Development of guidelines for four phase traffic signal scheme in the Kingdom of Saudi Arabia

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Abstract

Most of the signalized intersection in the Kingdom of Saudi Arabia operate on the basis of One-Phase-per-Approach system. This study was concerned with four-leg intersection with the aim of analyzing and setting guidelines and warrants for the operation of such system.

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A Thesis Presented to the

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DEVELOPMENT OF GUIDELINES FOR FOUR PHASE TRAFFIC SIGNAL SCHEME IN THE KINGDOM OF SAUDI ARABIA

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Dedicated to

my beloved mother and brother M.N. Akhtar

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ABSTRACT

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ملخص

معظم الاشارات الضوئية فى المملكة العربية السعودية تعمل على اساس السماح بالمرور لجهة واحدة من التقاطع فقط فى لحظة معينة ،

هذه الدراسة تحلل و بالتالى تقترح القواعد و الارشادات لعمل هذه الاشارات على هذا الاساس .

درجة الماجستير فى العلوم

جامعة الملك فهد للبترول والمعادن
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CHAPTER-1

INTRODUCTION

1.1 GENERAL

Safety and Capacity of intersections are dependent to a large extent on the application of sound principles in phasing. Their misapplication causes delay, confusion and disrespect the traffic rules and regulations(1).

All power operated devices used for regulating, directing or warning motorists or pedestrians are classified as traffic signals. There is wide spread belief among laymen that traffic signals offer the solution to all traffic control and accidents problems at an intersection. In many instances signals have been installed where they were not warranted. The consequence has often been excessive delay, disobedience of signals, diversions to inadequate alternate routes and increased accident frequency(1).

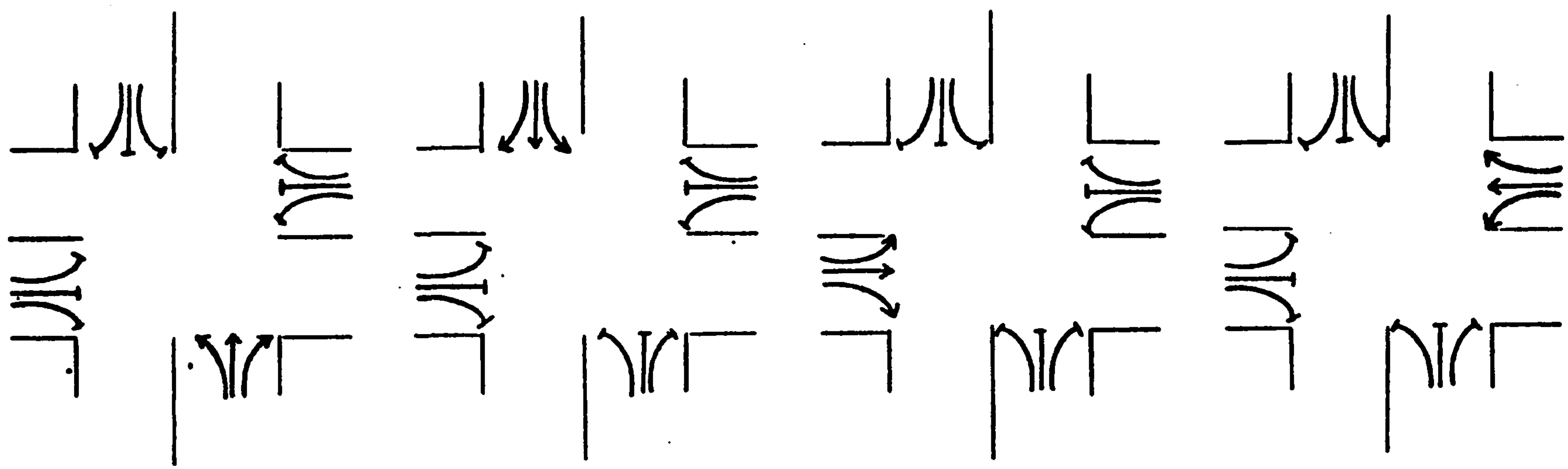
If properly used, traffic signal is a valuable device for the control of traffic and for its safe and efficient movement. Therefore, it is important that the selection and use of this traffic control device be based upon a thorough study of traffic and roadway conditions.

1.2 PROBLEM STATEMENT

In the Kingdom of Saudi-Arabia most of the intersections are operating under one-phase per-approach schemes. A typical scheme for four leg intersection is shown in figure 1.1. This phasing scheme allows each approach to operate separately on a pre-assigned phase. There are certain operational difficulties associated with this phasing scheme such as vehicle on a particular approach may have to wait for the other three approaches to clear the intersection until it is given the right of way to proceed. This signal phasing may cause the unnecessary stops and delays, driver frustrations, rear end collisions and increases in travel time and fuel consumption.

1.3 RESEARCH OBJECTIVES

The overall objective of this Thesis is to develop warrants and guidelines for using the one-phase per approach scheme. These warrants will be developed for the case of four-leg isolated signalized intersections, limited to equal distribution of approach volume.



PHASE 1 PHASE 2 PHASE 3 PHASE 4

FIGURE 1.1. EXISTING PHASING SCHEME

1.4 SCOPE OF THE STUDY

The study will be limited to four phase fixed time signalized intersections which are common in the Kingdom of Saudi Arabia. The effects of four phase signal scheme will be studied in terms of various factors, such as the cycle length, lost time per phase, headways, geometric configuration, saturation flow ratio and volume capacity ratio at critical approach. The whole study will be performed using the data collected, and the use of a simulation for different demand volumes and cycle lengths, which will provide the different measures of effectiveness and these in turn will facilitate the development of guidelines for four phase.

CHAPTER 2

LITERATURE REVIEW

The design of signalization scheme at an intersection is a complex process which involves the consideration of the prevailing conditions including the amount and distribution of traffic movement, traffic composition, geometric condition and detail of the intersection signalization. The following is a review of these elements together with the related literature and research.

2.1 PHASING

Phasing is the part of the cycle allocated to any combination of traffic movement receiving the right of way simultaneously during one or more intervals. The phasing depends mainly on the number of roads entering the junction and amount of traffic distributed on each approach and the various movements. In the signal design scheme, it is desireable to minimize the number of phases. Generally two phase control is appropriate for a four-leg intersection when there is no left turning movement or light left-turning movement of 60 vehicles per hour or less(1) in the approach, since in a two-phase, 60 second cycle, upto 2 vehicles per minute can be accommodated as waiting in the approach to cross following the green. The main consideration determining whether additional phases are required is whether special phases for particular movements such as left turn movement is to be

provided or not. There may be opportunities for left turners if the opposing traffic volume is not significant or less than left turn volume. But if there is no gap or gaps are insufficient in the opposing traffic and more left turners are waiting in the intersection, then special provision must be made for left turn treatment. It may be necessary to limit the cycle time either to avoid undue interference by left turning traffic to the through traffic flow in the intersection or to avoid locking of left turners. In case of heavy loaded traffic, it may be necessary to provide separate phase for opposing approaches of an intersection(1).

A 3-or 4-phase controller may be necessary at more complicated intersection with five or more roads and at ordinary cross roads where a pedestrian phase is required. Sometime a staggered intersection with a major road requires a 3-phase 4-part signal installation where the major road is given two green periods in each cycle and the less important roads just one each. Standard controllers give up to six phases, more than four can be vehicle actuated(1).

2.2 SEQUENCES AND TYPE OF PHASES

Once the choice has been made to use exclusive left-turn phases, two important decisions need to be considered. First the type of left-turn phasing to use, and second the sequence of the selected phasing. The Caltrans manual(2) lists the basic types of left-turn phasing that can be utilized and gives an idea of

advantage and disadvantage of different types of phasing. Which will be discussed later. After establishing the phasing type, the choice should be made for phase sequence type, whether they should lead or lag the through movement when dual left-turn phasing (also called protected only or basic three phase) is used or when Lead-lag phasing is used. Lead-lag phasing is less flexible when used for pretimed signal than for traffic responsive signals because of the capability of the later to vary the length of the leading (or lagging) phase or to skip it entirely.

2.3 CLEARANCE PERIOD

This period may consist of two portions, the yellow interval and the all red interval. Suitable values of the yellow time range from three to five seconds, depending primarily on approach speed. As mentioned in Caltran guidelines(2), It is generally accepted that long yellow periods become hazardous and if it is greater than 5 seconds then an all red period is used. Therefore, an all-red period is needed at very wide intersections. It is recommended to be 1 to 2 seconds(2).

2.4 CYCLE LENGTH

Caltrans(2) advises to use minimum possible cycle length. The cycle length should be no longer than is required to handle

the vehicular and pedestrian traffic movement to be controlled, There are several methods for finding cycle length for isolated intersections. Two approaches prevail for the cycle length calculation(2). The first one selects an optimum cycle length, which is defined in terms of minimizing total delay. Sometimes, minimum cycle length is also the optimal cycle length. This is not always true, particularly at heavily loaded intersections because the minimum cycle length is usually determined by pedestrian requirements. Normal practice limits the upper boundary to be about 120 seconds to control for the driver's patience (2). Webster(1) proposed a formula to compute the optimum cycle length. ebster

The Webster's model is given below.

$$C_o = \frac{1.5L+5}{1-Y_1-Y_2-\ldots Y_{n-1}} = \frac{1.5L+5}{1-\Sigma Y_{max}} \qquad 2.1$$

where $Y_1 \ldots\ldots\ldots Y_{n-1}$ are the maximum ratios of flow to saturation flow for phases 1,2....n, $Y = \Sigma y$ and L is the total lost time per cycle (in seconds). Australian method(3) have been modeled after Webster model, which determines the optimum cycle length. This model gives the result with a negligible difference of Webster model. Several other methods have been identified by Ale, Davidson, Bellis, the failure rate the canadian and the Kell method(2).

2.5 LENGTH OF INTERVALS(SPLIT)

After establishing the cycle length, the available green time must be splited among competing approaches. Caltrans guidelines propose that the green intervals be split proportional to the critical volumes in each phase which is called the volume-proportional method which is common practice in U.S(2). The NCHRP method proposed spliting the volume capacity ratio of the critical approach(2). The New Highway Capacity Manual(4) follows this as well. Webster proposed dividing the available effective green so the equal degree of saturation occur on the critical approach(1).

2.6 SATURATION FLOW

Saturation flow is the flow which would be obtained if there was a continuous queue of vehicles and they were given a 100 percent green time. It is generally expressed in vehicles per hour of green. Saturation flow is a fundamental parameter in Traffic signal design. Saturation flow is affected by factors such as gradient, traffic composition, left turn volume, parking facility, pedestrian flow area type Central Business District (CBD) or Non-CBD(4).

In Australia, there are several research which have been conducted in th context of improving the traffic signal design.

One of these research(3) proposed various recommendations in this connection. Degree of saturation (the ratio of volume to the capacity of the approach or the whole intersection, V/C) was recommended as a simple measure of the operating condition at signalized intersection instead of intersection flow ratio (the flow rate divided by the saturation flow rate, V/S), since the intersection flow ratio does not indicate the sufficiency of the cycle length which is allocated to a signalized intersection because the performance of an intersection depends on the length of cycle as well as the lost time.

At present intersection flow ratio is used as a simple measure of intersection operating conditions for preliminary design purpose in the Australian design practice(3). A value of saturation flow ratio of 0.70 and 0.75 is specified as an absolute upper limit. However, it is misleading to use saturation flow ratio as a measure of intersection performance, especially in the case of alternative analysis and when one or more signal phases have green times which do not satisfy vehicle or pedestrian's minimum green time requirements. For the critical values of saturation flow ratio greater than 0.65 intersection performances depend on the value of cycle time and lost time(3). This leads to the results that intersection degree of saturation is better tool in signal design than the flow ratio since it allows for lost time and cycle time as well as the saturation flow ratio. It is recommended that an acceptable maximum degree of saturation of 0.90 is used

for general design purpose.

2.7 CAPACITY

Intersection approach capacity is the maximum rate of flow which may pass through the intersection under prevailing roadway and signalization conditions. Bad signal design affects the intersection capacity. The available procedures(3,4) of the capacity analysis of intersections involve the computation of v/c ratios for individual movements and a composite v/c ratio for the sum of the critical movements or lane groups within the intersection. These V/C ratios, in turn, can be translated into delay values which is a measure of the level of service of the individual movement or the overall intersection as a whole.

2.8 Left-Turn Treatment

Traffic engineers are frequently faced with design decisions as whether to incorporate a protected left turn phasing. In Florida the Institute of Transportation Engineers studied the left turn problem and established some guidelines concerning the left turn phase design decision(5). Among these guidelines, one should recognize that as the number of phases increases some operational problems are likely to generate such as delays queue development, fuel consumptions. At any signalized cross intersection, left-turn movements are frequently recognized as

highly problematic operational elements. A protected left turn phase is warranted when the demands from the left-turn approach exceeds maximum unprotected flow rates. There is some research(5,6,7,8) undertaken for determining left turn capacity in which guidelines were developed for implementation of left turn phases and bays. It has been observed that in the four leg signalized intersection, sometimes left turn movements are very critical and they need special treatments such as providing protection because of safety. But due to this treatment other types of problem occur like excessive delay, long queue, excessive left turn and fuel consumption. Randy et al(6) studied the comparison of protected left turn phasing with combination of protected and permissive with protected left turn phases program as either dual leading or dual lagging. Cycle and phase lengths were held constant throughout the pretimed portion of experiment. Dual leading left turn was compared to dual lagging with both arrangements being supplemented by permissive turning during through green phases. Based upon these analyses, they come to the following findings

(1) From traffic operation perspective, provision of permissive left-turn during the through green will always be beneficial regardless of the type of signal control or left-turn sequence pattern. Only in situations where safety concerns are an effective influence, then permissive left-turn should be prohibited. Data published in reference 8 indicates that safety

problems associated with permissive left-turn are not severe. Intersection approach speeds in excess of 45 mph are frequently cited as a reason for prohibiting permissive left-turns.

