed-time signals were last retimed
  • less than a year ago: 
  • between 1 year and 2 years ago: 
  • between 2 years and 3 years ago: 
  • more than 3 years ago: 

What percentage of your fixed-time signal networks were last retimed
  • less than a year ago: 
  • between 1 year and 2 years ago: 
  • between 2 years and 3 years ago: 
  • more than 3 years ago: 

What percentage of your signals do you regard as optimally timed? 

Which performance measures do you explicitly use when developing a signal timing plan?
  Minimizing delay (either per vehicle or total) 
  Minimizing fuel consumption 
  Minimizing air pollutant emissions 
  Maximizing intersection capacity or throughput 

How do you verify and evaluate your traffic signal timing plans?

________________________________________________________________________
________________________________________________________________________
________________________________________________________________________

General comments and/or concerns you have about signal timing methodology and procedures.

________________________________________________________________________
________________________________________________________________________
________________________________________________________________________
## SECTION B.2  The Questionnaire Results\(^\text{12}\)

What follows are tables summarizing the results of the raw survey data. The data is organized along the major sections of the survey: a summary of the various types of control methods used by each community, the types of signal timing methodologies used, and miscellaneous information regarding signalized intersections. The data in each section is split into four population groups (see Chapter Three).

### B.2.1 Enumeration of the Various Signal Control Strategies Used by Respondent Communities

Table B.1 corresponds to the information obtained from page one of the returned surveys for communities with a population less than 50,000.

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| Total No. | 1061 | 510 | 134 | 144 | 232 |
| % of Total | 100.00% | 48.07% | 12.63% | 13.57% | 21.87% | 0.00% |

\(^{12}\) Note that there are some inconsistencies in the data due to round-off error and discrepancies in the raw data.
Table B.2 corresponds to the information obtained from page one of the returned surveys for communities with a population between 50,000 and 100,000.

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Table B.2  Enumeration of isolated signal control strategies used in communities with a population between 50,000 and 99,999, inclusive.
Table B.3 corresponds to the information obtained from page one of the returned surveys for communities with a population between 100,000 and 250,000.

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Table B.3 Enumeration of isolated signal control strategies used in communities with a population between 100,000 and 249,999, inclusive.
Table B.4 corresponds to the information obtained from page one of the returned surveys for communities with a population greater than 250,000.

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Table B.4 Enumeration of isolated signal control strategies used in communities with a population greater than or equal to 250,000.
Tables B.5, B.6, B.7, and B.8 summarize the information regarding coordinated signal network systems obtained from page two of the returned surveys.

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| Total | 87   | 551  | 81   | 451  | 6    | 100  |

| % of Total | 93.10% | 81.85% | 6.90% | 18.15% |
| Avg No. of Signals | 5.57 | 16.67 |

Table B.5 Enumeration of signals in coordinated control systems used in communities with a population between 0 and 49,999, inclusive.


### Table B.6

Enumeration of signals in coordinated control systems used in communities with a population between 50,000 and 99,999, inclusive.

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Table B.6: Details of traffic survey data.
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**Total** | 499 | 5537 | 405 | 2683 | 94 | 2824

**% of Total** | 81.16% | 48.72% | 18.84% | 51.28%

**Avg No. of Signals** | 6.62 | 30.04

Table B.7 Enumeration of signals in coordinated control systems used in communities with a population between 100,000 and 249,999, inclusive.
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<td>Avg No. of Signals</td>
<td></td>
<td>9.23</td>
</tr>
</tbody>
</table>

Table B.8 Enumeration of signals in coordinated control systems used in communities with a population greater than or equal to 250,000.
B.2.2 Categorization of the Timing Methodologies Used

Tables B.9, B.10, B.11, and B.12 aggregate the responses to the questions found on the third and fourth pages of the survey. These questions pertain to the timing methodologies for isolated intersections.

<table>
<thead>
<tr>
<th>Isolated Timing Plan Development Methodology</th>
<th>Fixed-Cycle</th>
<th>Semi-Actuated</th>
<th>Fully-Actuated</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF RESPONDENTS</td>
<td>26</td>
<td>29</td>
<td>23</td>
</tr>
<tr>
<td>SOAP</td>
<td>2</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>PASSER II</td>
<td>1</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>SICAP</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>NCAP</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>manual techniques</td>
<td>21</td>
<td>21</td>
<td>16</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>1</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>responsibility of other agency</td>
<td>2</td>
<td>2</td>
<td>4</td>
</tr>
</tbody>
</table>

Table B.9 Inventory of the isolated signal timing methodologies used by communities with a population less than 50,000.

<table>
<thead>
<tr>
<th>Isolated Timing Plan Development Methodology</th>
<th>Fixed-Cycle</th>
<th>Semi-Actuated</th>
<th>Fully-Actuated</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF RESPONDENTS</td>
<td>33</td>
<td>37</td>
<td>38</td>
</tr>
<tr>
<td>SOAP</td>
<td>1</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>PASSER II</td>
<td>2</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>FORCAST</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>CINCH</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>SIGNAL85</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>HCM software</td>
<td>2</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>manual techniques</td>
<td>25</td>
<td>28</td>
<td>30</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>1</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>responsibility of other agency</td>
<td>3</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

Table B.10 Inventory of the isolated signal timing methodologies used by communities with a population between 50,000 and 99,999, inclusive.
Table B.11  Inventory of the isolated signal timing methodologies used by communities with a population between 100,000 and 249,999, inclusive.

<table>
<thead>
<tr>
<th>Isolated Timing Plan Development Methodology</th>
<th>Fixed-Cycle</th>
<th>Semi-Actuated</th>
<th>Fully-Actuated</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF RESPONDENTS</td>
<td>32</td>
<td>39</td>
<td>42</td>
</tr>
<tr>
<td>SOAP</td>
<td>5</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>PASSER II</td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>CINCH</td>
<td>1</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>SICAP</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>HCM software</td>
<td>3</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>CAPPSI</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>manual techniques</td>
<td>28</td>
<td>32</td>
<td>34</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>7</td>
<td>5</td>
<td>7</td>
</tr>
</tbody>
</table>

Table B.12  Inventory of the isolated signal timing methodologies used by communities with a population greater than or equal to 250,000.

Table B.12  Inventory of the isolated signal timing methodologies used by communities with a population greater than or equal to 250,000.

<table>
<thead>
<tr>
<th>Isolated Timing Plan Development Methodology</th>
<th>Fixed-Cycle</th>
<th>Semi-Actuated</th>
<th>Fully-Actuated</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF RESPONDENTS</td>
<td>24</td>
<td>29</td>
<td>33</td>
</tr>
<tr>
<td>SOAP</td>
<td>5</td>
<td>8</td>
<td>13</td>
</tr>
<tr>
<td>PASSER II</td>
<td>1</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>CINCH</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SIGNAL85</td>
<td>0</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>HCM software</td>
<td>5</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>CAPPSI</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>manual techniques</td>
<td>23</td>
<td>23</td>
<td>30</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>7</td>
<td>10</td>
<td>15</td>
</tr>
</tbody>
</table>

The last tables of this section, Tables B.13, B.14, B.15, and B.16, aggregate the responses to the questions found page five of the survey. These questions are in regard to signal timing methodologies for coordinated signal systems.
### APPENDIX B. Traffic Survey Data

<table>
<thead>
<tr>
<th>Coordinated Timing Plan Development Methodology</th>
<th>Grid Network</th>
<th>Arterial Network</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF RESPONDENTS</td>
<td>6</td>
<td>31</td>
</tr>
<tr>
<td>TRANSYT-7F</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>PASSER II</td>
<td>1</td>
<td>11</td>
</tr>
<tr>
<td>MAXBAND</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>SIGOP</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>PROGO</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>SICAP</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>unknown computer software</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>manual (space-time diagrams)</td>
<td>4</td>
<td>17</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>

Table B.13 Inventory of the coordinated signal timing methodologies used by communities with a population less than 50,000.

