SIGNAL OPTIMIZATION AT ISOLATED INTERSECTIONS USING PRE-SIGNALS

A Thesis

by

TRISHUL AJIT PALEKAR

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

August 2006

Major Subject: Civil Engineering

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ABSTRACT

Signal Optimization at Isolated Intersections Using Pre-Signals.

(August 2006)

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This research proposes a new signal operation strategy aimed at efficient utilization of green time by cutting down on the start up and response loss times. The idea is to have a "pre-signal" on each main approach a few hundred feet upstream of the intersection in addition to the main intersection signal, which is coordinated with the pre-signal. The offset between the main and pre-signal ensures that the majority of start up losses does not occur at the main signal. The benefits of the system under various traffic conditions were evaluated based on analysis of the queue discharge process and Corridor Simulation (CORSIM) study. The proposed measure should reduce the travel time and total control delay for the signalized network.

To attain the objective the following two studies were undertaken:

1. Development of a queue discharge model to investigate the expected benefits of the system.
2. Simulation of the system: In the second part of the research, the proposed strategy was tested using CORSIM to evaluate its performance vis-à-vis the baseline case.

The queue discharge model (QDM) was found to be linear in nature in contrast to prior expectations. The model was used to quantify the benefits obtained from the pre-signal system. The result of this analysis indicated that the proposed strategy would yield significant travel time savings and reductions in total control delay.

In addition to the QDM analysis, CORSIM simulations were used to code various hypothetical scenarios to test the concept under various constraints and limitations. As per expectations, it was found that the system was beneficial for high demand levels and longer offsets. The upper limit on offsets was determined by visual observation of platoon dispersion and therefore the maximum offset distance was restricted to 450 feet. For scenarios where split phasing was used, the break even point in terms of demand level was found to be 2500 vph on a three lane approach, whereas that for a lag-lag type of phasing strategy was found to be 1800 vph, also on a three lane approach.

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I would also like to make a special mention of my supervisor, Mr. Srinivasa Sunkari, for his continued support all through the duration of my stay at the Texas Transportation Institute. He helped immensely by giving me access to licensed copies of CORSIM and Synchro without which my thesis would not have been possible.

I am also extremely thankful to Dr. Bonneson for having provided video data which was used for the development of the queue discharge model.

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CHAPTER I

INTRODUCTION

Traffic at signalized intersections exhibits complex vehicle interactions, which tends to have a significant impact on traffic operations. Frustration at signalized intersections or "stop light" as it is commonly called, often runs high. It happens so often that a driver is near the stop bar after waiting in the queue for a whole cycle and the signal indication turns red again. The infuriated driver ends up waiting for one more cycle. Ideally, this motorist should have been able to cross the intersection but for the start up loss. However, in real time queue discharge, braking, stopping, stalling and reaccelerating induces additional loss of green time. It therefore becomes extremely important to study the behavior of queue discharge at signalized intersections in order to quantify the inefficiencies in signal operations.

BACKGROUND

Queue discharge plays an important role in signal operation. At the onset of green indication, the first few vehicles experience start up loss time due to which the queue dispersion takes place at less than saturation flow rate headway (*1*). Lesser the start up loss time and time to accelerate to final speed, lesser would be the time required to clear the queue. Each vehicle in the queue needs some time to react to the onset of green. This is termed as the response time. According to Akcelik and Besley (*2*), a response time equal to 1.17 seconds is associated with every vehicle in the queue. Although this time is required for safety reasons, one cannot ignore the loss of green time because of it.

Let us consider a rough but simple example for more clarity. For a queue consisting of ten vehicles, a total of 10 x 1.17 = 11.7 seconds would be used by the vehicles only to

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This thesis follows the style of *Transportation Research Record.*

respond. This means that for a green interval of 35 seconds, approximately 33-percent of the time there are some vehicles that are stationary even though they all receive the green indication. In addition, some time is also used by vehicles to accelerate to attain the final speed. This implies that a portion of the green interval is wasted thereby creating the need for longer cycle lengths. In a coordinated signal system however, a platoon approaching a downstream intersection gets the green in such a manner that the start up loss time and the acceleration delay is greatly reduced or even eliminated totally. For example, consider a vehicle traveling at a constant speed of 25 mph and at a distance (D) equal to the distance if it were the 10^{th} vehicle in a stationary queue. For this, the following assumptions have been made:

- Average space clearance = 5 feet
- Average vehicle length = 20 feet

D = 200 + (9*5) = 245 feet

Time required by the subject vehicle to reach the stop bar, t = 245/ (25 * 1.47) = 6.7 seconds. The difference therefore is quite large. In the latter case, the last vehicle reaches the stop bar in less than 7 seconds while in the first the vehicle is around 5 seconds from mobilization. Assuming an acceleration of 10ft/sec^2, after starting up, the 10^{th} vehicle in the queue would take 3.7 sec to reach the speed of 25mph and take another 4.8 sec to reach the stop bar. This is to say the time difference could be as high as 5 + (3.7+4.8) - 6.7=6.8 sec.