(2) There is no difference between dual leading and dual lagging sequence when permissive left-turns are prohibited. When permissive turning is allowed, dual leading sequence produce less vehicular delay than dual lagging sequences if pretimed signal control is used. Under actuated control, dual lagging sequence patterns tend to produce less vehicular delay.

3) Split left-turn sequence patterns tend to produce less vehicular delay where critical left-turn and through movements occur on the same approach.

2.9 DELAY

Delay is an important factor in signal design. In signal designing scheme a main objective is to minimize the overall intersection delay and improve safety. Delay is directly related to the cycle length and number of phases. It is estimated in Great Britain that delays at traffic signal amounts to about 100 million vehicle-hours each year(1).

Application of off-line optimization program could be beneficial. For example in a National timing optimization project in the United States(9) a saving of 15470 vehicle-hrs of delays

were realized in a before-and-after implementing the optimization plan which was designed on Transyt-7F package. In a comparative analysis of Left-Turn phase sequencing(6) composite left-turn phasing was compared to dual and split left-turn phasing, the delay was compared for two cycle length with at least one hour of simulated observations time collected for each case. Each study found that intersection delay was reduced when protected permissive left-turn phase was used(5,6). Field studies(6) were conducted in the states of Maryland, California, and Kentucky in which vehicular delay data were collected before and after installation of permissive left-turn regulations. Each study found that intersection delay was reduced when permissive lef turning supplemented the protected phase.

2.10 SAFETY

In Kentucky study(6), it was found that left-turn accidents increased from 44 in the year before to 78 accident in the year after due to an error in judging the gap in the opposing traffic or inability to understand the permissive signalization. Rear-end accidents and other types of accidents did not increase. It was recommended that caution be used in installing permissive phasing with approach speed of over 45 mph(6). In a FHWA project(6) it was found that left turn accidents may incre ase when permissive phasing is installed at a protected left turn only

signal. At intersection where protected left turn phasing does not exist, however, installation of protected with permissive left turn phasing may not cause any increase in traffic accidents.

One effect of improper phasing on safety is what is called "Trap condition"(10). A trap condition can occur when one intersection approach has a lagging protected left turn phase and the opposing approach has permissive left turns. In case of two opposing approaches that have permissive left turns, vehicles wait for a gap to commence their maneuver. If a gap is not available in one approach, vehicles stored in the middle of the intersection will wait until the end of the green in the opposite approach. If the through vehicles in this opposite approach, having the protected lagging left turn phase, are released as overlap with this protected lagging left turn phase, vehicles awaiting the gap will find themselves trapped in the middle of the intersection with no chance to turn since their green has already terminated and may conduct a dangerous maneuver in front of the through traffic which is thought to be given the same treatment as they have been given. Once the choice has been made of whether exclusive left-turn phases should be used, several implementation possiblities are presented. Two important choices need to be considered; first, the type of left-turn phasing to be used and second, the sequences of the selected phasing scheme. Eventually, there must be trade-off between efficiency and safety needs at an intersection. Findings of published research(6) on

the effects of left turn phase sequence on safety and operation may be summarized as follows:

* Permissive/protected sequence compared to protected only sequence produces significant reductions in vehicular delay.

* Permissive phasing does not produce statistically significant changes in accident experience.

Based upon different left-turn-treatment analyses the researchers found the following developments(6):

* Permissive-left-turn through green combination is beneficial. In the case where safety is a problem, permissive left turn should be prohibited. Permissive left turn should be avoided when the approach speed is more than 45 mph.

* No operational differences between dual-leading and dual-lagging sequences when permissive left turns are prohibited. When permissive turning is allowed, however, dual leading sequence produce less vehicular delay than dual lagging sequence if pretimed signal control is used.

* Split left turn sequence pattern tends to produce less vehicular delay where critical left turn and through movement occur on the same approach.

2.11 PEDESTRIAN SAFETY

Pedestrian safety is generally accepted as a part of the traffic signal design at intersection. The presence of pedestrian can have an important effects on the operation of signalized intersection.

A very common problem encountered by pedestrian crossing a divided highway is the interruption of the crossing maneuver on the median. It is common for a pedestrian who starts crossing on green upon arrival at the median to encounter a red signal on the second roadway and then having to wait for another whole cycle. Adequate design of the phase sequence and the duration can allow a pedestrian to complete his crossing maneuver in one stretch.

At traffic signal where a pedestrian phase is shown simultenously on both parts of the divided highway, it is suggested by HAKKERT, BEN YAKOV and ARBIT(11) that the minimum green period be extended. The extension should be calculated such that any pedestrian waiting for green who started to cross the first roadway, will cross the median and be allowed to commence crossing of the second roadway on green.

The effects of pedestrian signal and timing sometime were not found effective in contradiction to the purpose that the pedestrian signal is used to provide for their safety. Recent pedestrian safety research(12) has uncovered numerous problems regarding current pedestrian signalization practices. The lack of uniformity

in strategies and devices for pedestrian signal timing has been thought to contribute to the ineffectiveness of the signals in achieving improved pedestrian safety. Further, pedestrians have expressed considerable confusion and misunderstanding regarding the meaning of the flashing DON'T WALK indication for the clearance interval and the flashing WALK indication to warn pedestrians of turning vehicles(12).

In terms of the effect of pedestrian signal on accidents, Fleig and Duffy(12) found no significant reduction in the proportion of unsafe acts or pedestrian accidents after the installation of scramble-timed pedestrian signal at 11 locations. The author of the study concluded that pedestrian signals are not effective in reducing pedestrian accidents(12).

2.12 OPTIMIZATION

Optimization may be defined as the process of analysis which gives the maximum benefits or the best results. The existing methods for the optimization of isolated fixed time signalized intersection are applicable either to undersaturated stationary or oversaturated conditions. According to Cronge(13), there is no existing model for the undersaturated or oversaturated conditions. He developed models for both macroscopic and microscopic simulation, where the arrival of vehicles per cycle is obtained by generating random numbers in the first case and in

the second case gaps between vehicles are obtained similarly. Both models were compared in the stationary zone with reference to average delay and number of stops. The differences were found negligible for practical purposes and macroscopic simulation is used for further development to save computer time.

There are lot of computer software which have been developed for the optimization, designing and performance evaluation of signalized intersection. Research(14) was undertaken to develop guiding techniques to determine the optimal signal timing plans for the coordinated arterial system and isolated intersections.

The following is description of various models which are used in signal optimization computer program(14).

NETSIM AND SOAP "A TEST OF COMPATIBILITY"

Zoltan and James had carried a test of compatibility of NETSIM and SOAP(15). NETSIM is an example of complex digital simulation model while SOAP is a simple micro computer program. An example of a relatively easy to use tool is SOAP, which offers a practical method of signal timing and intersection performance evaluation in the form of a computer program. According to Zoltan and James both programs are accepted as producing realistic results but they are very different in their computational basis. SOAP is a deterministic, macroscopic model based on a set

of simple equations. NETSIM, on other hand, is a stochastic, microscopic, digital simulation model that handles each vehicle separately. It is based on car-following and lane changing rules; It considers different vehicle types; and also recognizes conflicts between left-turn and oncoming traffic as well as the impact of traffic that is backed up from the preceding intersection. It can be said that SOAP is a relatively simple method while in comparison NETSIM is very complex. This raises a very intriguing question: do SOAP and NETSIM produce compatible results under similar traffic conditions? A compatibility test is performed with the following findings:

(1) There is a 7.5 or 8 seconds basic difference in average delays between SOAP and NETSIM in the range where delays do not increase very rapidly with increased intersection volumes.

(2) The pattern of delays predicted by SOAP and NETSIM are very similar in all cases. The relatively easy to use SOAP produced results entirely compatible with the much more complex, stochastic NETSIM, if the differences in the definition of delay and fuel consumption in the two models are taken into consideration. This should increase our trust in the reliablity of both methods.

PASSER II-80: It is an acronyms of Progression Analysis Signal System Evaluation Routine is an arterial optimization model to assist in determining optimal traffic signal timings for progression along an arterial considering various multi phase

sequences. This model was developed at Texas A & M University, Texas Institute of Transportation. It is written in Fortran. This program can be classified as a macroscopic, deterministic optimization model. It is primarily designed to calculate green splits, phase sequence and offset for signalized intersections along an arterial to maximize arterial progression and to reduce delay for a given set of traffic flow conditions. Passer II-80 calculates the v/c ratios and green times from a simple two-phase signal sequence to a complex eight-phase signal control. The methodology follows Webster's green split concept. At each intersection, the main street green phase sequence of left turn first through movement first, leading green, and lagging green can be evaluated by this program. This program can determine which of the four-phase sequence provide maximum progression.

TRANSYT-7F: It is an acronyms of Traffic Network Study Tool

This is the most widely used signal timing program. Originally it was developed by Transportation Road Research Laboratory in England in 1968. This latest version of the program provides optimum signal timing for a signalized intersection on an arterial or a network to achieve significant reduction in stops, delay and fuel consumption. The program optimizes split and offsets for a given cycle length and signal phasing. There is no provision to optimize cycle length. It is possible to perform multiple runs for various cycle length. The program is capable of evaluating a

coordinated network or an arterial of upto 50 intersections (nodes) with upto 250 directional links. The program deals explicitly with pretimed control signals. The program is capable of dealing with actuated controls as well. Only signalized intersections are directly modeled but the program has the capability to approximate sign-control intersection as signalized intersection.

SOAP 84:It is an acronym of SIGNAL OPERATION ANALYSIS PACKAGE This program was developed by Kenneth G. Courage of the university of Florida Transportation Research Center in 1977(16) SOAP is a microcomputer version written in basic. It carried out a complete design for signalized intersection timing, including calculation of the optimal cycle length and splits. It produces an evaluation of the intersection performance in terms of delay, stops and fuel consumption. A left turn analysis is also performed with separate capacity calculations made for protected phases, unprotected phases, and clearance intervals. SOAP has the capability to perform design and evaluation summary. It needs the intersection configuration and input the appropriate data, then SOAP produces all legitimate phasing patterns. It internally analyzes each pattern and selects the one which can be executed using the minimum amount of green time. A trial and error optimization procedure is used to find the cycle length which produces the minimum total delay, subject to constrains which govern the amount of queuing which can be tolerated.

Analysis is accomplished by computing the various measure of effectiveness which are delay, stops, fuel consumption, degree of saturation and left turn conflicts. More informations about the program is given in Appendix A.

CHAPTER 3

METHODOLOGY

3.1 GENERAL

The effects of four Phase signal scheme will be studied in terms of various factors, such as the Cycle length, lost time per phase, headways, geometric conditions, saturation flow rates and volume capacity ratios at critical approaches. The methodology is explained with reference to Figure 3.1 which consists of four distinct modules. Calibration of SOAP84, Relationship between V/S and V/C Ratio, Development of Guidelines, and Application of guidelines. Each of these will be explained in the forthcoming section of this chapter following the next section.

3.2 JUSTIFICATION OF USING SOAP 84

Considerable amount of research effort has been directed in the past at the problem of efficient signal timing, resulting in a variety of tools, ranging from relatively easy to apply computer programs to complex digital simulation model. SOAP is relatively easy to use for signal timing. It offers a practical method of signal timing and intersection performance evaluation in a form of microcomputer program. It has the provision of both types of signal designing and evaluation. Such as Pretimed and Actuated. This Package is a part of Arterial Analysis Package(AAP), which is applicable for coordinated operations. While SOAP is applicable

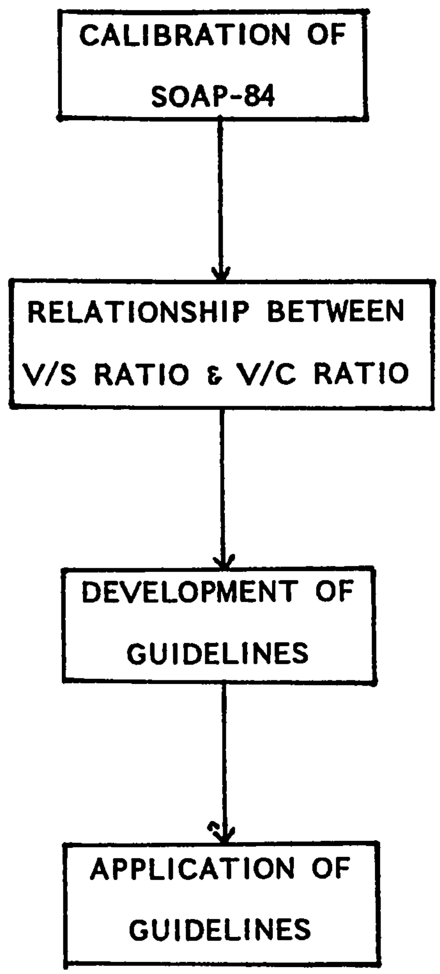


Figure: 3.1. METHODOLOGY

for isolated signalized intersection. But Meckemson et.al(15) found this package entirely compatible with much more complex Network Simulation(NETSIM) "if the difference in the definition of delay and fuel consumption in two models are taken into considerations".

SOAP requires limited coding effort and SOAP can be used quickly to test various phasing patterns and optimize signal timing. The basic difference between the SOAP and other programs is the examining of delay model. In SOAP, delay model is based on assumption of independent isolated operation. In other words, it is assumed that the arrival patterns of vehicles are not influenced by the operation of nearby signals. Specifically, the SOAP model assumes that a variation of arrivals is uniform with respect to any particular cycle. Description of SOAP is carried on in appendix A.