<table>
<thead>
<tr>
<th>Coordinated Timing Plan Development Methodology</th>
<th>Grid Network</th>
<th>Arterial Network</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF RESPONDENTS</td>
<td>23</td>
<td>39</td>
</tr>
<tr>
<td>TRANSYT-7F</td>
<td>13</td>
<td>11</td>
</tr>
<tr>
<td>PASSER II</td>
<td>6</td>
<td>22</td>
</tr>
<tr>
<td>MAXBAND</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>SIGOP</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>PROGO</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>FORCAST</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>NOSTOP</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>COPTRAFLO</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>AAP</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>manual (space-time diagrams)</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>1</td>
<td>7</td>
</tr>
<tr>
<td>responsibility of other agency</td>
<td>1</td>
<td>2</td>
</tr>
</tbody>
</table>

Table B.14 Inventory of the coordinated signal timing methodologies used by communities with a population between 50,000 and 99,999, inclusive.
### APPENDIX B. Traffic Survey Data

#### Table B.15

<table>
<thead>
<tr>
<th>Coordinated Timing Plan Development Methodology</th>
<th>Grid Network</th>
<th>Arterial Network</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TOTAL NUMBER OF RESPONDENTS</strong></td>
<td>40</td>
<td>47</td>
</tr>
<tr>
<td>TRANSYT-7F</td>
<td>27</td>
<td>16</td>
</tr>
<tr>
<td>PASSER II</td>
<td>9</td>
<td>33</td>
</tr>
<tr>
<td>SIGOP</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>PROGO</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>FORCAST</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>NOSTOP</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>COPTRAFL O</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>manual (space-time diagrams)</td>
<td>18</td>
<td>23</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>11</td>
<td>15</td>
</tr>
</tbody>
</table>

Inventory of the coordinated signal timing methodologies used by communities with a population between 100,000 and 249,999, inclusive.

#### Table B.16

<table>
<thead>
<tr>
<th>Coordinated Timing Plan Development Methodology</th>
<th>Grid Network</th>
<th>Arterial Network</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TOTAL NUMBER OF RESPONDENTS</strong></td>
<td>40</td>
<td>39</td>
</tr>
<tr>
<td>TRANSYT-7F</td>
<td>26</td>
<td>12</td>
</tr>
<tr>
<td>PASSER II</td>
<td>6</td>
<td>28</td>
</tr>
<tr>
<td>MAXBAND</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>SIGOP</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>PROGO</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>FORCAST</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>NOSTOP</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>COPTRAFL O</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>SIGRID</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>SIGART</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>SIGPROG</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>HCM software</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>manual (space-time diagrams)</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>some manually &amp; some computer</td>
<td>16</td>
<td>22</td>
</tr>
</tbody>
</table>

Inventory of the coordinated signal timing methodologies used by communities with a population greater than or equal to 250,000.
B.2.3 Miscellaneous Information

Table B.17 aggregates the results of the first question found in the miscellaneous information section which is on the last page of the survey. This question pertains to signals operated under isolated fixed-cycle control. Note that Table B.17 contains the information for all the community group sizes.

<table>
<thead>
<tr>
<th></th>
<th>Community Size 0-49,000</th>
<th>Community Size 50,000-99,999</th>
<th>Community Size 100,000-249,999</th>
<th>Community Size 250,000+</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 1 yr ago</td>
<td>17.8%</td>
<td>5.3%</td>
<td>17.4%</td>
<td>15.8%</td>
</tr>
<tr>
<td>1-2 years ago</td>
<td>8.0%</td>
<td>15.1%</td>
<td>28.4%</td>
<td>17.8%</td>
</tr>
<tr>
<td>2-3 years ago</td>
<td>24.0%</td>
<td>20.3%</td>
<td>7.7%</td>
<td>16.7%</td>
</tr>
<tr>
<td>more than 3 yrs ago</td>
<td>50.2%</td>
<td>59.0%</td>
<td>46.4%</td>
<td>49.6%</td>
</tr>
<tr>
<td>No. of respondents</td>
<td>25</td>
<td>32</td>
<td>30</td>
<td>28</td>
</tr>
</tbody>
</table>

Table B.17 Percentage of a community's isolated fixed-cycle signals which were last retimed in the indicated time in the past.

Table B.18 contains values analogous to those in previous table with respect to fixed-cycle coordinated signal systems.

<table>
<thead>
<tr>
<th></th>
<th>Community Size 0-49,000</th>
<th>Community Size 50,000-99,999</th>
<th>Community Size 100,000-249,999</th>
<th>Community Size 250,000+</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 1 yr ago</td>
<td>27.1%</td>
<td>22.0%</td>
<td>23.6%</td>
<td>21.9%</td>
</tr>
<tr>
<td>1-2 years ago</td>
<td>19.2%</td>
<td>34.1%</td>
<td>29.8%</td>
<td>19.1%</td>
</tr>
<tr>
<td>2-3 years ago</td>
<td>10.4%</td>
<td>13.2%</td>
<td>10.1%</td>
<td>14.9%</td>
</tr>
<tr>
<td>more than 3 yrs ago</td>
<td>43.4%</td>
<td>28.7%</td>
<td>36.7%</td>
<td>44.2%</td>
</tr>
<tr>
<td>No. of respondents</td>
<td>26</td>
<td>25</td>
<td>36</td>
<td>35</td>
</tr>
</tbody>
</table>

Table B.18 Percentage of a community's fixed-cycle control coordinated signal systems which were last retimed the indicated time in the past.

The results for the questions, in the miscellaneous information section of the survey, pertaining to the percentage of optimally timed signals and to the performances measures which are explicitly used when developing a signal timing plan are found in Table B.19. Again, this table contains results for all four population groups.
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<table>
<thead>
<tr>
<th>Delay Measure</th>
<th>Community Size 0-49,000</th>
<th>Community Size 50,000-99,999</th>
<th>Community Size 100,000-249,999</th>
<th>Community Size 250,000+</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fuel Measure</td>
<td>6</td>
<td>8</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>Air Pollution Measure</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Intersection Capacity</td>
<td>24</td>
<td>31</td>
<td>32</td>
<td>33</td>
</tr>
<tr>
<td>Total No. of Respondents: performance measure question</td>
<td>35</td>
<td>44</td>
<td>45</td>
<td>38</td>
</tr>
<tr>
<td>Percentage of Signals which are optimally timed</td>
<td>61.7%</td>
<td>54.9%</td>
<td>63.0%</td>
<td>62.7%</td>
</tr>
<tr>
<td>No. of Respondents</td>
<td>34</td>
<td>42</td>
<td>42</td>
<td>36</td>
</tr>
</tbody>
</table>

Table B.19. Inventory of the performance measures which are explicitly used by communities and the percentage of signals which are regarded as optimally timed.

The final two questions of the survey required responses. There was not much variation in the responses to the question regarding the evaluation of the signal timing plans. By far the two most common responses were field observations and public complaints (or lack thereof). Included in the field observations are delay studies, travel time studies, and traffic counts. Other responses included the following: (1) staff driving the systems after a new timing plan implementation to evaluate its performance and to make fine-tuning adjustments, (2) the use of computer packages (including TRANSYT-7F, PASSER II and TRAF-NETSIM) to check each other’s output and to compare to the field performance, and (3) the use of a central computer to monitor the system performance. One community indicated that they could use their computer system to easily implement different timing plans on a short-term basis and perform field evaluations for each candidate plan. Based upon the results, they choose the plan which produced the best values of the measures during the field testing.

I close this appendix with each of the written comments I received in response to the last question of the survey soliciting general comment and/or concerns about signal timing methodology. These are grouped by the four community sizes.