Let's consider another scenario by looking at the first vehicle in the queue. If the vehicle starts after the onset of the green, it will take 1.17 sec (assumed) for the initial response, and 25*1.47/10=3.7 sec to reach the speed of 25 mph. After 1.17+3.7 ≈ 5 sec, the vehicle will be $0.5*10*3.7^2$=68 ft downstream of the stop bar. If the vehicle were passing through the intersection at a constant speed of 25 mph, it would have taken it only 68 / (25*1.47) = 1.9 sec to reach the same location. That is a difference of more than 3 seconds.

It is to be noted that queue discharge at signalized intersections cannot be simply explained by the fundamental equations of motion used above as it is a complicated phenomenon to model. A large variety of vehicles take up randomized positions in the queue thereby making modeling a difficult task. In order to determine the discharge time of a vehicle from a particular position in a queue at an intersection approach, many factors need to be looked at and so far, there is not a commonly agreed upon model that describes individual vehicle's discharge characteristics.

Those two simple examples roughly show that the time delay due to start-up is considerable, especially for high volume situations where long queues are present before the onset of green The strategy proposed in this paper tries to exploit this property of queue dispersion to optimize traffic operations at isolated signalized intersection by setting up a pre-signal (*3*) signal upstream of the main intersection. Although setting up such a signal would violate the MUTCD warrants, this paper wishes to impress upon the fact that the benefits obtained could be used to justify the need for such a signal and also a special warrant for such a system. This warrant could well be an extension of Warrant 6 of MUTCD, which states that, "the need for a traffic signal shall be considered if an engineering study finds that one of the following criteria is met:

1. On a one-way street or a street that has traffic predominantly in one direction; the adjacent traffic control signals are so far apart that they do not provide the necessary degree of vehicular platooning.
2. On a two-way street, adjacent traffic control signals do not provide the necessary degree of platooning and the proposed and adjacent traffic control signals will collectively provide a progressive operation" (*4*).

CHAPTER II

SYSTEM DESCRIPTION

A Main and Pre-signal signal system would essentially consist of a pre-signal placed at an appropriate distance upstream of the actual intersection, and a signal located at the actual intersection. The pre-signal would serve as the primary signal while the signal at the intersection will serve the purpose of not allowing conflicting movements to enter the intersection simultaneously. All the vehicles will have to stop at the pre-signal during red indication. The objective here would be to keep the offset section free of any queue formation as far as possible. It is expected that occasionally a few vehicles may be left stranded in the offset section which would reduce the pre-signal effect. However, the system would still be able to cut down on the start up loss time and the response time to a major extent. The main and pre-signal will have an offset between them so as to avoid the stragglers from being left stranded in the offset section. Preferably, the offset would also be such that the leading vehicles do not have to slow down as they approach the intersection on observing a red indication. The offset would in a way "absorb" all or most of the loss time depending upon the offset distance. The process of start-up and accelerating at the onset of green indication would take place while a conflicting phase is being served. As a result, at the main signal, the cycle length will be much less than what would otherwise be required. In other words, travel time and control delay should come down significantly.

The system will resemble a coordinated network of signalized intersections. Each pre-signal signal will have green, yellow and real red. The pre-signal signal would not have an all red as only one movement will be served at the pre-signal signal. The onset of red at pre-signal signal would overlap with the onset of green at the pre-signal signal of the subsequent phase. The main signal however will have the complete clearance interval. Even under the worst possible combination, the last vehicle departing from the pre-signal stop bar of one approach will be much ahead of the first vehicle of the conflicting phase. Assuming that the last vehicle to cross the stop bar is traveling at a speed of 25 mph and that the offset distance is 200 feet. The average time required by this vehicle to reach the stop bar would be 5.5 sec. On the other hand, the vehicle form the conflicting phase would need 1.17 sec to respond to the onset of green. After that, assuming that the vehicle takes off with an acceleration of 10 ft/sec^2 to cover the offset distance of 200 feet, the time required to reach the stop bar would be, 6.34 seconds + 1.17 seconds of start up loss time which is equal to approximately 7.50 seconds. Therefore, the former vehicle has a head-start of approximately 2 seconds over the latter. In these 2 seconds, the former would be able to cover at least a distance of 70 feet which is almost the other end of an intersection with 3 lanes in either direction.