3.3 Module 1: CALIBRATION OF SOAP-84

Since the analysis is to be done by means of SOAP-84, there is need to calibrate it by using typical regional parameters, such as saturation flow rate, headways, actual delays and lost time. In particular the critical parameter is the saturation headway as all the other parameters depend on it. In essence, the calibration process would involve the assessment of the correlation between the simulated and observed intersection delays for different

TABLE 3.1
OBSERVED AND SIMULATED DELAYS

INTERSECTION NAME	APPROACH	VOLUME (VPH)	OBSERVED DELAYS (SEC/VEH)	SIMULATED DELAYS (SEC/VEH) HEADWAY (SEC) 2.0	2.5	3.0	3.5
K.A.AZIZ ST. vs 28th ST.	NORTH	2436	50	45	50	61	75
	SOUTH	1283	27	30	32	34	37
	EAST	909	40	32	35	39	46
AZIZIAH ST. vs DHAHRAN ST.	NORTH	923	42	38	44	56	70
	SOUTH	1085	40	39	40	41	42
	EAST	1994	36	33	36	41	51
	WEST	1449	32	32	33	35	37
MAKKAH ST. vs 28th ST.	NORTH	729	27	27	31	36	43
	SOUTH	665	37	39	48	63	73
HAMOUD ST. vs 28th ST.	EAST	378	22	21	23	25	28
	WEST	821	35	33	35	38	41
K.A.AZIZ ST. vs 10th ST.	NORTH	1075	38	35	41	53	51
	SOUTH	1246	45	41	44	53	51
MAKKAH ST. vs DHAHRAN ST.	NORTH	872	35	29	30	31	32
	SOUTH	1389	58	51	68	83	95
	EAST	974	41	30	32	34	38
	WEST	1810	45	42	45	50	57

Table 3.2 Simulated Headways

Headway (second)	Correlation Coefficient Between Actual and Simulated Delays
2.0	0.846
2.5	0.924
3.0	0.840
3.5	0.811

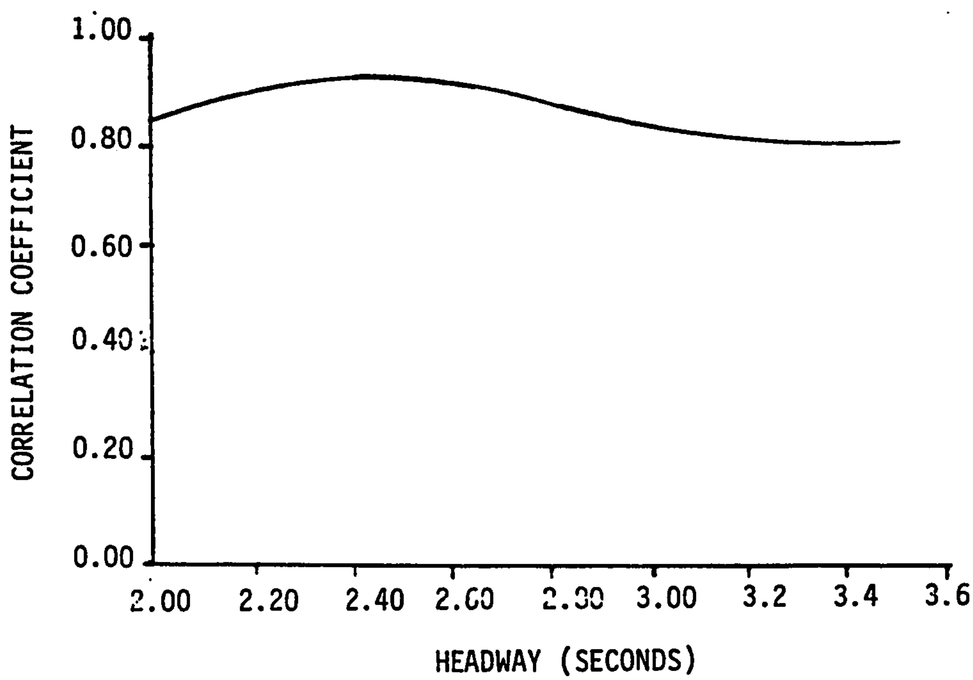


FIGURE 3.2. OPTIMUM HEADWAY (SECONDS)

values of saturation headways. The saturation headway which gives the highest correlation between the actual and simulated delays will be used for the further analysis. So, we look for such value of headway to use in the program, which can be the representative of the local traffic condition. For this purpose a field survey which included traffic volume and intersection delay studies was conducted as explained below.

Data required to calibrate the SOAP, includes flow rates, traffic conditions and signalization conditions. Details of the data collection are discussed in Appendix B. Another major set of data was the intersection delay study, in which the objective is to find out approach delays per vehicle at a signalized intersection.

The simulated delays were found using the observed volumes, green time, cycle length, lost time for different headways as shown in Table 3.1. The correlation coefficient between actual delays and simulated delays are given in Table 3.2 and plotted in Figure 3.2. It is found that the optimum range of Headway is from 2.2 seconds to 2.6 seconds. Therefore, for the analysis and design purpose, a saturation headway of 2.2 seconds was used as a reasonable parameter which gives a good performance in signal analysis and design(17), and which in fact the default value of the SOAP package(16).

Lost Time

Lost time is defined as the time during which the intersection is not effectively used by any movement; these times occur during the change interval (when the intersection is cleared), and at the begining of each phase as the first few cars in a standing queue experience start up delay. In the analysis we assumed lost time of 3.5 seconds, as a typical value at signalized approaches.

3.4 MODULE II Relationship between saturation flow ratio (V/S) and volume capacity ratio (V/C)

The purpose of this module is to establish a relationship between saturation flow ratio and volume capacity ratio for different cycle length, so that the degree of saturation can be determined with respect to geometric and signalization condition. The various steps involved in this process as shown in figure 3.3. The details are explained in section 3.4.2

3.4.1 DEMAND VOLUME AND ITS DISTRIBUTION

This is a sensitivity analysis conducted to explore the effect of different distribution of intersection volumes on the approach to the delay which is a measure of effectiveness of the signalization at the intersection as will be explained later. In signal designing, demand volume and its distribution play an important role, because the green times are allocated to each

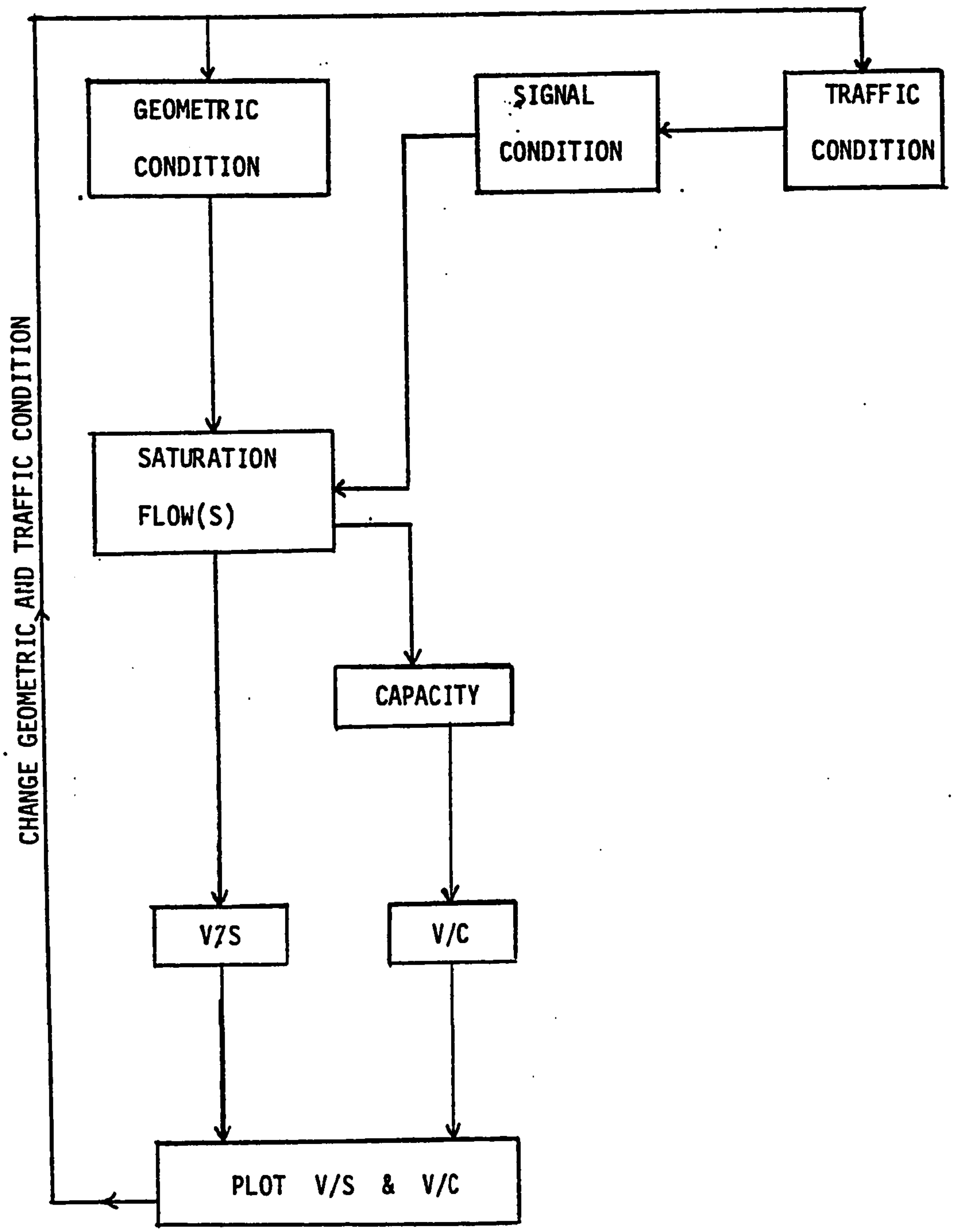


FIGURE. 3.3. MODULE II RELATIONSHIP BETWEEN V/S & V/C

phase according to the demand. In the analysis the impact of balanced and unbalanced flow on the overal intersection delay will be invistigated. For this purpose, different directional traffic splits such for North, South, East and West were attempted and shown in Table 3.3 for the case of 600 vph total intersection volume. Intersection delays are computed according to movement split shown in Table 3.3 for the optimum cycle as given by SOAP The Table shows that delay fluctuates in the narrow range of 3.57 to 3.88 veh-hrs with change in volume distribution. This low sensitivity of delay with respect to a given volume conveys an important findings. It is useful to assume that volume is equally distributed among the approaches for the purpose of simplicity of the analysis in this research as well as generalizing the delay values for the case of unequal distribution of volume, assuming that the appropriate cycle has already been selected for the given intersection volume and its distribution. In fact, the former findings is in line with intuition since it is logical to use the one phase-per-approach if volume on all approaches are equal since this gives equal treatment of the approaches in terms of allocating a separate phase for each. This implies green-cycle ratio of 0.25 on all approaches.

The later finding i.e generalizing the delay values for unequal distribution of volume, is going to be utilized in defining the optimum range of cycle lengths for a given volume as will be explained in section 3.4.3

TABLE 3.3
IMPACT OF EQUAL & UNEQUAL DISTRIBUTION OF VOLUME ON DELAY

DIRECTIONAL Split (%age) Volume(vph) N-S	E-W	Volume Distribution On Approach (Percentage) Vehicle/hr NORTH	SOUTH	EAST	WEST	CYCLE LENGTH (Sec)	DELAYS (VEH-HRS)
(80)	(20)	80	20	80	20		
480	120	384	96	96	24	95	3.88
		70	30	70	30		
		336	144	84	36	75	3.81
		60	40	60	40		
		288	192	72	48	65	3.76
		55	45	55	45		
		264	216	66	54	65	3.80
(70)	(30)	80	20	80	20		
420	180	336	84	144	36	75	3.81
		70	30	70	30		
		294	126	126	54	70	3.80
		60	40	60	40		
		252	168	108	72	65	3.74
		55	45	55	45		
		231	189	99	81	60	3.64
(60)	(40)	80	20	80	20		
360	240	288	72	192	48	65	3.76
		70	30	70	30		
		252	108	168	72	65	3.74
		60	40	60	40		
		216	144	144	96	60	3.61
		55	45	55	45		
		198	162	132	108	60	3.59
(55)	(45)	80	20	80	20		
330	270	264	66	216	54	65	3.80
		70	30	70	30		
		231	99	189	81	65	3.24
		60	40	60	40		
		198	132	162	108	60	3.61
		55	45	55	45		
		182	148	148	122	60	3.58
(50)	(50)	50	50	50	50		
300	300	150	150	159	150	60	3.57

3.4.2 ESTABLISHING RELATIONSHIP BETWEEN V/S & V/C

The performance of a signalized intersection depends on two classes of factors. First class is the geometric and traffic conditions, and the second is the Signalization conditions. Factors of the first set includes traffic condition such as peak hours demand, composition of Traffic, etc. It also includes Geometric conditions such as number of lanes, lane width and grade. This input information will be necessary to conduct an operational analysis to determine a saturation flow ratio (V/S) which is unique to every intersection, indicating the degree of saturation. The second class of factors include parameters such as cycle length, lost time, effective green time and green proportion. With the above stated parameters, the saturation flow ratio (V/S) and volume capacity ratio (V/C) are determined. The overall intersection performance depend on the saturation flow rate and demand capacity ratio. These parameters depict the level of performance of the intersection.