Community Size: 0-49,999

(B.1) I am looking for good books on how to maintain signals and troubleshoot solid state controllers other than IMCA. This field is limited except for hands on experience.
(B.2) Our major problem is that we are close to a major sports complex. At that time we have to have our police department manually operate the signals to get the traffic through.

(B.3) Happy with what we have. Few complaints even though our two main arteries carry in excess of 30,000 vehicles per day.

(B.4) There needs to be an universal standard relating to minimum greens, clearance intervals and uses of certain phasing, i.e. lead lag vs. concurrent.

(B.5) With the type of controllers available today all communities should look at updating their timing plans with the use of TRANSYT-7F or other means offered through consulting firms.

(B.6) Observed wasted times for protected turn movements. I prefer protected/permisive left turn signal operation whenever there is not a visibility problem, a double turn lane or geometry that might make it unsafe. I like to set vehicle detector approximately 45' to 30' behind stop bar to skip left turn phase when only two vehicles are waiting to make a left turn.

(B.7) We do not have the resources (engineering) to keep up with the timing and traffic data acquisition like I would like to see.

(B.8) Obviously our city's traffic signals are few and the village is in the process of upgrading to newer controllers in the next few years or so. These will replace the current equipment which is 1940's vintage (making replacement parts almost unobtainable).

(B.9) Any method or combination of methods used can only give a "ball park" answer, field observations and fine-tuning of timing used will increase efficiency.

(B.10) There is all kinds of programs out there and several methods for doing capacity analysis. I wish there were more uniformity.

(B.11) I have yet to find a computer program that will give more accurate time with simple input and instructions.

(B.12) Current FHWA software could be made more user friendly and easier for data entry.

(B.13) All traffic signals should be evaluated at less than a year during AM, PM and noon peak periods.

(B.14) I am not pleased with all my methods. A private consultant firm was recently hired to evaluate all signals and timing. All except for fixed-time signals met standards. By March 1991, our city will have one arterial computer network and one computer grid network. Another grid network is planned by 1993. I believe fuel consumption, air pollutant emissions can be reduced by a system network.

(B.15) The most difficult aspect about optimally timing traffic signals is the amount of data collection required to develop timing plans. Most agencies do not have the manpower available to do this and are not willing to commit the funds to hire additional help or contract out. As a result, much timing is done by the "seat-of-the-pants".

(B.16) Lack of staff prevents us from being active in signal retiming.

(B.17) Tried PASSER II but was dissatisfied with results. Cycle lengths during peak periods ended up being very short.

(B.18) Many times we adjust timing based upon complaints received from driving motorists. We have very little back-up, even during high rush hour peak.
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(B.19) I tried PASSER II but ignored results when output resulted in cycles shorter than reasonable—the too short of cycle lengths resulted in congestion.

(B.20) Many of our fixed-cycle plans have not been adjusted in years.

Community Size: 50,000-99,999

(B.21) Are there any persons or person regulating or testing the flood of “ideal” programs for traffic systems?

(B.22) Signal timing procedures are tedious but necessary and difficult to justify with local funds.

(B.23) Signal85 (based on 1985 HCM) is a good method for isolated timing however a better procedure needs to be developed to handle permissive left turns as well as right-turns-on red.

(B.24) There are signals within the city that although retimed have basically remained the same for 10+ years. A large percentage of the signals within the city were installed because of past and current "P" (political) factors.

(B.25) Work needs to be done with TRANSYT-7F so that it will produce good offsets. The offsets that it calculates are horrible.

(B.26) There are many varied techniques used for signal timing, but actual field verifications and modifications are necessary to fine-tune signal timings. Signal timings can not be done totally in the office and expected to work under all conditions.

(B.27) Planning to use PASSER II this year. We have not used a lot of computer packages because this is a small department and it is difficult for me to get all the traffic counts necessary on an arterial.

(B.28) Time base coordination is best with pre-timed loop activated non streets and background cycle for populated areas. Traffic responsive is glorified pre-timed and cannot be engineered significantly better than using different pretimed cycle lengths and plans based on historical traffic data. It’s a nice new try. Is there any city in America that has 16 timing plans for an average weekday. If so, please let me know.

(B.29) Would like to see a program for network timing that combines bandwidth maximization and delay minimization techniques.

(B.30) Many machine packages require large data input; trained manpower to collect and input is had to come by in the press of large public expectations and small staff.

(B.31) Due to insufficient manpower, worktime has to be rationed among many fields of responsibility. Traffic signal timing always seems to get pushed down on the priority list.

(B.32) In the past few years very little research was devoted to isolated signal timing methodology. Hopefully, the 1990 Clean Air Act will trigger some awareness and focus on isolated intersections which are mostly located in smaller cities and towns. Improved signal timing at intersections yields in minimized delay, fuel consumption, air pollution and maximize capacity. Consequently, improve quality of life.

(B.33) Areas which need to be explored: (1) Conditional service i.e. serving a movement twice in one signal cycle. (2) Detector counts gathered by controllers providing turning movement counts for signal timing optimization. (3) A graphics package to generate good time-space plots from
PASSER-TRANSYT data that can be "tweaked" and stored as a graphics file to document the timing plan used in the field. (4) Increased public awareness of the 30:1 benefit/cost improvement gained by signal timing optimization.

(B.34) In our city, we do not have signal networks. Demographic changes, seasonal changes, changes in trip destination and trends which affect peak hour volume and time are not documented due to manpower constraints. Therefore the single most important determinant in the timing of our signal plans is engineering judgement.

(B.35) It's not as easy as just plugging in timing: detector placement; amplifier delay, extension, minimum green and safety all have impacts on signal timing. Any generalization in this area is a dangerous proposition.

(B.36) Computer programs are useful tools for implementing timing plans. But speaking from experience, the user needs to be able to evaluate computer results and sort out what is usable and what is garbage.

(B.37) No one computer methodology (manual or computer) provides the output necessary to program a computer controlled system (or any other system). Picking the right combination of various methods working together produce reasonable starting points to begin or enhance programming. Often people rely on formulas and software to produce a "final" product. Common sense is stressed at all times which may conflict with the results of equations and computer output. The goal to reduce delay which in turn decreases fuel used and pollution, is strived for. The perceptions and attitudes of travelling motorists which cannot be calculated by equations or computer must be accounted for as well.

(B.38) HCM procedure does not give optimal timing directly. Phase times solved by trial and error.

(B.39) Technicians in X seem to rely heavily on computer results without thinking about the outcome, input or possible alternatives. They seem to have the technology without the training. We have the training without the technology. I have so many other tasks and responsibilities that unless there is a glaring problem--I do not devote the time I should to signal timing and phasing. Phasing can be just as important as timing.

(B.40) When timing semi-actuated signals in a system, it is difficult to use existing optimization programs. I also find time/space outputs not refined enough to prove useful when fine-tuning a system.

(B.41) Computer models are a good start but field modifications are often necessary.

(B.42) Please note that while we use various software packages for signal and system timing, they only serve as a guide and manual modifications are made using time-space, driving and field observations. No computer program can duplicate actual field occurrences, to use them exclusively would not be effective.

(B.43) Majority of signal timing programs available need to be more user friendly.

(B.44) The computer programs are good for initial analysis. However, there can be too much reliance on the program results. Common sense and good engineering judgement must be part of the process.

(B.45) I worked with a traffic responsive system for ten years and now manage a system that runs on time-of-day patterns. My own observation is that a lot of agencies WANT traffic responsive systems thinking that it will provide better system operation than time-based systems. However, they eventually learn that any control system is only as good as the timing plans implemented. I would rather manage a four dial time-based system with 3 out of 4 good timing plans than sink a lot of money in occupancy detectors that most agencies never maintain.
APPENDIX B. Traffic Survey Data

(B.46) Most of our actuated controllers have capabilities for gap reductions but only a few are in use. The few that are in use are set to allow only small reductions.