CHAPTER III

PROBLEM STATEMENT

The performance of the proposed system is expected to depend heavily on the queue discharge phenomenon existing at signalized intersections. It was therefore felt that simulation models like CORSIM (*5*) should be used to evaluate the system under various conditions.

Although simulation was a natural choice for evaluation, the need for evaluating the benefits using a queue discharge model was also felt. However, an extensive review of literature suggested that the existing queue discharge models were inadequate to achieve the objective of this research study.

To evaluate the proposed system precise information on the time taken by the n^{th} vehicle in queue (with reference to the start of green interval), to cross the stop bar was required. The existing models found during the literature review suffered from the following deficiencies:

1. These models were developed in other countries and hence their applicability in the US cannot be achieved without inducing unnecessary errors in the process.
2. The models were primarily described in terms of velocity or saturation flow rate. To calculate the time required to cross the stop bar, knowledge on the acceleration profile was also needed thereby making them difficult to solve and calibrate.

It was therefore felt that a simpler model based on data collected in the US would be more suitable for the evaluation of the proposed system. This significant portion of this research therefore deals with the development and validation of a new queue discharge model.

CHAPTER IV

OBJECTIVES

The primary objective of this research study was to evaluate the performance of the proposed system by comparing various pre-signal configurations with the baseline case, i.e. the "do-nothing" option. The simulation runs were designed to investigate the relative improvements obtained by installing pre-signals on two, three and all four approaches to finally give suggestions for the best offset distance to be used. A queue discharge model was also developed for evaluating the theoretical benefits of the new system. Video data recorded in Miami, Florida was used to develop a multiple linear regression model. This model was then used to validate the system using hypothetical values for the independent variables in the model. The independent variables which are expected to have a significant impact on the queue discharge time are:

- Position of the vehicle in the queue.
- Type of vehicle, viz. car, truck etc.
- Turning movement.
- Percentage of heavy vehicles.
- Width of lanes.
- Flow rate.

Although speed would also play an important part in the model, the information on speed limits was not available and hence this parameter had to be left out. The other reason why speed limit could be omitted is that most vehicles would tend to cross the stop bar before they attain the final speed. As a result, a change in speed limit would not really affect the queue discharge time by much especially for the first few vehicles.

CHAPTER V

LITERATURE REVIEW

Queue discharge characteristics have been extensively researched over the years. However, the most common variable which is considered in most of these studies is the saturation flow rate. Although it can also give us the headways, but without accounting for various influencing factors like lane width, proportion of heavy vehicles etc., such an estimate could invariably be inaccurate.

Mahalel et al (*6*) conducted an experiment by studying the discharge rate at two intersections in Israel which had long cycle lengths (263 to 351 seconds). They found that the discharge rate attained the maximum value, (i.e. saturation flow rate) after 50 to 60 seconds. This finding greatly differs from numerous other studies (*7, 8*) wherein the corresponding times were found to be 6 to 10 seconds from the onset of green. In another study, in Poland (*9*), the time to attain saturation flow was found to be around 25 seconds while the US Highway Capacity Manual indicates that the queue tends to stabilize after 10 to 14 seconds of green (*10*).

It is evident that there is no consistency in what various studies have found. Although one can determine headway from saturation flow rate, it is important to note that this procedure tends to neglect numerous factors which may also influence the headway distribution.

Akcelik et al (*11*) conducted a study to evaluate various aspects of queue discharge at signalized intersections. The following data was collected in their study with the help of two Video Data Acquisition System (VDAS) detectors spaced approximately 3m apart:

1. Queue discharge headway and speeds of individual vehicles crossing the VDAS detector.
2. The times when the third through seventh vehicles cross the stop bar. This was done manually with the operator manually pressing a handheld button connected

to a laptop.

3. The time at which the last vehicle from the back of the queue crosses the detector. This again was done manually by an operator with a handheld button connected to the same laptop. The identification of the last vehicle in queue under oversaturated conditions was not done manually.
4. The queue length in terms of number of vehicles at a given distance from the stop bar.
5. Times associated with changes in signal indications recorded automatically using light sensors linked to a second VDAS unit.
6. Distance to the back of the downstream queue recorded using a laser gun activated at the start of the green period at the upstream signal and at various stages of the green interval.