The critical V/C ratio for the intersection as a whole is defined as:

$$X_c = \Sigma(V/S)_{ci} * \frac{C}{C-L} \qquad (3.1)$$

Where

X_{ci} = critical V/C ratio for the intersection.

$\Sigma(V/S)_{cl}$ = the summation of flow ratio for all critical lane groups

C = cycle length (seconds)

L = lost time per cycle(seconds)

Although the above equation seems simple to "crank", it involves a trial and error prcedure in terms of selecting the appropriate cycle length for a desireable level of service. The task here, actually is to reach a quick device which enables the analyst to make such a decision as will be explained in the application in module IV. To move in this direction this equation is plotted in the form of V/C versus V/S for various cycle length as shown in Figure 3.4 which is facilitated through the use of SOAP. This figure is shown for the case 1 which is defined as two lanes on each approach(N = 2), lost time of 3.5 seconds (L = 3.5) and saturation headway is of 2.2 seconds (H = 2.2) and equal distribution of volumes among the approaches in consistancy with the findings and results of the previous sections. Similar plots Figure 3.5 and Figure 3.6 are done for case 2 and 3 for cases of N = 3 and N = 4 respectively. Appendix C contains summaries of SOAP outputs for the various cases.

The computed V/S ratio and V/C ratio for individual movements and for whole intersection indicate the general level of congestion which ultimately cause delays. With the help of this

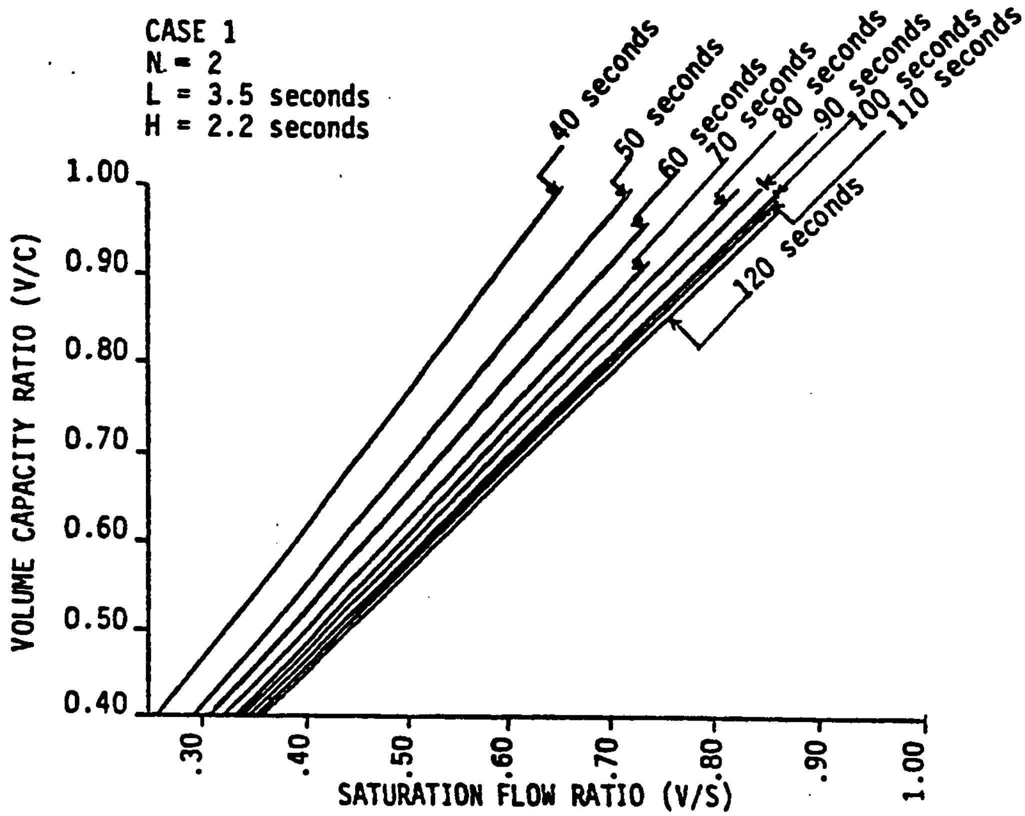


Figure 3.4. Relationship between V/S & V/C

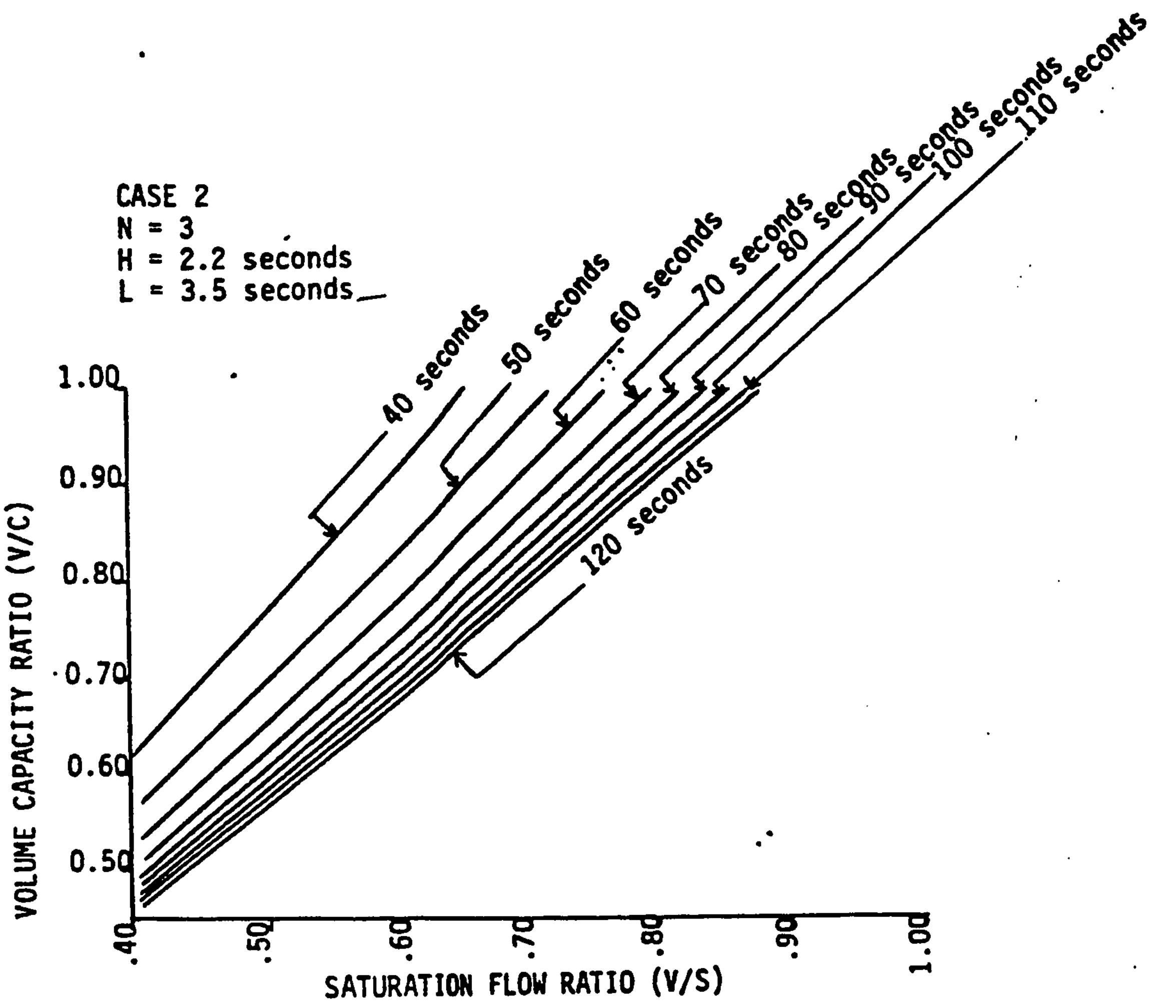


Figure 3.5. Relationship between V/S & V/C

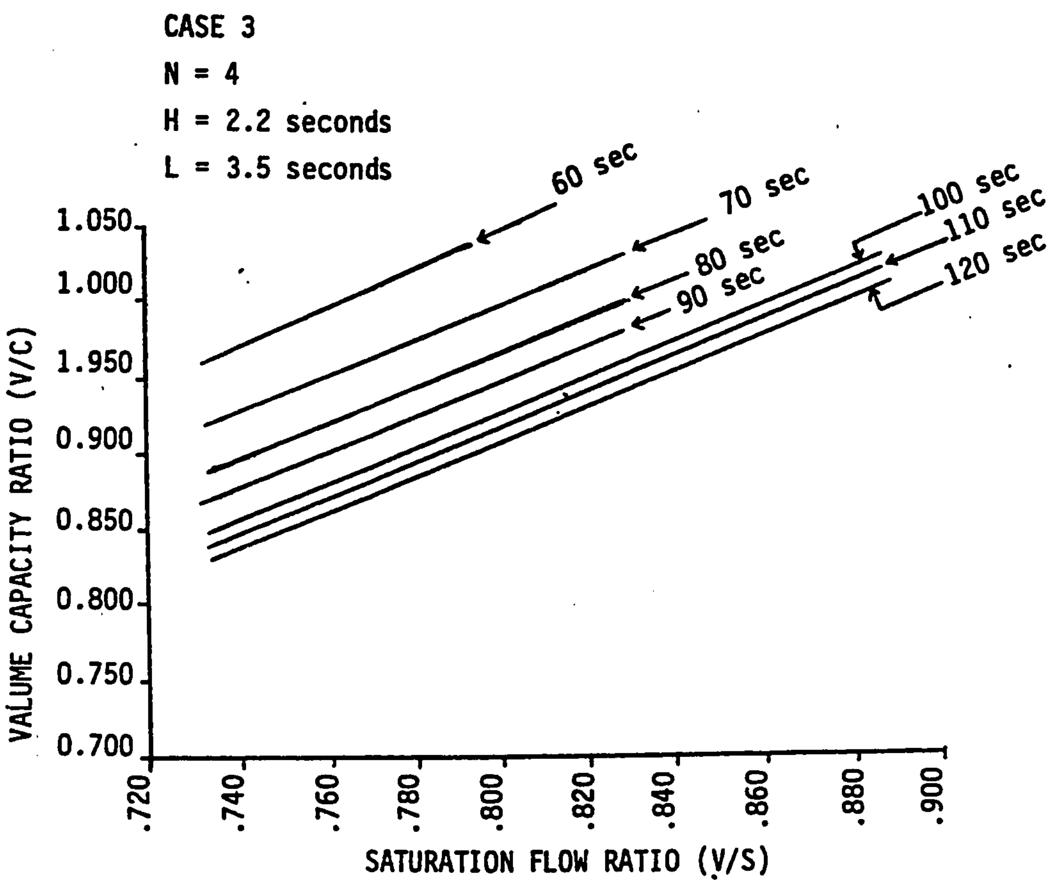


FIGURE 3.6. RELATIONSHIP BETWEEN V/S AND V/C RATIO

relationship, for a particular cycle time and for given set of flow and flow ratio, an optimum signal plan is calculated for a chosen phasing system. The method of obtaining an optimum signal is discussed in module III.

3.5 MODULE-III DEVELOPMENT OF GUIDELINES

In this module, with the help of relationship developed in module II Figure 3.4, 3.5 and 3.6, an optimum signal plan which tries to optimize a measure of effectiveness (for example average delay, fuel consumption or queue length is found.

Using the set of traffic and signalization condition stated in previous section various measure of effectiveness were attempted. Relationships were developed for delay as a measure of effectiveness of a given cycle as shown in Figures 3.7 to 3.9 for various demand volumes. These figures show the relation between average delays and cycle lengths for different cases. Using light traffic conditions the optimum cycle time as deduced from these figures may be very short. From a practical point of view including safety considerations, it may be desireable to regard a cycle time of about 40 seconds as the lower limit and 120 seconds as upper limit, since the gain in capacity with very long cycles is often insignificant. It can be noticed that delay is first decreasing then increasing as cycle length is increasing. Intuitively, for large volumes of traffic the figure shows short cycle lengths are not possible because the intersection will be