(B.47) Regardless of the method used to create an effective signal timing scheme, the benefits will be greatly offset if proper signal maintenance is not stressed. Although your survey does not concern itself with equipment maintenance, we feel some mention should be made of this. Side-streets and left-turn lanes that are "false calling" render any timing plan almost worthless and increase fuel consumption and pollution. We hope you will find room to include this in your conclusions.

(B.48) The procedure used by myself and staff are adequate for low volume, very directional flows of traffic. The city recently authorized me to obtain an IBM-compatible computer so that I can obtain signal timing software like TRANSYT-7F or SIGOP II. In the near future we will be checking our timing plans with those recommended from one of these programs.

Community Size: 100,000-249,999

(B.49) Although most of our signals operate under some type of coordination plan, at least during the morning and evening peak hours, there is still a segment of the travelling public that perceives that there is no coordination at all.

(B.50) I simply do not have time to do what should be done.

(B.51) Signal timing for maximum efficiency is extremely labor intensive. The best optimization programs offer only a starting point, because they cannot possibly be fed all real-world data that engineers are capable of assimilating. My experience has been that "fine-tuning" is always necessary.

(B.52) Budget constraints and lack of assigned personnel to do studies.

(B.53) TRANSYT-7F is not very user friendly and very data intensive.

(B.54) Computer systems require a great deal of input data (TRANSYT-7F) etc. that make the results highly questionable. Most computer results are crude and modified by traffic engineer before they "go to the streets".

(B.55) Need more person power to stay even.

(B.56) Experience has shown that all algorithm generated timing plans require field adjustments. However, we rely quite heavily on certain simulation packages for impact assessment studies. In general very little time is allocated towards signal retiming. Even in a larger city with qualified staffing, consultants are hired to re-time traffic signals. We would love to review signal timing plans on an annual basis but statewide very little political impetus exists for such a program.

(B.57) None of the computer models for signal optimization is a panacea. Each can provide some insights to the engineer about a network but we've found that the manual fine-tuning of computer generated timing is required in all cases to get the best field coordination timing.

(B.58) TRANSYT-7F is too data intensive.

(B.59) We do not have staff or facilities to truly "optimize" signal timing. Also, our computer system is no longer "state of the art". We don't have a scientific basis for evaluating signal timing (i.e. delay, travel time, etc) other than theoretical and observed capacity.
There are numerous signal timing computer software packages that are available in the market today, but none of them can provide a perfect solution to a system, other than you being the implementor of the system spend as much time fine-tuning the patterns supplied by these packages out in the field.

Theory and practice of traffic signal optimization are not always similar. What may work on paper or on the computer may not work at all in the field. Computer methods still require extra manual calculations. All methods still require adjustments in the field.

Generally speaking, best timing procedure I have found is to run the data through all programs available. Then analyze results to try and optimize each program's output, as well as notice how each program "describes" the network operation. The combination of these descriptions is then used to determine what will and will not work in conjunction with a time-space diagram analysis. Additionally, one should be familiar with network operation before and after timing changes. A follow-up is also advised due to the fact that many times driving habits are a function of timing.

The methodology of signal timing simulation and optimization as incorporated in the TRANSYT-7F software package has to a large extent produced much improved traffic progression along major arterials with as much as 20% reduction in travel time and delay.

More time should be placed on looking over new timing plans before they are installed in the street. Most of the systems should be looked at in off-peak time. We get a lot of call-ins about off-peak timing causing side-street delays.

The department currently lacks the capabilities to use the several analytical techniques and computer programs used to determine the most appropriate phasing and timing schemes.

Signal timing for fixed-time signals in a grid network is very dependent on the speeds and progression available for different cycle lengths. This somewhat hampers the ability to use a wide variety of cycles. Signals timing for actuated signals is very dependent on loop placement and the type of responsive nature you desire in the signal. We prefer responsive signals and therefore use several loops on each approach. Maximum times are set based on peak hour flows while trying to minimize delay.

No matter what method is used for setting up timing plans, whether is be by hand or computer model, the main thing to remember is that no matter how accurate your data is and calculations are, field adjustments will have to be made. This may mean driving the arterial several times and making adjustments to implement the best timing plan for that roadway.

Would like to see more workable ways of measuring delay in the field at signalized intersections.

There seems to be no standardization of terminology in reference manuals in regard to vehicle actuated timing parameters.

Most traffic engineering organization need more staff to work with modern day computer packages to evaluate timing programs before and after installation.

Can't rely on computer programs, one must actually observe.

Volume density features are used on streets that have 85th percentile speeds of 30 mph or greater, i.e. 30 mph detector 160' from crosswalk, minimum initial, added initial, time to reduce, time before reduction, initial gap, minimum gap and last car passages are typical timing features for volume density NEMA-CalTrans Type 90 solid state traffic controller.
APPENDIX B. Traffic Survey Data

(B.73) New signal installations in the city utilize type 3 detectors capability with 1-2 seconds input at detectors as extension time. The city has specified Type 170 Controllers early 1980's.

(B.74) The downtown area was setup by taking the six top priority roadways (all one-way except one) and using space-time diagrams and trial and error. Then rest of downtown was coordinated accordingly. Four years later we are still using same plans that were developed and system is still very effective even with traffic growth.

(B.75) We do not have enough staff or computer support to be able to operate computer packages. TRANSYT-7F has been used by consultants hired by the City of X for retiming projects.

Community Size: 250,000+

(B.76) Our primary concern is pedestrians. Conflicts between pedestrians and vehicles are the primary cause of our delays. Priority of pedestrian intervals (length of crossing time) dictates many of our timing plans that result in vehicular delay.

(B.77) Biggest concern is for the frequency of retiming projects--should be every two years or so. Also--signal timing projects should include all periods and weekends--not just Monday-Friday peak hours. Field fine-tuning is critical. Special signal phasing schemes should be considered more for bandwidth improvements.

(B.78) We need a simple, effective PC-based programs for timing isolated intersections, evaluating alternative phasings and for maximizing green bands along arterial and grid networks. Existing programs either require excessive data entry or don't handle all the currently used phasings (like split-phase or turning movements from through lanes).

(B.79) Obtaining true optimal signal operations is unaffordably man-power intensive. When technology advances to enable sufficient vehicle detectorization, then traffic-responsive operation will start to work and optimal signal operations will be attainable.

(B.80) 1. In the signal networks, we have 9 systems of 826 intersections that run UTCS enhanced system. The "Traffic Responsive" feature selects the best timing plan from a database using a pattern matching of actual flow information to a flow for each possible plan. The "Critical Intersection Control" feature cyclically optimizes the split at major intersections based on volume and occupancy data collected from detectors. We also have 1.5 Generation Control to run real-time online TRANSYT-7F. Volume is collected for the entire network simultaneously, and then downloaded if desired.

(B.81) You need to expand into the future. Going into year 2000, real-time system (IVHS) will require software and analysis of a higher generation.

(B.82) Fine-tuning in the field is very important.

(B.83) Most computer programs do not include options for: multiple services to minor movements within one cycle or two uneven cycles to one cycle intersections within a system.

(B.84) We are currently implementing a central UTCS type system. Phase 1 involves 230 signals with expansion of 70 more to follow soon. All of these signals will be timed using PASSER II and TRANSYT-7F. In addition, through a separate project, our CBD grid timing is being evaluated with NETSIM. Our staff is extremely limited, however we hope to eventually gain some evaluation hardware to perform moving car delay studies and stop delay studies.
(B.85) Staff reductions have made it difficult to use TRANSYT-7F projects and other major optimization programs. We have the knowledge for this but lack the staff and funding for major retiming projects.