The exponential queue discharge model was found to be the most accurate one and was used for the research. The speed models were expressed as a function of time since start of green and not as a function of vehicle position in queue. The exponential speed and flow rate models are described below:

$$v_s = v_n (1 - e^{-mv(t - tr)}) \quad (1)$$

$$q_s = q_n (1 - e^{-mq (t - tr)}) \quad (2)$$

Where,

t = time since the start of the displayed green period (seconds)

t_r = start response time related to an average driver response time for the first vehicle to start at the start of the displayed green period. (seconds)

v_s = queue discharge speed at time t (km/h).

v_n = maximum queue discharge speed (km/h)

m_v = a parameter in the queue discharge speed model.

q_s = queue discharge flow rate at time t (veh/h)

q_n = maximum queue discharge flow rate (veh/h) and

m_q = a parameter in the queue discharge flow rate model.

It was found that the queue discharge headways (h_s) tend to decrease with the passage of time from the start of green, eventually stabilizing and becoming equal at the two VDAS detectors.

Akcelik et al (*2*) developed a relation between parameters m_q and m_v by applying the boundary condition of $V_s = 0$ and for $L_{hs} = L_{hj}$. Equation 3.0 below provides the relationship between the aforementioned parameters.

$$m_q = m_v\ L_{hn}/L_{hj} \tag{3}$$

where,

L_{hj} = Average jam spacing (m/veh)

L_{hn} = Average spacing at minimum queue discharge headway (m/veh)

Queue departure response times (t_x) were also studied. The study yielded the following equation for calculating queue departure response time.

$$t_x = h_s - 3.6L_{hj}/V_s \tag{4}$$

where,

t_x = queue departure response time sec

h_s = saturation headway (seconds)

V_s = saturation speed (km/h)

The field studies carried out by Akcelik et al (*2*) suggested that for through traffic at isolated intersection, the queue departure response time was 1.17 seconds while Vs was found to be 45.1 km/h. This finding seems to be in contrast with previous findings which state that the response time tends to decrease with passage of time from the onset of green.

A few other parameters studied were start up loss time t_s, departure response time t_r, average acceleration delay d_a and queue clearance wave speed V_x.
The equations for d_a and v_x are given below in equations 5 and 6 respectively.

$$d_a \text{ (seconds)} = t_s + h_s - t_r = t_s + h_s - t_x = t_s + h_s - t_r = t_s + 3.6L_{hj}/V_s \qquad (5)$$

$$v_x \text{ (km/h)} = 3.6L_{hj}/\ t_x = 3.6L_{hj} / (t_s + 3.6L_{hj}/V_s) \qquad (6)$$

The most positive aspect of these queue discharge models is that it brings in a new dimension to traffic modeling instead of simpler models based only on saturation flow and effective green time (*2*).

Modeling queue discharge becomes extremely complex for heterogeneous traffic composition. The variation in acceleration rates of the various vehicle types tends to have an adverse impact on the accelerating capabilities of vehicles capable of moving faster. A simple example would be that of a slow-moving logging truck waiting at a stop bar. A car waiting behind this logging truck would be unable to accelerate as fast as it would like to. Such instances tend to reduce the capacity of signalized intersections significantly. Data collection for such traffic compositions can be tricky. Use of loop detectors could yield erroneous data on account of not being able to detect the difference between the various types of vehicles. Under such circumstances video data collection is possibly the most accurate data collection method. Maini et al (*12*) used full motion video recording to collect data in two cities in India, namely, Baroda and New Delhi. Video data also provides the option of analyzing special events like a vehicle impeding traffic in a lane for some reason (*12*). A procedure used by Huber and Tracy in 1968 was used to convert screen coordinates to roadway coordinates. The coordinates for four points with no three points being in a straight line were identified for this coordinate conversion. The dynamic characteristics such as acceleration rates, deceleration rates, the speed at which deceleration starts and the speed achieved on accelerating from the intersection were determined at one second intervals. The main focus of the study was queue clearance speed at one intersection in Baroda and two intersections in New Delhi.

The study suggested that the queue discharge was at a uniform rate and the presence of bicycles in the traffic stream did not affect queue discharge. It was found that two-wheelers and cars have 33 & 66% higher acceleration rates than those of auto-rickshaws (three-wheeled taxis) and buses respectively. Another interesting finding of this study was that clearing speeds attain a constant value much early than in homogeneous composition. These findings are interesting in that these need to be considered while devising signal timing plans.

Lin et. al. (*13*) came up with an interesting model which unlike some of the other studies looked at the time variable rather than the saturation flow rate. Their model (from regression analysis) estimated the number of vehicles that can be discharged during the usable portion of the green interval.

$$N_{gi} = -1.37 + 0.464G + 0.00091G^2 \quad (7)$$

where,

N_{gi} = expected number of vehicles that can be discharged during the green interval of usable phase i.