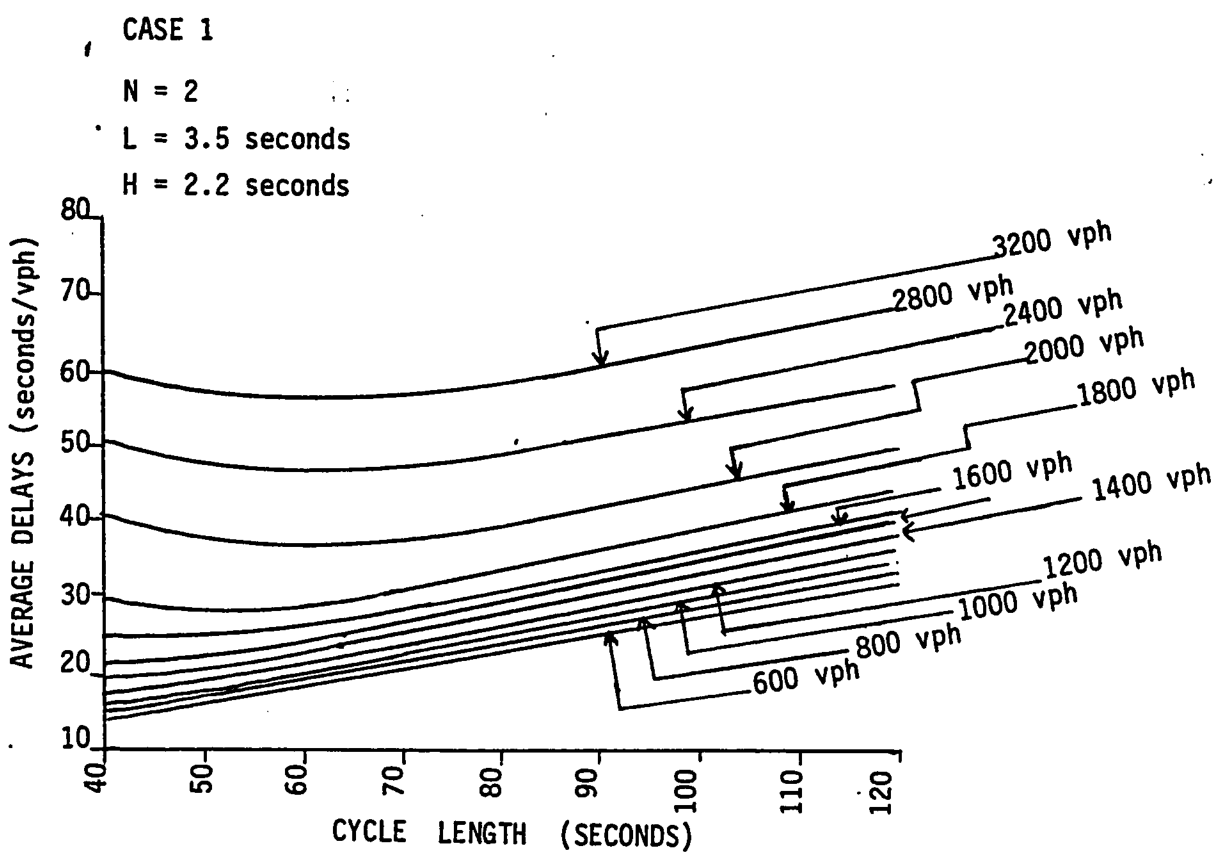


FIGURE 3.7 RELATIONSHIP BETWEEN CYCLE LENGTHS AND AVERAGE DELAYS (SECOND/VEHICLE PER HOUR)

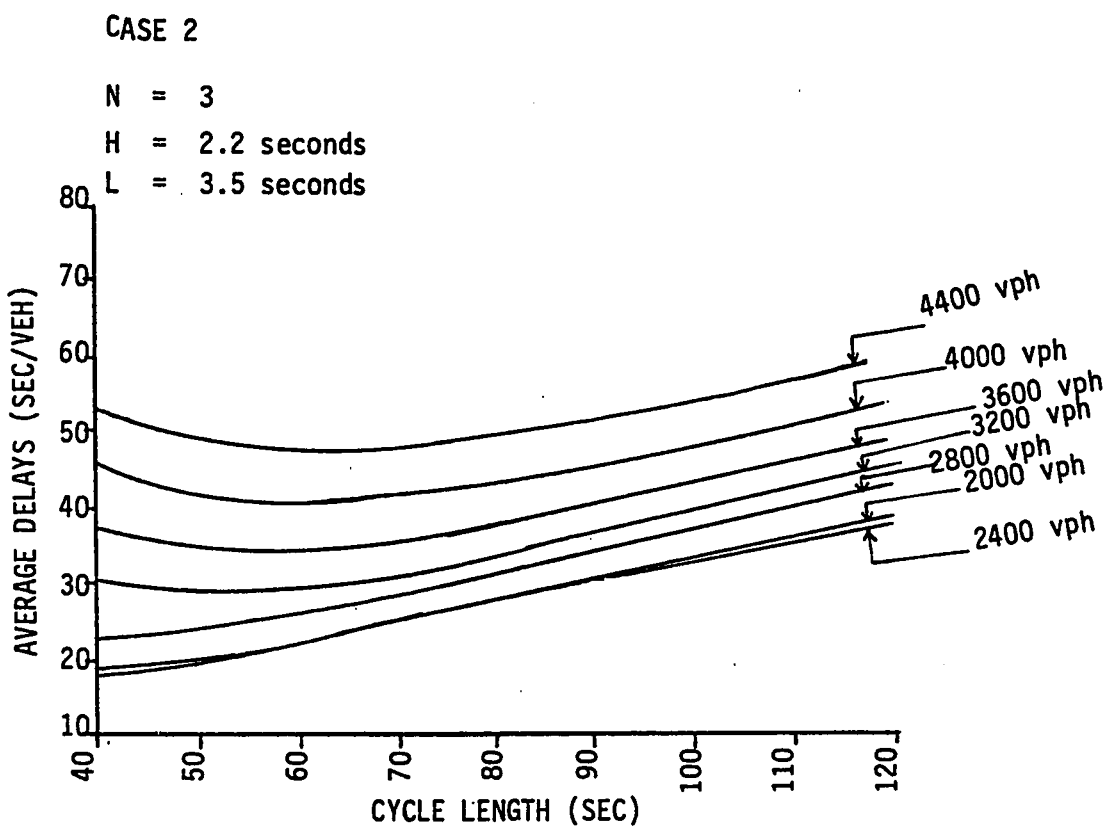


FIGURE 3.8 RELATIONSHIP BETWEEN CYCLE LENGTHS (SECONDS) AND AVERAGE DELAYS (SECOND/VEHICLE PER HOUR)

CASE 3

N = 4

H = 2.2 seconds

L = 3.5 seconds

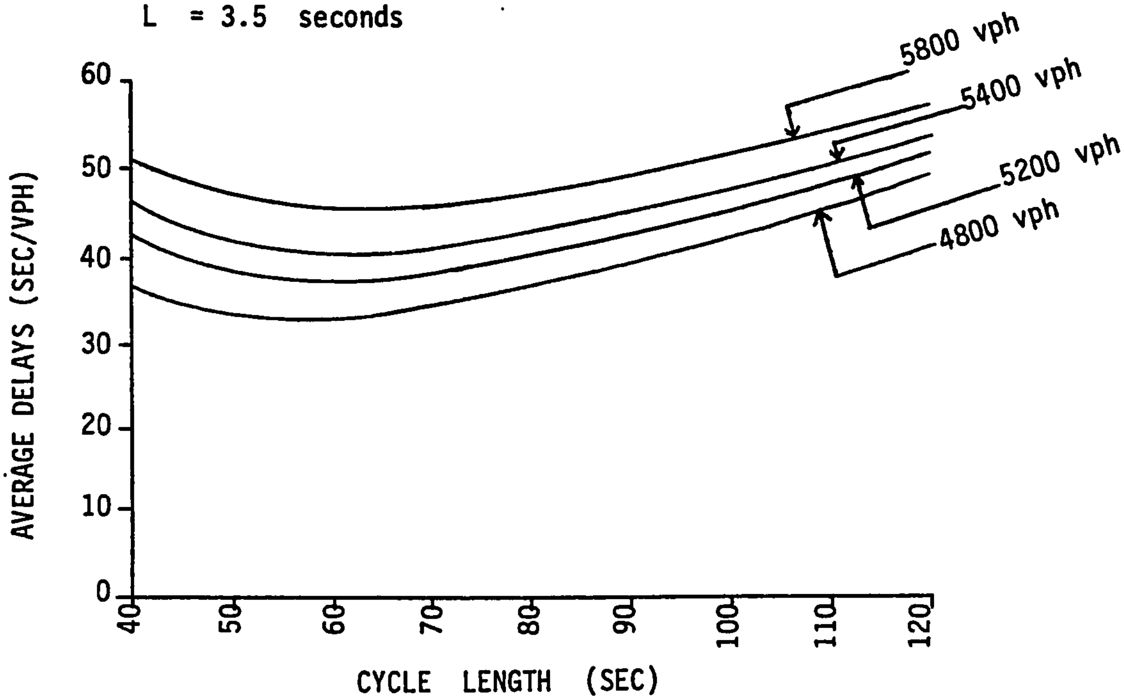


FIGURE 3.9 RELATIONSHIP BETWEEN CYCLE LENGTHS (SECONDS) AND AVERAGE DELAYS (SECOND/VEHICLE PER HOUR)

oversaturated. Based on the criterion of optimum delay Figures 3.7 to 3.9 can be utilized to obtain optimum ranges of cycle lengths for each demand volume. It has been found that for cycles within the range 4/5th to one and-a-quarter times of the optimum cycle the delay is never more than 5 to 15 percent above that given by the optimum cycle infact, this is in line to what has Webster found(1). Based on this, Tables 3.4, 3.5 and 3.6 show optimum cycle length ranges for cases 1, 2, and 3 respectively together with the feasible V/C ratios.

3.5.1 ESTABLISHMENT OF RELATIONSHIP BETWEEN CYCLE LENGTHS AND INTERSECTION VOLUMES

Tables 3.4 to 3.6 were plotted in convenient graphs as shown in Figures 3.10 to 3.12 respectively. These are for the three cases 1 to 3 respectively. Each of these figures show a band that defines the optimum range of cycle length for a given volume, together with the feasible V/C ratios, for a given cycle as shown by the contours. The area below the curve is not feasible for a decent level of service since V/C ratio is high triggering high value of starting and stopping delay which corresponds to cycles less than 4/5th of the optimum cycles of Figures 3.7 to 3.9. The area above the band of each Figures (3.10 to 3.12) indicate a region of cycle lengths that are too long for the given volume suggestion excessive delay due to long

TABLE: 3.4. Optimum Cycle Lengths(seconds) for Case 1

VOLUME (vph)	Optimum Cycle Length (seconds)	V/S	V/C
600	40---50	0.183	0.28--0.25
800	40---50	0.244	0.38--0.34
1000	40---55	0.306	0.47--0.41
1200	40---55	0.367	0.56--0.49
1400	40---60	0.428	0.62--0.56
1600	40---63	0.489	0.75--0.63
1800	40---63	0.550	0.85--0.71
2000	48---75	0.611	0.86--0.75
2400	56---88	0.733	0.97--0.87
2800	56---88	0.856	1.14--1.01
3200	60---95	0.978	1.27--1.15

TABLE:3.5. Optimum Cycle Lengths(seconds) for Case 2

VOLUME (vph)	Optimum Cycle Length (seconds)	V/S	V/C
2000	40---50	0.406	0.63--0.57
2400	40---63	0.489	0.75--0.68
2800	45---65	0.570	0.88--0.79
3200	55---85	0.652	1.00--0.84
3600	60---90	0.733	0.96--0.87
4000	65---100	0.815	1.15--1.00
4400	68---106	0.896	1.19--1.07

TABLE: 3.6. Optimum Cycle Lengths(seconds) for Case 3

VOLUME (vph)	Optimum Cycle Length (seconds)	V/S	V/C
4800	52---80	0.733	1.00--0.89
5200	60---88	0.794	1.10--0.94
5400	60---94	0.829	1.08--0.97
5800	72---112	0.887	1.10--1.01

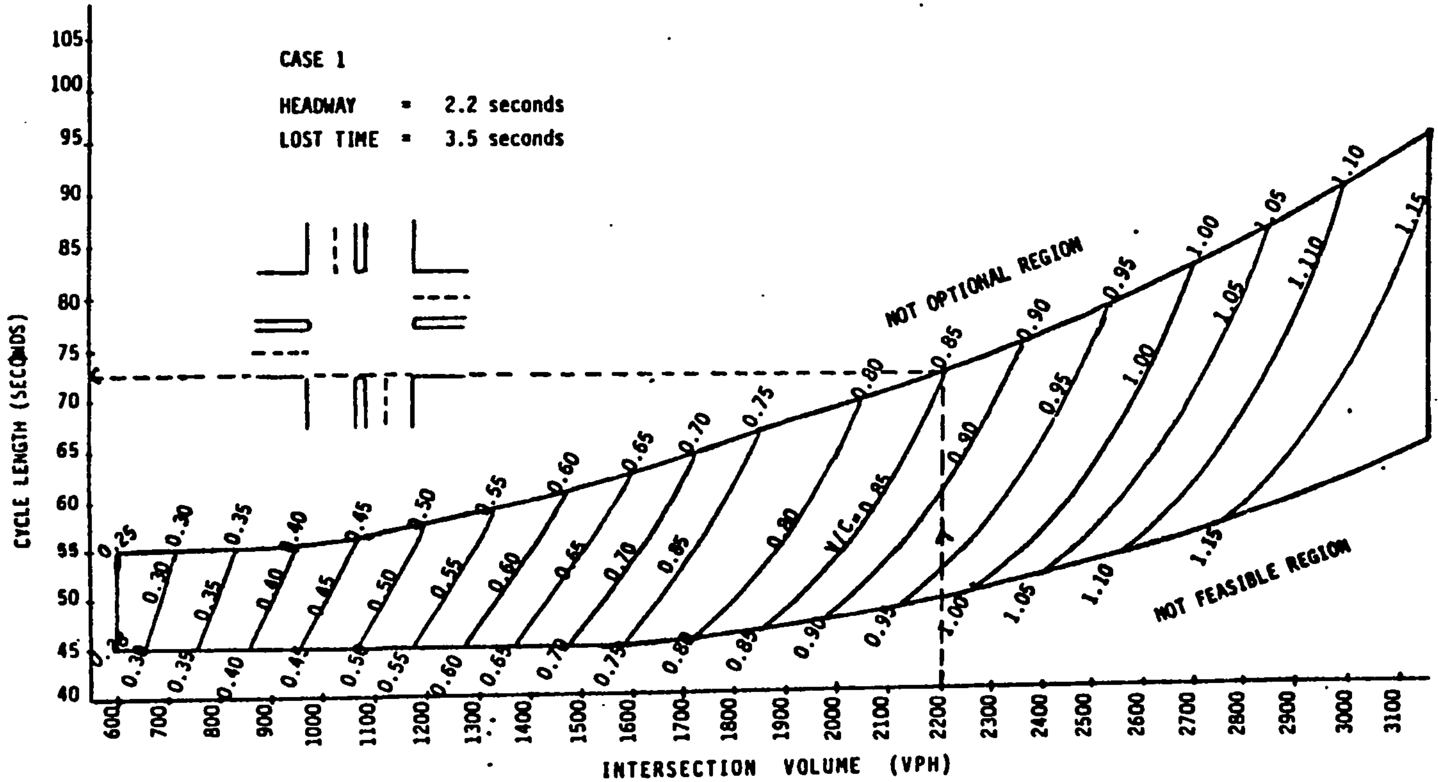


FIGURE. 3.10. NOMOGRAPH FOR OPTIMUM CYCLE LENGTH (SECONDS)