(B.86) 1. Current software packages (e.g. TRANSYT-7F) do not address the motorist's desires with regard to green bandwidth on the arterial streets.  
2. Traffic signal timing programs are not continuous and on-going.  
3. State-of-the-art signal controllers may have too many frills and many excess capabilities which really don't address the motorists' desires for travel.

(B.87) The greatest problem in developing signal timing plans is finding the time to gather all the pertinent data needed by the computerized models. For networks, I feel the computerized models yield better results than manual methods.

(B.88) The computer methods require too much data gathering to be useful. Even though a timing plan may be optimum by some measure it may be unacceptable in practice.

(B.89) Timing plan generation is often restricted by constraints other than the performance measures. Ped Crossing requirements, phase, signal spacing generally have the greatest effect often placing a volume generated/oriented solution outside of the optimal solution space.

(B.90) Most computer programming packages are very data intensive. Most are also not very user-friendly. Not much interrelationship among packages.

(B.91) Existing computer system is a Perkins-Elmer Hardware. Software is a 1980 version of 1950 D.C. Junk. ... of manual timing is done on computer--both system computer and a software package (COPTRAFLO 1985).

(B.92) Most municipalities hesitate to use actuated control due to concerns of maintenance costs and the complexity of actuated signal timing plans. I urge that more actuated signals be installed to improve operational effectiveness. Actuated signals should be flexibly coordinated during peak hours. Computer simulation should be widely used for selecting optimal timing plans.

(B.93) We feel that TRANSYT-7F generates the most efficient timing plans compared to other software. Initially the data input for this software required a lot of effort and time but with the development of E-Z7 program, this problem has been resolved. The time-space diagram output, however, still needs improvement.

(B.94) All timing plans require field verification. Computer outputs are at most a decent starting point.

(B.95) Prefer programs that provide best bandwidth rather than those that minimize delay by building in a stop. The traffic engineer appears more competent to the motoring public if (s)he reduces the number of stops on an arterial even though (s)he may have greater side-street delay, air pollution, fuel use, etc.

(B.96) Computer software will give information on paper, but field fine-tuning is needed.

(B.97) "Computer Packages" are nothing more than "rough" approximations and should be treated as such. The only way to get "good" signal timing is through continual observation and adjustment, by experiences and trained personnel.

(B.98) The program "NOSTOP" considers the following factors when calculating timing plans: fuel economy (mpg), fuel usage (gallons), carbon monoxide (lbs), hydrocarbons (lbs), nitrous oxide (lbs), total main delay (hrs) total side delay (hrs) and operational costs (dollars).
(B.99) Although we find our computer programs to be a good comparison tool between existing and proposed timing plans; generally, they do not accurately reflect actual field conditions. Therefore, we tend to rely heavily on field observations by experienced technicians and engineers.

(B.100) We mesh several types of systems to maximize the efficiency. Our criteria is to "keep 'em rolling" (minimize stops) at or near posted speed limits.

(B.101) Prior to the latest release of PASSER II program, which in addition to the arterial analysis allows detailed analysis of isolated intersections, SOAP was our main tool for developing optimal timings and number of phases at each signal.

(B.102) Isolated intersections are not operated under traffic responsive control. The UTCS however, does have the traffic responsive provisions. Under this mode, the system compares volume and occupancy data pre-stored in the computer for each pre-determined traffic responsive pattern to the volumes and occupancies existing on the street. The system then selects the most suitable timing pattern.
Appendix C

Pareto Optimal Set Derivations and Relationships

Derivations of the quantities used in Chapter Five are contained in this appendix. The notation and equation numbers used in both places are identical.

C.1 MODEL1: Simplest Light Traffic Model

C.1.1 Delay Measure

For a given set of values of the parameters, various relationships can be explored. In particular, let \( \lambda_1 = 3 \) veh/min, \( \lambda_2 = 5 \) veh/min and lost time of \( L = 4 \) seconds per cycle for the entirety of this section.

Figure C.1 reveals how the optimal proportion of vertical green time for the delay measure, \( \tilde{d}_1 \), changes as \( C \) becomes larger. As cycle length increases, the \( \pi_{1,\tilde{d}_1}^* \) value shifts to the left as discussed. For this example, as \( C \to \infty \), \( \pi_{1,\tilde{d}_1}^* \to \frac{2}{8} = 0.375 \). Moreover, as cycle lengths become longer the difference in the delay values between \( \pi_{1,\tilde{d}_1}^* \) and other \( \pi_1 \) values increases. When the cycle length is greater than 0.5 minute, it can be seen that the delay is relatively insensitive to the allocation of green time whereas when \( C = 2 \) minutes, it is much more sensitive.
Another interesting plot is found in Figure C.2. This shows, for the same set of values for the arrival rates and lost time, how the optimal proportion \( \pi_1^*, \overline{d}_1 \) varies with cycle time. Again, it reveals that for the commonly used cycle lengths, those between \( C=0.5 \) to \( C=2 \) minutes, the optimal value for \( \pi_1 \) is insensitive to changes in cycle length. This suggests that one might be able to choose a \( \pi_1 \) value near its asymptotic value and then optimize the cycle length.

![Figure C.1 \( \overline{d}_1 \) as a function of \( \pi_1 \) values for different cycle lengths.](image)

For a given cycle length of \( C=1 \) minute, \( L=4 \) seconds per cycle, and total average arrival rate of \( \lambda=8 \) vehicles/minute, Figure C.3 illustrates the relationship of the expected delay per randomly arriving vehicle as a function of \( \pi_1 \) for various different flow ratios \( r=\frac{\lambda_1}{\lambda_2} \). It reveals, as expected, that \( \pi_1^*, \overline{d}_1 \) favors the approach with the higher flow rate. Furthermore, for the larger magnitudes of difference between \( r \) and \( 1/2, \overline{d}_1 \) is more sensitive to the \( \pi_1 \) value. This suggests that there is greater potential
savings in $\overline{d_1}$ associated with providing intersections with highly uneven flow rates with an optimal timing plan for a chosen $C$.

![Optimal $\pi_1$ Values](image)

Figure C.2  Optimal $\pi_1$ values versus different cycle lengths.

Upon examining the relationship between the optimal cycle length and the $\pi_1$ values for varying ratios of flows on each approach (Figure C.4), it can be observed that as flow ratios approach one, the difference in the optimal cycle length and $L$ becomes smaller. The further in magnitude the ratio is from one, the more sensitive the cycle time is to the $\pi_1$ values.

C.1.2 Fuel Measure and Pollution Measures

For the expected fuel consumption performance measure, I provide Figures C.5, C.6, and C.7 which match Figures C.1, C.2, and C.3 provided for the expected delay measure (including the same arrival rate and lost time values), respectively.

As the cycle lengths become longer, the $\pi^*_{1,\overline{d_1}}$ values shift to the left as did the $\pi^*_{1,\overline{d_1}}$ values. Specifically, as $C \rightarrow \infty$, it can be observed from equation (5.19) that
\( \pi_1^{\ast} \tilde{f}_1 \rightarrow \pi_1^{\ast}, \tilde{d}_1 \). In fact, as \( C \rightarrow \infty \), \( \pi_1^{\ast} \tilde{f}_1 \) approaches \( \frac{\lambda_1}{\lambda} \) as it does for \( \pi_1^{\ast}, \tilde{d}_1 \). It can also be observed from Figure C.5 that \( \tilde{f}_1 \) is more sensitive to the \( \pi_1 \) values as \( C \) increases. This is also true of the delay measure, \( \tilde{d}_1 \).

![Figure C.3](image)

Figure C.3 \( \pi_1 \) values versus \( \tilde{d}_1 \) for different approach 1 and approach 2 flow ratios.

Figure C.6 plots the optimal \( \pi_1 \) values versus the cycle length for \( \tilde{d}_1 \) and \( \tilde{f}_1 \). It can be seen more clearly that for a given cycle length, the lighter trafficked approach (the vertical one in this case), is awarded a smaller fraction of green time to minimize fuel consumption than when minimizing delay. Also the \( \pi_1^{\ast}, \tilde{f}_1 \) values are more sensitive to
changes in cycle length for the usual range of cycle lengths, $C=0.5$ to $C=2$ minutes, than the $\pi^*_1, \bar{d}_1$ values.

![Graph](image)

Figure C.4  Optimal cycle lengths, with respect to $\bar{d}_1$, as a function of $\pi_1$ values for different flow ratios on approach 1 and approach 2.

Regarding Figure C.7, the same general observations hold true for the expected fuel consumption rate as were made for the analogous graph with regard to the expected delay measure, Figure C.3.

Because the functional form of the expected emissions of pollutant $i$ ($i = \text{CO, HC}$ and $\text{NO}_x$) is the same, I haven't included analogous graphs for these functions.
Figure C.5  \( \tilde{\eta} \) versus \( \pi_1 \) values for different cycle lengths.
Figure C.6  Optimal $\pi_1$ values versus cycle length for $d_1$ and $\tilde{\pi}_1$. 
Figure C.7 $\pi_1$ values versus $\tilde{r}_1$ for different approach 1 and approach 2 flow ratios.
C.1.3 Additional Efficient Frontiers for MODEL1

For the following analysis, I choose to only depict Pareto optimal sets involving the CO pollutant measure (versus the other pollutant measures). The exhaust from a vehicle contains all three gases but both HC and NO\textsubscript{x} are in smaller quantities implying the main pollutant is carbon monoxide. Figures C.8 and C.9 contain the efficient frontiers for the delay and CO emissions measure pair, and fuel and CO emissions measure pair, respectively.

![Efficient frontier for the delay and CO emissions performance measures (MODEL1).](image)
Figure C.9  Efficient frontier for the fuel and CO emissions performance measures (MODEL1).
C.2 MODEL2: More Realistic Light Traffic Model

The quantities and additional graphs found in this section pertain to MODEL2 which is discussed in Section 5.4.

C.2.1 Delay Measure

To calculate \( \overline{d}_2 \), it is easiest to start with equation (5.44) of Section 5.4. This equation can be rewritten as

\[
\mathbb{E}[W_1] = \frac{\lambda_1[(C-(C-L)\pi_1)]}{2(1-\rho_1)} \left[ C-(C-L)\pi_1 + \frac{s}{1-\rho_1} \right] \tag{c.1}
\]

and

\[
\mathbb{E}[W_2] = \frac{\lambda_2[(C-(C-L)\pi_1+L)]}{2(1-\rho_2)} \left[ (C-L)\pi_1 + L + \frac{s}{1-\rho_2} \right] \tag{c.2}
\]

for approach 1 and 2, respectively. Recognizing that the expected total delay per cycle, \( \overline{W}_2 \), is merely the sum of (c.1) and (c.2), and that \( \overline{D}_2 \), the total expected delay per unit time for MODEL2, is merely \( \overline{W}_2 \) divided by \( C \), and finally that \( \overline{d}_2 \) is just \( \overline{D}_2 \) divided by \( \lambda \), it is possible to obtain an expression for \( \overline{d}_2 \):

\[
\overline{d}_2 = \frac{\lambda_1[(C-(C-L)\pi_1)]}{2\lambda C(1-\rho_1)} \left[ C-(C-L)\pi_1 + \frac{s}{1-\rho_1} \right] + \frac{\lambda_2[(C-(C-L)\pi_1+L)]}{2\lambda C(1-\rho_2)} \left[ (C-L)\pi_1 + L + \frac{s}{1-\rho_2} \right]. \tag{5.45}
\]

Claim C.1: Given the intersection is stable and given a cycle length \( C \), the expected delay per randomly arriving vehicle, \( \overline{d}_2 \), is a strictly convex function in \( \pi_1 \).

Proof: Obviously, \( \overline{d}_2 \) is quadratic in \( \pi_1 \). Holding the cycle time \( C \) fixed, the second derivative of \( \overline{d}_2 \), with respect to \( \pi_1 \) is

\[
\frac{\partial^2 \overline{d}_2}{\partial \pi_1^2} = \frac{(C-L)^2}{\lambda C} \left[ \frac{\lambda_1 + \lambda_2}{1-\rho_1} \right] > 0
\]

because \( C > 0 \), \( \lambda > 0 \) and the assumption that the intersection is stable implies that \( 1 > \rho_i \) for \( i = 1, 2 \).

Similarly, given \( \pi_1 \), it is possible to show that \( \overline{d}_2 \) is strictly convex in the cycle length, \( C \).
Claim C.2: Given the intersection is stable and given the proportion of useable green time awarded to the vertical approach, \( \pi_1 \), the expected delay per randomly arriving vehicle, \( \bar{d}_2 \), is a strictly convex function in \( C \).

**Proof:** Taking the second partial derivative of \( \bar{d}_2 \) with respect to \( C \) produces

\[
\frac{\partial^2 \bar{d}_2}{\partial C^2} = \frac{L^2}{\lambda C^3} \left[ \frac{\lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2}{1-\rho_1} \right] + \frac{L^2}{\lambda C^3} \left[ \rho_1 \pi_1 (2-\rho_1) + \rho_2 (1-\pi_1)(2-\rho_2) \right] > 0
\]

because all the constants are positive and the intersection is stable. \( \square \)

The relationships between the optimal \( \pi_1 \) values and the cycle length, \( C \) are shown in Figure C.10 for both MODEL1 (from Figure C.2) and MODEL2. The curve is basically just shifted to the left, suggesting that the approach with the lower average arrival rate gets even less green time under MODEL2 than MODEL1. Again, the optimal value of \( \pi_1 \) for MODEL2 is relatively insensitive to the cycle length for range of cycle lengths used in practice, \( C \in [0.5, 2.5] \).

![Figure C.10](image-url)  
*Figure C.10  Optimal \( \pi_1 \) values versus cycle length for MODEL1 and MODEL2.*
Figure C.11 is analogous to Figure C.1. It illustrates the effect that the different cycle lengths have on $\bar{d}_2$. As before, the longer the cycle lengths the more sensitive $\bar{d}_2$ is to $\pi_1$. From a comparison of Figures C.1 and C.11 it is also seen that for cycle lengths of the same length, $\bar{d}_2 > \bar{d}_1$ at all values of $\pi_1$. This same observation could have been drawn from (5.47). Such a result is not surprising because MODEL1 ignored the delay experienced by a queued vehicle incurred as a result of waiting for arrivals ahead of it to clear the intersection.

![Graph showing $\bar{d}_2$ as a function of $\pi_1$ values for different cycle lengths.](image)

Figure C.11  $\bar{d}_2$ as a function of $\pi_1$ values for different cycle lengths.

Finally, Figure C.12 shows the relationship, for a given cycle length, between the $\pi_1$ values and $\bar{d}_1$ and $\bar{d}_2$. The graphs in this figure are typical of graphs for other flow ratios, $r = \frac{\lambda_1}{\lambda_2}$. The MODEL2 curve has the same form except it is shifted upward to reflect the increase in expected delay per vehicle as a result of assuming vehicle service times are positive (rather than zero). For these graphs, the arrival rate on the
vertical approach is 3 veh/min and on the horizontal approach it is 5 veh/min and the lost time is 4 seconds per cycle.