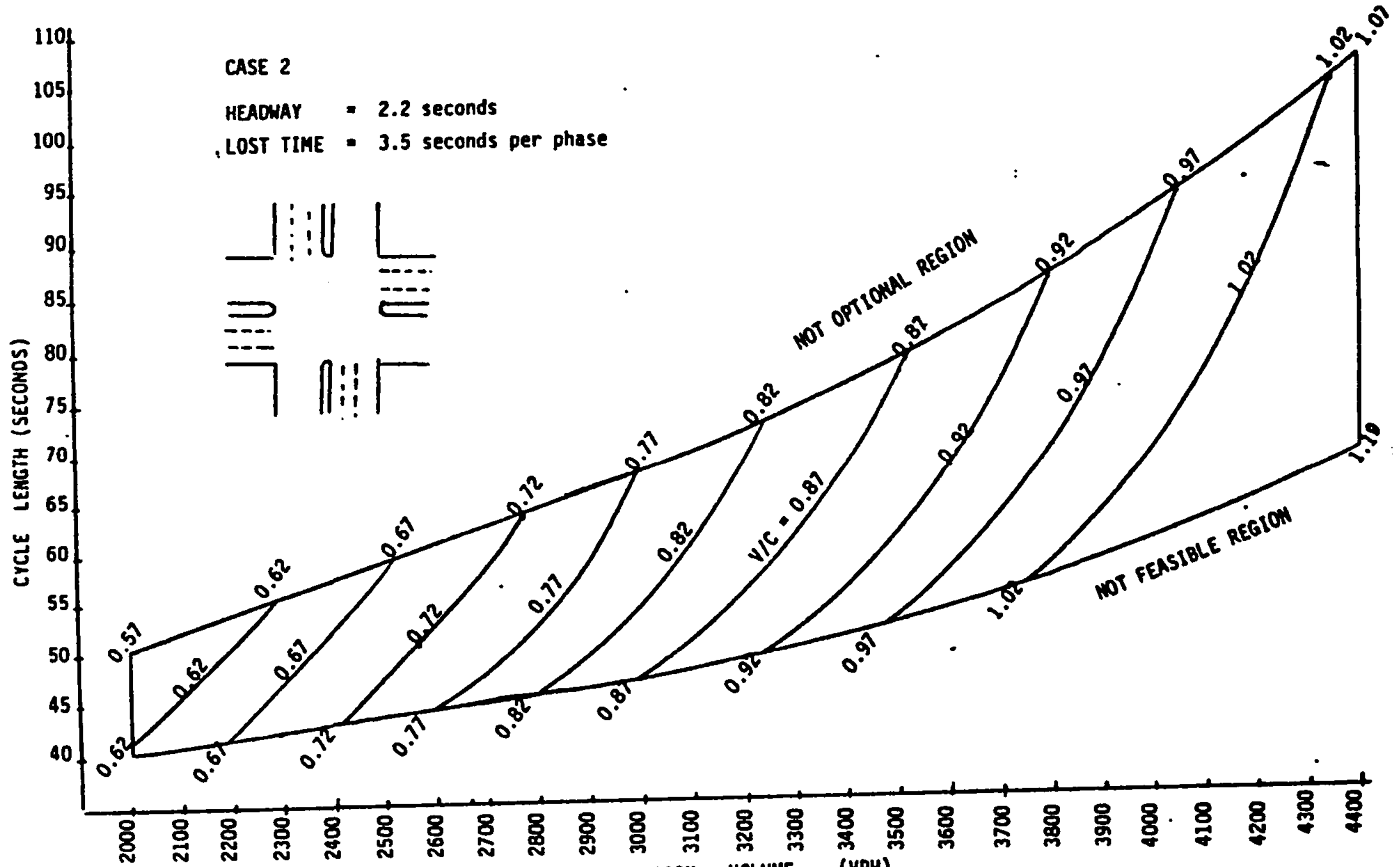


FIGURE 3.11. NOMOGRAPH FOR OPTIMUM CYCLE LENGTH (SECONDS)

CASE 3
HEADWAY = 2.2 seconds
LOST TIME= 3.5 seconds per phase

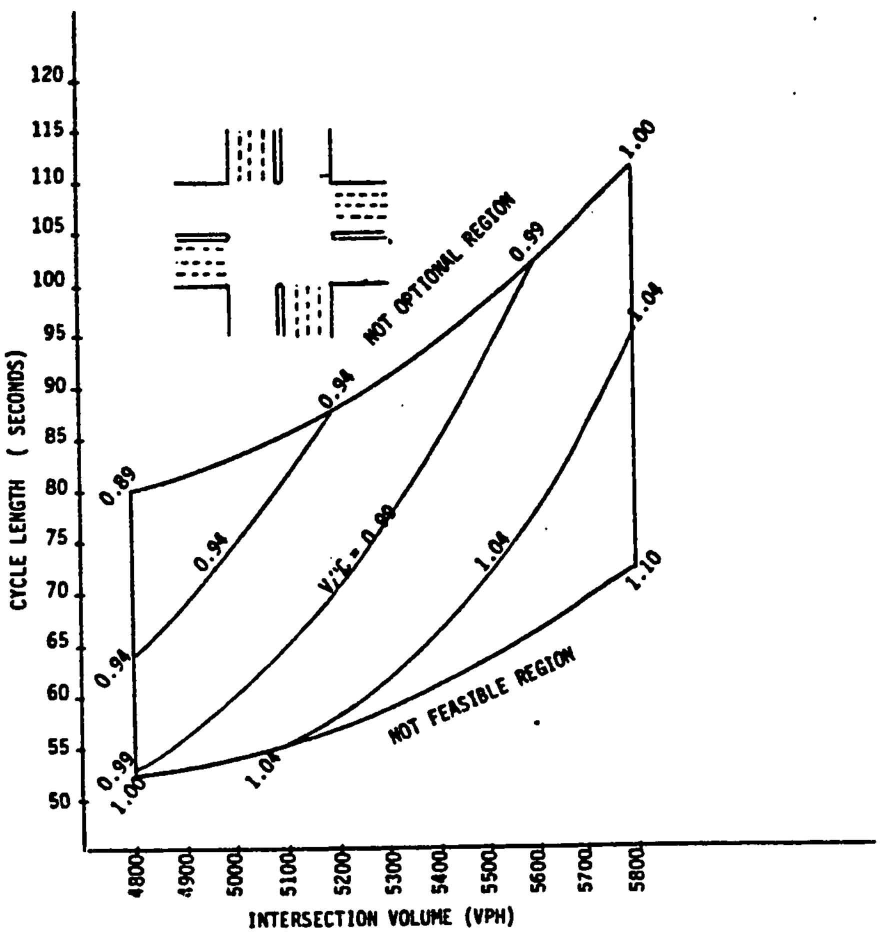


FIGURE.3.12. NOMOGRAPH FOR OPTIMUM CYCLE LENGTH (seconds)

waiting for the green on each approach. This corresponds to cycles greater than one and-a-quarter times of the optimum cycle value of in Figures 3.7 to 3.9. Therefore, considering these factors it is convenient to define a feasible range of cycle lengths for different volumes(vph) pertaining different geometrics. For the intersection with two lanes per approach, the optimum cycle lengths are found within the range of 45 seconds to 90 seconds for the total intersection volume 600 vph to 2900 vph. For the intersection with three lanes per approach, the optimum cycle lengths are found in the range of 40 seconds to 110 seconds for the whole intersection volume 2000 vph to 4400 vph. Similarly for four lanes per approach intersection the optimum cycle lengths are obtained within the range of 55 seconds to 115 seconds for the intersection volume 4800 vph to 5800 vph.

3.6 MODULE-IV: APPLICATION OF GUIDELINES

This module finally focuses on the application of the guideline. The guidelines are useful in a way that it avoids detailed and lengthy calculation for the selection of the appropriate four phase signal cycle. Application of these guidelines are as follow.

1. Choose the desired geometric condition (e.g number of lanes 2, 3 or 4 lanes),.
2. Choose Intersection demand volume vph;
3. Enter the graph with the above condtions;

4. Move upward and strike the desired V/C ratio(indicated on the curve);
5. Then move left to the cycle length and find the cycle length corresponding to the selected V/C ratio and demand volume.

This application can be illustrated by the following example.

Let the total intersection demand volume be 2200 vph with 2 lanes on each approach, Lost Time is 3.5 seconds and Headway 2.2 seconds. This is case 1 which is Figure 3.10. Move upward to strike the curve of desired V/C ratio, (in this case say V/C ratio is 0.85). Then move left to select the cycle length which is 73 seconds.

Figure 3.10 can be utilized in another way. For the given volume (2200 vph) we could obtain the following condition:

Feasible cycle lengths	Corresponding V/C ratio
48 seconds to 73 seconds	0.98 - 0.85
73 seconds to 92 seconds	0.85 - 0.78

This is useful in some ways. For example if the analyst is to decide on a system cycle for progression, then he would inspect the ranges of possible cycles at this particular intersection with the expected, corresponding V/C ratio. So, finally he would get a compromise for one cycle for a tolerable range of V/C ratio. The application of the other Nomographs (Figures 3.11 to 3.12) would be the same.

CHAPTER 4

CONCLUSION AND RECOMMENDATIONS

Conclusion of this research may be described as follow

1. From this study, it is found that the Saturation headway values found from 2.2 seconds to 3.0 second to represent the local traffic condition in the Kingdom of Saudi Arabia.
2. It is logical to use the one phase-per-approach scheme, if volume on all approaches are equal since this gives equal treatment of the approaches in terms of allocating a separate phase for each. This implies green-cycle ratio (g/c) of 0.25 on all approaches.
3. The general level of congestion of an intersection can be determined with the help of relationship between V/S and V/C ratio for a particular cycle time and for a given set of flow and flow ratio.
4. For large volumes of traffic, the short cycle lengths are not ssible because the intersection will be oversaturated.
5. Similarly for low volumes of traffic longer cycle lengths are not decent because the green time will be ineffective.
6. It is found that for cycle times within the range 4/5th to one-and-a-quarter times of the optimum value the delay is not more than 5 to 15 percent above that given by the optimum cycle.
7. The developed guidelines are useful in a way that it avoids

detailed and lengthy calculation for the selection of the four phase signal cycle.

8. For two lanes per approach in four leg signalized intersection the optimum cycle length should be within the range 45 seconds to 90 seconds for the intersection volume 600 vph to 2900 vph.
9. For three lanes per approach four leg signalized intersection, the optimum cycle length should be within the range 40 seconds to 110 seconds for the intersection volumes 2000 vph to 4400 vph.
10. For four lanes per approach four leg signalized intersection the optimum cycle length should be within range 55 seconds to 115 seconds for the intersection volume 4800 vph to 5800 vph.
11. Using of the nomograph may help to choose an optimum cycle length for four phase signal scheme.

RECOMMENDATIONS

During the course of this study the follwing recommendation has risen:

1. A similar study may be developed to determine the giudelines for the case of co-ordinated system.
2. Since this study was done for four phase four-leg isolated signalized intersection, a similar study may be conducted for

other than four leg intersection.

3. The optimum headway is found in the range of 2.2 seconds to 2.6 seconds and the guidelines have been developed for the saturation headway of 2.2 seconds. A sensitivity analysis is recommended to varify the effect of using 2.5 seconds in leiu of 2.2 seconds on the results of this study.

4. It is felt important that a sensitivity analysis is carried on to explore the effect of different values of lost time other than 3.5 seconds on the result found in this study.

APPENDICES

APPENDIX A

SOAP84

A.1 INTRODUCTION:

SOAP-84 is the acronyms of Signal Operation Analysis Package. This program was developed at the University of Florida, Transportation Research Center in 1977(15). This package works as a Tool for design and evaluation of the Operation of the Signalized intersection. The Package has been distributed by the Federal Highway Administration since 1979. The current version is SOAP-84.

A.2 SCOPE OF THE PROGRAM:

It offers a practical method of Signal Timing and intersection performance evaluation in the form of computer program. This package is based on a simple equation. SOAP determines the Measure of effectiveness(MOE's) for the comparison of Traffic Signal Control alternatives. SOAP gives two types of reports. First, the Signal Timing design report and second, Signal Timing Evaluation report.

The Signal Timing Design Report includes all the input Data and Various intermediate calculation during the design stage. The signal Timing calculations include volume to capacity ratio,

Saturation flow ratio, Left-turn Saturation capacity and the Green Time for each movement.

The Signal Timing Evaluation Report consist of two stages of evaluation the Left-turn check and the measure of Effectiveness. The Left-turn check stage shows the adjusted left-turn volumes with the type of protection associated with them. The protected turning movement is identified as a permissive "PERM" or restrictive "REST". The unprotected movement is identified by the word "NONE". The capacity is then presented in its appropriate components. A protected permissive movement is comprised of protected capacity during its green phase and unprotected capacity during its permissive phase.

The measure of effectiveness(MOE's) stage shows all the necessary results during the initial run and final run (Optimization), such as degree of saturation, delays, stop, fuel consumption. The delays are presented in two ways. The average delay per vehicle is shown for each movement. This is calculated by using Webster's basic delay method. Then the total delay is converted to vehicle - hour per hour using the movement volume. The stops are expressed in percent and the fuel consumption is expressed in gallon per hour.

This Package consists of two Programs. One is Data Input Manager (DIM) which used for input data and other is SOAP84, which used to carryout the analysis.

A.3 PROGRAM OPERATION

The SOAPDIM Program is written in Basic. The SOAPDIM Program consists of several cards. Each card has its own characteristics. Some of the important cards are briefly described below:

NAME OF CARD	TYPE	P U R P O S E
BEGIN CARD	Instruction	Initialize Data Arrays and Enters Initial parameter
CAPACITY CARD	Data	Enters Saturation Flow Values By Approach
CONTROL CARD	Parameters	Specifies Operation Parameters. For Pretimed Controller Dials
HEADWAY CARD	Data	Specifies Steady State Headway by Approach.
VOLUME CARD	Data	Specifies Traffic Volumes by Approach.
TIMING CARD	Parameters	Specifies Green Times Per Phase
SEQUENCE CARD	Parameters	Specifies Phasing Sequence
RUN CARD	Instruction	Initial SOAP run.

A.3.1. THE CONTROL SCREEN:

The control screen will be displayed throughout many of the operations to be performed. It is divided into three general areas:

- The status area: tells about data deck and mode

- The display area: tells about current and previous cards

- The command area: tells the program what to do

A.3.2. READING A DATA DECK

This mode is used to retrieve the file.

A.4 COMPUTATIONAL METHODOLOGY

A.4.1. MEASURE OF EFFECTIVENESS (MOE)

"The measurement of the quality of traffic service involves quantitative consideration of the frequency, expediency, smoothness and safety with which people and goods are moved from their origins to their destinations on a network of streets and highways" (ref: Special report # 130 TRRB).

"No single measure of the quality of traffic service can describe the performance of a highway system adequately for all operating and engineering goals. The following measure of effectiveness (MOE's) are available from SOAP for the comparison

of traffic signal control alternatives.

A.4.2. Degree of saturation: It is an indication of the general level of traffic congestion which may be anticipated. There are two measures of the degree of saturation:

a) The flow ratio (V/S)

b) The volume capacity ratio (V/C)

a) The flow ratio (V/S) may apply to individual approaches or to the intersection as a whole. The flow ratio is an important consideration in critical movement analysis and is expressed as

$$Y_i = \Sigma \frac{V}{S} \qquad A.1$$

(where i = 1 to n, n= # of phase)

b) Volume Capacity Ratio (V/C) this is determined by multiplying the saturation flow by the proportion of green time available to the movement or in equation it can be expressed as

$$X = \frac{q}{g/c*s} \qquad A.2$$

where x = v/c ratio

q = the traffic volume (vph)

g/c = green ratio

s = saturation flow rate(vphgl)

A.4.3. DELAY: Delay is well recognized as a useful measure of effectiveness in a traffic control system. Delay may be determined either by field measurement or estimated by analytical or simulation models. SOAP determines delay by WEBSTER's pretimed delay model as explained below:

1. The component due to uniform vehicle arrivals, which is derived analytically is the form:

$$D_1 = \frac{C\ (1-\Lambda)^{\ 2}}{2\ (1-\Lambda\ X)} \qquad A.3$$

where D_1 = the delay per vehicle (second)

C = cycle length (second)

$\Lambda = g/c$ & $X = \frac{V}{s}(Degree of Saturation)$

2. The component due to random arrivals:

$$D_2 = \frac{X^2}{2\ q\ (1-X)} \qquad A.4$$

where D_2 = the delay per vehicle (second)

$X = \frac{V}{S}$

q = approach flow (vehicle/second)

This component expresses the additional delay which results from the random arrival characteristics of the traffic stream.

A.4.4. ADJUSTMENT FACTOR

$$D_3 = -0.65\ (\frac{C}{q^2})^{\frac{1}{3}}\ \{X\ (2 + 5\ \Lambda) \qquad A.5$$

This term was developed semi-empirically to provide a better mathematical fit to the theoritical curve.

A.4.5. STOPS & FUEL CONSUMPTION

Stops are very significant in the estimation of fuel consumption at a traffic signal. The proportion of vehicles required to stop at a signal is equal to the number of vehicles in the queue at the beginning of the green plus the number of vehciels which join the queue while the queue is still discharging, divided by the average number of arrivals per cycle.

$$P_s = \frac{r\ s}{c}\ (s-q) \qquad A.6$$

where PS = proportion of vehciels required to stop

r = length of red (second)

s = saturation flow during green (vehicle/second)

c = cycle length (second)

q = flow rate

WEBSTER recommends the following expression for the vehicle stopped.

$$P_s = \frac{(1-\Lambda)}{(1-\Lambda X)} \qquad A.7$$

where P_s = The proportion of vehicle stopped

lamda= the proportion of effective green time and x = v/c ratio

The values of PS as computed above must be limited to a maximum of 1.0.

From the percent of stops, the excess fuel consumption may be estimated as:

$$E_s = \alpha * q * P_s \qquad A.8$$

where E_s = fuel consumption in gallon per hour

α = Fuel consumption in gallons of gasoline per stop

q = volume (vph)

P_s = *percent of stops*

Data complied by Claffey (NCHRP iii, 1971) indicate that, for a passenger car, the excess fuel consumed per stop is approximately 0.01 gallons when the approach speed is 30 mph.

The fuel consumption associated with delay as given by following equation:

$$E_d = B\ q\ d \qquad A.9$$

Where

E_d = Fuel consumption in gallons due to idling of vehicles.

B = Fuel consumption in gallons of gasoline per vehicle-hour of idling

q = volume (vehicle/second)

d = average delay per vehicle (second/vehicle)

The total fuel consumption

$$E_a = E_s + E_d \qquad A.10$$

APPENDIX B

DATA COLLECTION

B.1 DELAY STUDY: The principal objective of the intersection study is to collect data on the approach to a signalized intersection such that an accurate estimate of approach delay per vehicle can be made. Delay data was obtained from the compliment of Mr. Shah, who did the intersection delay study for his Graduate Research. He followed the recommended procedure by JHK & Associates, San Francisco, volume 3 (18), briefly described below.

Upon arrival at the site, one observer records the number of vehicles approaching to the intersection at every 13 seconds interval for 60 samples (say 13 minutes). The timing device for sampling points either a stopwatch or a cassette recorder is used. Simulteneously the other observer records the number of stopping and not stopping vehicles. Later on a data reduction form is filled out for each study. The observed delay study is shown in Table B.1

TABLE B.1. SUMMARY OF OBSERVED DELAYS

INTERSECTION NAME	APPROACH	VOLUME (VPH)	OBSERVED DELAY (SEC/VEH)
K.A.AZIZ ST.	NORTH	2436	50
VS	SOUTH	1283	27
28TH ST.	EAST	909	40
	WEST	1810	37
AZIZIZIAH ST.	NORTH	923	42
VS	SOUTH	1085	40
DHAHRAN ST.	EAST	1994	36
	WEST	1449	32
MECCA ST.	NORTH	729	26
VS			
28TH ST.	SOUTH	665	37
HAMOUD ST.	EAST	378	22
VS			
28TH ST.	WEST	821	35
K.A.AZIZ ST.	NORTH	748	29
VS			
22ND ST.	SOUTH	794	27

APPENDIX C

This appendix consist of the analytical results. The procedure for obtaining these results are discussed below. The same volume use as input data to all four approach and allocated the same green time to each phase (total phase = 4). In this analysis the cycle lengths have been considered from 40 seconds to 120 seconds as Lower and upper limit. Headway was used 2.2 seconds. By running SOAP with the above input information the output shown in Table C.1,C.2 and C.3. These tables show the measure of effectiveness computed by the program. These tables contained the results for the case 1, case 2 and case 3. A relationship was established between saturation flow ratio and volume capacity ratio as shown in figure 3.4 to 3.6.

TABLE C.1 Measure of Effectiveness (MOE)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 600 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.183	0.183	0.183	0.183	0.183	0.183	0.183	0.183	0.183
V/C	0.28	0.25	0.24	0.23	0.22	0.22	0.21	0.21	0.21
AVG DELAY (sec/veh)	13.980	16.08	18.30	20.58	22.86	25.20	27.54	29.88	32.28
INT. DELAY (veh-hrs)	2.33	2.68	3.05	3.43	3.81	4.20	4.59	4.98	5.37
STOPS (%)	82.70	80.10	78.40	77.20	76.30	75.50	75.0	74.50	74.10
FUEL (gph)	6.36	6.42	6.54	6.69	6.86	7.05	7.25	7.46	7.67
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	0.70	0.9	1.1	1.2	1.4	1.6	1.7	1.9	2.0

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 800 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.244	0.244	0.244	0.244	0.244	0.244	0.244	0.244	0.244
V/C	0.38	0.34	0.32	0.31	0.30	0.29	0.28	0.28	0.28
AVG DELAY (sec/veh)	12.84	16.96	19.22	21.51	23.89	26.28	28.66	31.09	33.48
INT. DELAY (veh-hrs)	2.85	3.77	4.27	4.78	5.31	5.84	6.37	6.91	7.44
STOPS (%)	83.40	82.20	80.40	79.20	78.30	77.50	76.9	76.50	76.10
FUEL (gph)	7.62	8.84	9.00	9.21	9.45	9.71	9.98	10.26	10.55
EXCESS (left-turn)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	1.0	1.2	1.4	1.7	1.9	2.1	2.3	2.5	2.8

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 1000 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.306	0.306	0.306	0.306	0.306	0.306	0.306	0.306	0.306
V/C	0.47	0.42	0.40	0.38	0.37	0.36	0.36	0.35	0.35
AVG DELAY (sec/veh)	15.94	17.92	20.19	22.57	24.98	27.43	29.92	32.40	34.60
INT. DELAY (veh-hrs)	4.43	4.98	5.61	6.27	6.94	7.62	8.31	9.00	9.61
STOPS (%)	86.90	84.30	82.50	81.20	80.30	79.60	79.00	78.5	78.10
FUEL (gph)	11.35	11.42	11.62	11.88	12.19	12.53	12.88	13.25	13.62
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	1.3	1.5	1.8	2.1	2.4	2.7	3.0	3.2	3.5

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 1200 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.367	0.367	0.367	0.367	0.367	0.367	0.367	0.367	0.367
V/C	0.56	0.51	0.48	0.46	0.44	0.43	0.43	0.42	0.42
AVG DELAY (sec/veh)	17.28	19.08	21.30	23.70	26.17	28.68	31.23	33.78	36.36
INT. DELAY (veh-hrs)	5.76	6.36	7.10	7.90	8.72	9.56	10.41	11.26	12.12
STOPS (%)	89.10	86.40	84.60	83.40	82.40	81.60	81.00	80.6	80.10
FUEL (gph)	14.15	14.19	14.42	14.74	15.12	12.53	15.97	16.42	16.89
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	1.5	1.9	2.2	2.6	2.9	3.3	3.6	3.9	4.3

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 1400 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.428	0.428	0.428	0.428	0.428	0.428	0.428	0.428	0.428
V/C	0.66	0.59	0.56	0.53	0.52	0.51	0.50	0.49	0.48
AVG DELAY (sec/veh)	18.95	20.44	22.60	24.99	27.50	30.00	32.68	35.30	37.95
INT. DELAY (veh-hrs)	7.37	7.95	8.79	9.72	10.69	11.69	12.71	13.73	14.76
STOPS (%)	91.30	88.60	86.80	85.80	84.50	83.80	83.20	82.70	82.30
FUEL (gph)	17.21	17.18	17.43	17.80	18.25	18.74	19.27	19.81	20.37
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	1.8	2.2	2.6	3.0	3.5	3.9	4.3	4.70	5.1

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 1600 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.489	0.489	0.489	0.489	0.489	0.489	0.489	0.489	0.489
V/C	0.75	0.68	0.64	0.61	0.59	0.58	0.57	0.56	0.55
AVG DELAY (sec/veh)	21.24	22.16	24.14	26.48	29.00	31.61	34.26	36.96	39.69
INT. DELAY (veh-hrs)	9.44	9.85	10.73	11.77	12.89	14.05	15.23	16.43	17.64
STOPS (%)	93.70	90.90	89.00	87.70	86.72	86.00	85.30	84.80	84.40
FUEL (gph)	18.38	20.45	20.68	21.10	21.61	22.18	22.80	23.43	24.09
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	2.1	2.6	3.1	3.5	4.0	4.5	5.0	5.40	5.9

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 1800 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.550	0.550	0.550	0.550	0.550	0.550	0.550	0.550	0.550
V/C	0.85	0.76	0.72	0.69	0.67	0.65	0.64	0.63	0.62
AVG DELAY (sec/veh)	24.50	24.40	26.04	28.26	30.76	33.38	36.08	38.84	41.62
INT. DELAY (veh-hrs)	12.25	12.20	13.02	14.13	15.38	16.69	18.04	19.42	20.81
STOPS (%)	96.00	93.20	91.30	90.00	89.00	88.20	87.60	87.10	86.70
FUEL (gph)	24.63	24.09	24.25	24.68	25.24	25.89	26.59	27.32	28.08
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	2.4	3.0	3.5	4.1	4.6	5.1	5.7	6.20	6.8