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure_c12.png}
\caption{A comparison of $\bar{d}_1$ and $\bar{d}_2$ as a function of $\pi_1$ values for given approach 1 and approach 2 flow ratios.}
\end{figure}

Figure C.12 shows the optimal cycle lengths for the range of $\pi_1$ values and different flow ratios $r = \frac{\lambda_1}{\lambda_2}$. Notice that the cycle times are longer than for MODEL1 as they should be. Looking at Figure C.13, it appears as though the optimal cycle lengths are too small to be acceptable to traffic engineers. Hence, under light traffic conditions, an acceptable cycle length should be determined and then the optimal red-green splits should be found.
Figure C.13  Optimal cycle lengths, with respect to $\overline{d}_2$ as a function of $\pi_1$ values for different approach 1 and approach 2 flow ratios.

C.2.2 Fuel Measure and Pollution Measures

This subsection provides derivation for equations (5.48) and (5.49). It also proves claims for the performance measures produced under MODEL2 analogous for those of MODEL1.

The expression for $\overline{D}_2$ is $\lambda \overline{d}_2$,

$$\overline{D}_2 = \frac{\lambda_1[1(C-L)\pi_1]}{2C(1-\rho_1)} \left[ C-(C-L)\pi_1+s\left(\frac{2-\rho_1}{1-\rho_1}\right) \right] + \frac{\lambda_2[(C-L)\pi_1+L]}{2C(1-\rho_2)} \left[ (C-L)\pi_1+L+s\left(\frac{2-\rho_2}{1-\rho_2}\right) \right].$$

(c.3)

Now it is necessary to determine $\overline{s}_2$. As before, let $\overline{s}_1$ be the expected number of vehicles per unit time which queue during the red phases of a cycle. The expression for this quantity is found in equation (5.16). Also let the following notation denote the indicated quantities:

$$A_{1,1} \equiv \text{number of approach 1 arrivals in the interval } (R, R+sQ(R)),$$
APPENDIX C. Pareto Optimal Set Derivations and Relationships

\[ A_{1,2} \equiv \text{number of approach 1 arrivals in the interval } (R+sQ(R), R+s[Q(R)+A_1]), \]
and in general,
\[ A_{1,n} \equiv \text{number of approach 1 arrivals in } (R+s[Q(R)+A_1+\ldots+A_{n-1}), R+s[Q(R)+A_1+\ldots+A_{n-1}], \]
for \( n = 3, 4, 5, \ldots \).

and likewise for approach 2,
\[ A_{2,1} \equiv \text{number of approach 2 arrivals in the interval } (R, R+sQ(R)), \]
\[ A_{2,2} \equiv \text{number of approach 2 arrivals in the interval } (R+sQ(R), R+s[Q(R)+A_1]), \]
and in general,
\[ A_{2,n} \equiv \text{number of approach 2 arrivals in } (R+s[Q(R)+A_1+\ldots+A_{n-1}), R+s[Q(R)+A_1+\ldots+A_{n-1}], \]
for \( n = 3, 4, 5, \ldots \).

So as an approximation of the expected number of vehicles required to stop per unit time under MODEL2, \( \bar{S}_2 \), I use
\[ \bar{S}_2 = \bar{S}_1 + \frac{1}{C} \left[ E[A_{1,1}+A_{1,2}+\ldots] + E[A_{2,1}+A_{2,2}+\ldots] \right]. \]  
(c.4)

Note that under MODEL2 it is assumed that \( E[Q(0)]=0 \). Upon simplifying (c.4),
\[ \bar{S}_2 = \bar{S}_1 + \frac{1}{C} \left( \frac{\lambda_1}{1-\rho_1} + \frac{\lambda_2}{1-\rho_2} \right) \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] \]
\[ = \frac{\lambda_1}{C(1-\rho_1)} \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] + \frac{\lambda_2}{C(1-\rho_2)} \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right]. \]  
(c.5)

Now using the fact that \( \tilde{f}_2=\alpha\tilde{D}_2+\beta\bar{S}_2 \) and \( \tilde{p}_{i,2}=\gamma_i\tilde{D}_2+\delta_i\bar{S}_2 \) (for \( i=\text{CO, HC and NO}_x \))
and substituting in (c.3) and (c.5) yields,
\[ \tilde{f}_2 = \frac{\lambda_1}{2C} \left[ \frac{(C-L)\pi_1}{(1-\rho_1)} \right] \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] \left[ \frac{(C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1}}{1-\rho_2} \right] \]
\[ + \frac{\beta}{C} \left[ \frac{(C-L)\pi_1}{(1-\rho_1)} \right] \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] \left[ \frac{(C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1}}{1-\rho_2} \right] \]  
(5.48)
\[ \tilde{p}_{i,2} = \frac{\gamma_i}{2C} \left[ \frac{(C-L)\pi_1}{(1-\rho_1)} \right] \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] \]
\[ + \frac{\delta_i}{C} \left[ \frac{(C-L)\pi_1}{(1-\rho_1)} \right] \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right] \left[ (C-L)\pi_1 + \frac{2-\rho_1}{1-\rho_1} \right]. \]  
(5.49)

Claim C.3: Given a cycle length \( C \) and that the intersection is stable, the expected fuel consumption per minute, \( \tilde{f}_2 \), and the expected amount of pollutant \( i \), \( \tilde{p}_{i,2} \), released per minute (\( i=\text{CO, HC, NO}_x \)) are convex functions in \( \pi_1 \).

Proof: It is clear that, \( \tilde{f}_2 \) and \( \tilde{p}_{i,2} \) (for all \( i \)) are quadratic in \( \pi_1 \). Holding the cycle length \( C \) fixed, the second partial derivative of \( \tilde{f}_2 \), and \( \tilde{p}_{i,2} \) (for all \( i \)) with respect to \( \pi_1 \) are
\[
\frac{\partial^2 \bar{f}_2}{\partial \pi_1^2} = \frac{\alpha (C-L)^2}{C} \left( \frac{\lambda_1}{1-\rho_1} + \frac{\lambda_2}{1-\rho_2} \right) > 0
\]
and
\[
\frac{\partial^2 \bar{p}_{i2}}{\partial \pi_1^2} = \gamma_i (C-L)^2 \left( \frac{\lambda_1}{1-\rho_1} + \frac{\lambda_2}{1-\rho_2} \right) > 0
\]
respectively. Because all the constants are positive and the intersection is stable, both second partial derivatives are positive.

Figure C.14 shows \( \bar{f}_1 \) and \( \bar{f}_2 \) for all \( \pi_1 \) values. The \( \bar{f}_2 \) curve is just shifted upwards as is expected. The curves in the graphs are for \( \lambda_1 = 3 \) veh/min, \( \lambda_2 = 5 \) veh/min, \( L=1/15 \) and \( s=1/30 \) (i.e., it takes two seconds to service a vehicle).

![Image](image_url)

Figure C.14  \( \bar{f}_1 \) and \( \bar{f}_2 \) values versus \( \pi_1 \) values.

Given \( \pi_1, \bar{f}_2 \) and \( \bar{p}_{i2}, i = \text{CO, HC, NO}_x \), are also strictly convex in \( C \).
Claim C.4: Given a proportion of usable green time awarded to the vertical approach and that the intersection is stable, \(\pi_1, \tilde{f}_2, \text{ and } \bar{p}_{i,2}, \text{ (i=CO, HC, NO_x)}\) are convex functions in C.