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 2000 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.611	0.611	0.611	0.611	0.611	0.611	0.611	0.611	0.611
V/C	0.94	0.85	0.80	0.76	0.74	0.72	0.71	0.70	0.69
AVG DELAY (sec/veh)	29.32	27.50	28.51	30.49	32.88	35.50	38.18	40.96	43.81
INT. DELAY (veh-hrs)	16.29	15.26	15.84	16.94	18.27	19.71	21.21	22.76	24.34
STOPS (%)	98.40	95.60	93.70	92.30	91.30	90.50	89.90	89.40	88.90
FUEL (gph)	29.46	28.27	28.24	28.63	29.22	29.93	30.71	31.53	32.39
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	2.7	3.4	4.0	4.6	5.2	5.8	6.4	7.0	7.7

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 2400 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.733	0.733	0.733	0.733	0.733	0.733	0.733	0.733	0.733
V/C	1.13	1.02	0.96	0.92	0.89	0.87	0.85	0.84	0.83
AVG DELAY (sec/veh)	40.33	37.41	36.54	37.32	39.09	41.37	43.98	46.65	49.50
INT. DELAY (veh-hrs)	26.89	24.94	24.36	24.88	26.06	27.58	29.28	31.10	33.00
STOPS (%)	100.00	100.00	98.60	97.20	96.20	95.30	94.70	94.20	93.70
FUEL (gph)	40.13	38.96	38.28	38.26	38.72	39.43	40.29	41.26	42.29
EX.LT (veh)	136.4	21.8	0	0	0	0	0	0	0
QUEUE (max veh)	3.3	4.2	4.9	5.7	6.5	7.2	8.0	8.8	9.5

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 2800 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.856	0.856	0.856	0.856	0.856	0.856	0.856	0.856	0.856
V/C	1.32	1.19	1.12	1.07	1.04	1.01	0.99	0.98	0.97
AVG DELAY (sec/veh)	50.68	47.15	46.44	55.13	48.92	51.13	53.55	65.26	58.56
INT. DELAY (veh-hrs)	39.42	36.67	36.12	36.75	38.05	39.77	41.65	43.51	45.55
STOPS (%)	100.00	100.00	100.00	100.00	100.00	100.00	99.80	99.30	98.80
FUEL (gph)	51.65	50.00	49.67	50.00	50.83	51.86	52.93	53.90	55.00
EX.LT (veh)	336.4	221.8	145.5	90.9	50.0	18.2	0	0	0
QUEUE (max veh)	3.9	4.9	5.8	6.8	7.8	8.8	9.70	10.6	11.60

TABLE C.1(continued)

CASE 1
N = 2
L = 3.5 seconds
H = 2.2 seconds

VOLUME 3200 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.978	0.978	0.978	0.978	0.978	0.978	0.978	0.978	0.978
V/C	1.50	1.36	1.28	1.22	1.19	1.16	1.14	1.12	1.11
AVG DELAY (sec/veh)	59.95	56.71	56.00	56.70	58.24	60.33	62.77	65.51	68.41
INT. DELAY (veh-hrs)	53.29	50.41	49.78	50.40	51.77	53.63	55.81	58.23	60.81
STOPS (%)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
FUEL (gph)	63.98	62.24	61.87	62.24	63.06	64.18	65.49	66.94	68.49
EX.LT (veh)	536.4	421.8	345.5	290.9	250.0	218.2	192.7	171.9	154.50
QUEUE (max veh)	4.4	5.6	6.7	7.8	8.9	10.0	11.10	12.2	13.30

TABLE C.2

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 2000 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.408	0.408	0.408	0.408	0.408	0.408	0.408	0.408	0.408
V/C	0.630	0.570	0.530	0.510	0.490	0.480	0.470	0.470	0.460
AVG DELAY (sec/veh)	17.77	19.85	22.37	25.05	27.81	30.60	33.43	36.25	39.09
INT DELAY (veh-hrs)	9.87	11.03	12.43	13.92	15.45	17.00	18.57	20.14	21.72
STOPS (%)	91.80	89.50	87.90	86.80	85.90	85.30	84.80	84.30	84.00
FUEL (gph)	24.29	24.51	25.04	25.71	26.46	27.26	28.09	28.95	29.83
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	3.4	4.2	5.0	5.8	6.5	7.3	8.1	8.9	9.6

TABLE C.2(continued)

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 2400 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.489	0.489	0.489	0.489	0.489	0.489	0.489	0.489	0.489
V/C	0.750	0.680	0.640	0.610	0.590	0.580	0.570	0.560	0.550
AVG DELAY (sec/veh)	18.58	20.02	22.240	24.73	27.34	30.00	32.71	35.44	38.19
INT. DELAY (veh-hrs)	12.39	13.35	14.83	16.49	18.23	20.00	21.81	23.63	25.46
STOPS (%)	94.60	92.30	90.70	89.60	88.80	88.10	87.60	87.20	86.90
FUEL (gph)	28.43	28.49	29.04	29.79	30.65	31.57	32.54	33.54	34.56
EXCESS (left-turn)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	4.2	5.2	6.1	7.1	8.0	9.0	9.9	10.9	11.8

TABLE C.2(continued)

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 2800 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.570	0.570	0.570	0.570	0.570	0.570	0.570	0.570	0.570
V/C	0.880	0.790	0.740	0.710	0.690	0.680	0.660	0.650	0.650
AVG DELAY (sec/veh)	22.00	24.20	26.12	28.62	31.37	34.22	37.17	40.15	43.16
INT. DELAY (veh-hrs)	17.11	18.82	20.32	22.26	24.40	26.62	28.91	31.23	33.57
STOPS (%)	97.30	94.80	93.20	92.10	91.20	90.60	90.00	89.60	89.20
FUEL (gph)	35.26	37.85	38.30	39.14	40.18	41.33	42.55	43.82	45.13
EX.LT (veh)	0	0	0	0	0	0	0	0	0
QUEUE (max veh)	5.1	6.2	7.3	8.5	9.6	16.7	11.9	13.0	14.1

123F

TABLE C.2(continued)

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 3200 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.652	0.652	0.652	0.652	0.652	0.652	0.652	0.652	0.652
V/C	1.000	0.900	0.850	0.810	0.790	0.770	0.760	0.750	0.740
AVG DELAY (sec/veh)	30.63	28.35	29.31	31.38	30.17	36.73	39.64	37.91	45.69
INT. DELAY (veh-hrs)	27.23	25.20	26.05	27.90	33.94	32.65	35.24	42.65	40.62
STOPS (%)	100.00	97.60	96.00	94.80	93.90	93.30	92.70	92.30	91.90
FUEL (gph)	48.34	46.36	46.35	47.08	48.16	49.43	50.82	52.28	53.79
EX.LT (veh)	4.4	0	0	0	0	0	0	0	0
QUEUE (max veh)	5.9	7.3	8.6	9.9	11.2	12.6	13.9	15.2	16.5

TABLE C.2(continued)

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 3600 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.733	0.733	0.733	0.733	0.733	0.733	0.733	0.733	0.733
V/C	1.13	1.020	0.960	0.920	0.890	0.870	0.850	0.840	0.830
AVG DELAY (sec/veh)	37.08	34.66	34.33	35.59	37.70	40.23	43.02	45.95	48.99
INT. DELAY (veh-hrs)	37.08	34.66	34.33	35.59	37.70	40.23	43.02	45.95	48.99
STOPS (%)	100.0	100.00	98.80	97.60	96.80	96.10	95.50	95.10	94.70
FUEL (gph)	58.73	56.79	56.17	56.51	57.45	58.72	60.19	61.79	63.48
EX.LT (veh)	136.4	21.8	0	0	0	0	0	0	0
QUEUE (max veh)	6.7	8.3	9.9	11.4	13.0	14.5	16.0	17.6	19.1

123F

TABLE C.2(continued)

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 4000 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.815	0.815	0.815	0.815	0.815	0.815	0.815	0.815	0.815
V/C	1.25	1.13	1.060	1.020	0.990	0.970	0.950	0.930	0.920
AVG DELAY (sec/veh)	45.42	41.51	40.73	41.62	43.28	45.39	47.89	50.63	53.55
INT. DELAY (veh-hrs)	50.47	46.12	45.26	46.24	48.09	50.44	53.21	56.26	59.50
STOPS (%)	100.0	100.0	100.00	100.00	99.60	98.90	98.40	97.90	97.50
FUEL (gph)	70.28	67.67	67.16	67.74	68.71	69.84	71.28	72.93	74.72
EX.LT (veh)	7.40	9.3	77.5	22.9	0	0	0	0	0
QUEUE (max veh)	268.4	153.8	11.1	13.0	14.8	16.5	18.3	20.00	21.8

TABLE C.2

CASE 2
N = 3
L = 3.5 seconds
H = 2.2 seconds

VOLUME 4400 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.896	0.896	0.896	0.896	0.896	0.896	0.896	0.896	0.896
V/C	1.38	1.25	1.17	1.12	1.090	1.060	1.040	1.030	1.020
AVG DELAY (sec/veh)	52.48	48.60	47.56	48.10	49.57	51.63	54.10	56.79	59.69
INT. DELAY (veh-hrs)	64.15	59.40	58.14	58.79	60.59	63.11	66.10	69.41	72.96
STOPS (%)	100.0	100.0	100.0	100.0	100.00	100.00	100.00	100.0	100.0
FUEL (gph)	82.49	79.64	78.88	79.28	80.35	81.86	83.66	85.65	87.78
EX.LT (veh)	404.4	289.8	213.5	158.9	118	86.2	60.7	39.9	22.5
QUEUE (max veh)	8.1	10.2	12.2	14.3	16.3	18.3	20.4	22.40	24.4

TABLE C.3

CASE 3
N = 4
L = 3.5 seconds
H = 2.2 seconds

VOLUME 4800 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.733	0.733	0.733	0.733	0.733	0.733	0.733	0.733	0.733
V/C	1.130	1.020	0.960	0.920	0.890	0.870	0.850	0.840	0.830
AVG DELAY (sec/veh)	36.33	32.85	32.76	34.29	36.60	39.30	42.21	45.90	49.10
INT. DELAY (veh-hrs)	48.45	43.81	43.68	45.72	48.79	52.39	56.28	61.18	65.43
STOPS (%)	100.0	100.00	98.90	97.80	97.00	96.30	95.80	95.70	95.40
FUEL (gph)	77.07	74.29	73.68	74.37	75.82	77.67	79.76	82.66	85.05
EX.LT (veh)	136.4	21.8	0	0	0	0	0	0	0
QUEUE (max veh)	10.0	12.5	14.8	17.1	19.4	21.7	24.0	26.3	28.6

TABLE C.3(continued)

CASE 3
N = 4
L = 3.5 seconds
H = 2.2 seconds

VOLUME 5200 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.794	0.794	0.794	0.794	0.794	0.794	0.794	0.794	0.794
V/C	1.220	1.100	1.040	0.990	0.960	0.940	0.920	0.910	0.900
AVG DELAY (sec/veh)	42.35	38.18	37.40	38.35	40.16	42.53	45.24	48.56	51.67
INT. DELAY (veh-hrs)	61.18	55.15	54.02	55.40	58.01	61.44	65.35	70.14	74.63
STOPS (%)	100.00	100.00	100.00	99.80	99.00	98.30	97.80	97.60	97.20
FUEL (gph)	88.71	85.09	84.41	85.15	86.27	87.99	90.07	92.82	95,34
EX.LT (veh)	236.4	121.8	45.5	0	0	0	0	0	0
QUEUE (max veh)	10.8	13.5	16.3	18.9	21.5	24.0	26.5	29.1	31.6

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TABLE C.3(continued)

CASE 3
N = 4
L = 3.5 seconds
H = 2.2 seconds

VOLUME 5400 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.829	0.829	0.829	0.829	0.829	0.829	0.829	0.829	0.829
V/C	1.29	1.17	1.100	1.030	1.000	0.990	0.970	0.950	0.940
AVG DELAY (sec/veh)	45.78	41.50	40.42	41.12	42.81	45.00	47.58	60.64	53.69
INT. DELAY (veh-hrs)	68.68	62.24	60.64	61.68	64.22	67.52	71.37	75.96	80.54
STOPS (%)	100.0	100.0	100.00	100.00	100.00	99.50	98.90	98.60	98.30
FUEL (gph)	95.43	91.56	90.61	91.23	92.75	94.44	96.47	99.07	101.6
EX.LT (veh)	288.4	173.8	97.5	42.9	2.0	0	0	0	0
QUEUE (max veh)	11.5	14.3	17.2	20.0	22.9	25.7	28.50	31.2	33.90

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TABLE C.3

CASE 3
N = 4
L = 3.5 seconds
H = 2.2 seconds

VOLUME 5800 vph

CYCLE LENGTH	40	50	60	70	80	90	100	110	120
V/S	0.887	0.887	0.887	0.887	0.887	0.887	0.887	0.887	0.887
V/C	1.37	1.23	1.16	1.11	1.08	1.05	1.030	1.020	1.000
AVG DELAY (sec/veh)	50.81	46.63	45.40	45.81	47.20	49.22	51.65	54.37	57.27
INT. DELAY (veh-hrs)	81.86	75.15	73.15	73.81	76.05	79.31	83.22	87.59	92.27
STOPS (%)	100.0	100.0	100.0	100.0	100.0	100.00	100.0	100.0	100.0
FUEL (gph)	107.12	103.1	101.9	102.3	103.6	105.6	107.93	110.55	113.36
EX.LT (veh)	300.4	265.8	189.5	134.9	94.0	62.2	36.7	15.9	0
QUEUE (max veh)	12.1	15.1	18.1	21.2	24.2	27.2	30.30	33.3	36.30

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