Proof: Taking the second partial derivative of \(\tilde{f}_2\) and \(\bar{p}_{i,2}\) (for i = CO, HC and NO\(_x\)) yields

\[
\frac{\partial^2 \tilde{f}_2}{\partial C^2} = \frac{\alpha L^2}{C^3} \left[ \frac{\lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2}{1-\rho_1} - \frac{\lambda_2 (1-\pi_1)}{1-\rho_2} \right] + \frac{\gamma \delta L}{C^3} \left[ \frac{\lambda_1 \pi_1 + \lambda_2 (1-\pi_1)}{1-\rho_1} \right] > 0
\]

and

\[
\frac{\partial^2 \bar{p}_{i,2}}{\partial C^2} = \frac{\gamma L^2}{C^3} \left[ \frac{\lambda_1 \pi_1^2 + \lambda_2 (1-\pi_1)^2}{1-\rho_1} - \frac{\lambda_2 (1-\pi_1)}{1-\rho_2} \right] + \frac{\gamma \delta L}{C^3} \left[ \frac{\lambda_1 \pi_1 + \lambda_2 (1-\pi_1)}{1-\rho_1} \right] > 0
\]

for i = CO, HC, NO\(_x\), respectively. Because all the constants are positive and the intersection is stable, both second partial derivatives are positive.

\[\square\]

C.2.3 Pareto Optimal Sets

This section contains a proof of Theorem 5.6 and also contains two efficient frontier curves for the performance measures.

Theorem 5.6: Assuming the intersection is stable, the \(\pi_i\) values (where \(0 \leq \pi_1 \leq 1\)) associated with the Pareto optimal set for \(\bar{d}_2, \tilde{f}_2\) and \(\bar{p}_{i,2}\) are:

(i) \(\mathcal{F}_* = (\pi_{1, \bar{d}_2}, \pi_{1, \text{NO}_x})\) if \(\lambda_1 > \lambda_2\);

(ii) \(\mathcal{F}_* = (\pi_{1, \text{NO}_x}, \pi_{1, \bar{d}_2})\) if \(\lambda_1 < \lambda_2\);

(iii) \(\mathcal{F}_* = \pi_{1, \bar{d}_2}\) if \(\lambda_2 = \lambda_1\).

Proof: Let \(\lambda_1 = \lambda_1'\) and \(\lambda_2 = \lambda_2'\) in theorem 5.5, where \(\lambda_1' = \lambda_1 (1-\rho_2)\) and \(\lambda_2' = \lambda_2 (1-\rho_1)\).

Figures C.15 and C.16 depict the efficient frontier for the delay and CO emissions performance measure pair and for the fuel consumption and CO emissions performance measure pair, respectively. As can be seen, the efficient frontiers for the measures under MODEL2 have essentially the same form as under MODEL1.
Figure C.15 Efficient frontier for the delay and CO emissions performance measures (MODEL2).
C.3 MODEL3: Moderately Heavy Traffic Model

This subsection contains derivations and graphs pertaining to the moderately heavy trafficked intersection model of Section 5.5.
C.3.1 Delay Measure

Figure C.17 depicts the $\bar{d}_3$ as a function of the $\pi_1$ values for three different cycle lengths. This figure matches Figures C.1 and C.11. The values in the curves are for $\lambda_1=8$ veh/min, $\lambda_2=12$ veh/min, $L=4$ seconds per cycle and $s=2$ seconds per vehicle. Again, as the cycle increases, the graphs shift upward as is to be expected.

C.3.2 Fuel Measure and Pollutions Measures

This subsection provides derivation for equations (5.64) and (5.65).

The expression for $\bar{D}_3$ is $\lambda \bar{d}_3$, where the expression for $\bar{d}_3$ is in equation (5.61):
\[
\bar{D}_3 = \frac{\lambda_1[(C-(L)\pi_1)]\rho_1 C}{2C(1-\rho_1)\lambda_1[(C-(L)\pi_1-p_1 C]} + \frac{1}{\lambda_1} + C-(C-L)\pi_1 + s(\frac{2-p_1}{1-\rho_1})
\]
\[
+ \frac{\lambda_2[(C-(L)\pi_1+L)]\rho_2 C}{2C(1-\rho_2)\lambda_2[(C-(L)(1-\pi_1)-p_2 C]} - \frac{1}{\lambda_2} + (C-L)\pi_1 + L + s(\frac{2-p_2}{1-\rho_2})
\]. \quad (c.6)

Now it is necessary to determine \(\bar{S}_3\). I approximate this quantity as \(\bar{S}_2\) plus the expected number of arrivals which occur during the time it takes the initial queue at the beginning red to clear. Making use of the approximation for \(E[Q(0)]\), equation (5.59), I obtain

\[
\bar{S}_3 = \frac{\lambda_1[(C-(L)\pi_1)]}{C(1-\rho_1)} + \frac{\lambda_2[(C-(L)\pi_1+L)]}{C(1-\rho_2)} + \frac{\rho_1 C}{[(C-(L)\pi_1-p_1 C]} + \frac{\rho_2 C}{[(C-(L)(1-\pi_1)-p_2 C]} - 1 \quad . \quad (c.7)
\]

Now using the fact that \(\bar{f}_3 = \alpha_D3 + \beta\bar{S}_3\) and \(\bar{p}_{i,3} = \gamma_i\bar{D}_3 + \delta_i\bar{S}_3\) (for \(i=CO,HC\) and \(NO_x\)) and substituting in (c.6) and (c.7) yields,

\[
\bar{f}_3 = \frac{\alpha}{\lambda_1[(C-(L)\pi_1)]\rho_1 C}{2C(1-\rho_1)\lambda_1[(C-(L)\pi_1-p_1 C]} + \frac{1}{\lambda_1} + C-(C-L)\pi_1 + s(\frac{2-p_1}{1-\rho_1})
\]
\[
+ \frac{\alpha}{\lambda_2[(C-(L)\pi_1+L)]\rho_2 C}{2C(1-\rho_2)\lambda_2[(C-(L)(1-\pi_1)-p_2 C]} - \frac{1}{\lambda_2} + (C-L)\pi_1 + L + s(\frac{2-p_2}{1-\rho_2})
\]
\[
+ \beta \left[ \frac{\lambda_1[(C-(L)\pi_1)]}{C(1-\rho_1)} + \frac{\lambda_2[(C-(L)\pi_1+L)]}{C(1-\rho_2)} + \frac{\rho_1 C}{[(C-(L)\pi_1-p_1 C]} + \frac{\rho_2 C}{[(C-(L)(1-\pi_1)-p_2 C]} - 1 \right] \quad , \quad (5.64)
\]

\[
\bar{p}_{i,3} = \gamma_i \frac{\lambda_1[(C-(L)\pi_1)]\rho_1 C}{2C(1-\rho_1)\lambda_1[(C-(L)\pi_1-p_1 C]} + \frac{1}{\lambda_1} + C-(C-L)\pi_1 + s(\frac{2-p_1}{1-\rho_1})
\]
\[
+ \gamma_i \frac{\lambda_2[(C-(L)\pi_1+L)]\rho_2 C}{2C(1-\rho_2)\lambda_2[(C-(L)(1-\pi_1)-p_2 C]} - \frac{1}{\lambda_2} + (C-L)\pi_1 + L + s(\frac{2-p_2}{1-\rho_2})
\]
\[
+ \delta_i \left[ \frac{\lambda_1[(C-(L)\pi_1)]}{C(1-\rho_1)} + \frac{\lambda_2[(C-(L)\pi_1+L)]}{C(1-\rho_2)} + \frac{\rho_1 C}{[(C-(L)\pi_1-p_1 C]} + \frac{\rho_2 C}{[(C-(L)(1-\pi_1)-p_2 C]} - 1 \right] \quad . \quad (5.65)
\]

C.3.3  Pareto Optimal Sets for Pairs of Measures in Terms of the C Values

The efficient frontiers for the the delay and CO emissions measures, and for the fuel and CO emissions measures as a function of the cycle length, \(C\), are presented in this section.
Figure C.18  Efficient frontier for $\bar{d}_3$ and $\bar{\rho}_{co,3}$ (MODEL3) as a function of $C$. 
Figure C.19  Efficient frontier for \( \bar{f}_3 \) and \( \bar{f}_{co,3} \) (MODEL3) as a function of \( C \).
Works Cited