G = Green Interval (sec)

Although their model (Eq. 7) has an average error, it can be said that it has the following limitations:

- The authors say that the data used to develop this model comes form different countries and from traffic lanes that differ in geometric design and traffic conditions. The applicability of the model in the US thus may be questionable.

- The model seems to have missed out on other important factors such as flow rate, lane width, which may have a significant impact on the phenomenon of queue discharge.
- The model gives negative values for N_{gi} when G is less than 3 seconds. This further limits the model's applicability in that it cannot really be used to determine the time required by a queued vehicle to cross the stop bar.

The system being evaluated for this study depends heavily on the queue discharge characteristics and there was a need to use a queue discharge model which could make it possible to compare the travel time savings between the base case and the proposed system. The models discussed previously may not be suitable for various reasons and hence it was felt that investigation should be carried out into a more close to home queue discharge model which can predict the time required by a queued vehicle to cross the stop bar.

The actual interest in developing this model lies in using the model to evaluate the proposed system, described later in the report. Moreover, another reason for studying this area in detail was the fact that the presence of a new queue discharge model would prove useful in the development of micro-simulation models.

CHAPTER VI

STUDY DESIGN

CORSIM was used to simulate a Main and Pre-signal system for this study. However, as CORSIM does not allow users to provide signal head in only one direction, the other direction was also provided a signal head and was so timed as to provide perpetual green to traffic moving away from the intersection. The offset distance between the Main and pre-signal enabled the introduction of a one second overlap between phases such that the start of green of one phase would overlap with the first second of red of the previous (conflicting) phase. This one second value was calculated assuming that the first vehicle departing from the pre-signal stop bar starts off with an acceleration of 15 ft/s^2 to attain final speed. It is important to note that this acceleration rate is high, resulting in conservative benefits. Therefore, the following calculations are on the conservative side. The vehicle departure profile would consist of two parts, viz.,

1. constant acceleration to attain final speed:
   $u = 0$ ft/s, $v = 40$ mph = 58.8 ft/s
   Distance to achieve final speed, $S_1 = (v^2 - u^2)/ 2a = 115.25$ feet.
   Time to attain final speed, $t_1 = 1.17 + \sqrt{\frac{2S_1}{a}} = 5.09$ seconds
2. remaining distance at constant speed:
   Assuming an offset distance of 200 feet,
   Time to cover the remaining distance = (200 – 115.25) / 58.8 = 1.44 seconds
   An advance warning of 2 seconds would be required to prevent drivers from slowing down on observing the signal indication to be red.
   Therefore, the offset would be equal to 5.09 + 1.44 – 2 = 4.5 seconds.

It is also important to provide sufficient time for the last vehicle from the previous phase to cross the intersection before vehicles from a conflicting approach reach the intersection. This last vehicle would be traveling at its final speed of 58.8 ft/s and hence would need 200 / 58.8 = 3.40 seconds.

The additional green that can be provided = 4.5 – 3.4 = 1.1 seconds ≈ 1 second. Moreover, since the pre-signal will not be having the all red phase, that time can also be allocated to the green phase. Therefore, the total additional green time provided at the pre-signal was 2 seconds assuming a one-second all-red time.

METHODOLOGY

The research was carried out in two parts in pursuit of the objectives. The objective was to evaluate feasibility of the proposed system over the "do-nothing" option. This investigation was done using analytical analysis as well CORSIM simulation and the details of the same are presented below:

Analytical Evaluation

The analytical evaluation consisted of development of a Queue Discharge Model (QDM) which was used to evaluate and quantify the savings in green time expected from the use of pre-signals. The proposed model yields information on the time (T) that a queued vehicle (depending upon its position/rank in the queue, e.g.: first second, third etc.) requires to cross the stop bar, starting from the onset of green indication for the corresponding approach. The model has position in queue, flow rate for the study approach, vehicle type, movement type, and proportion of heavy vehicles in traffic, as the independent variables from which the dependent variable (T) would be determined. The model was developed from the video data collected in Florida by Dr. Jim Bonneson of the Texas Transportation Institute. Validation of the model was done using data collected locally in College Station, TX.

The model was used to estimate "T" for vehicles in queue (generated randomly using Excel) and these values of T were used for the baseline scenario (or "do-nothing" option). These values of "T" were then compared with a case where vehicles were moving at higher speeds and at distances equal to the corresponding vehicles in the base case. The difference in travel times across the stop bar for all corresponding vehicles in the two scenarios was calculated for estimation of savings in travel time.

In addition to quantifying travel time savings, HCM analysis was also used to calculate and compare control delay for the two cases. The standard delay terms mentioned in HCM were calculated for a hypothetical intersection with input parameters similar to those presented in the next section.

CORSIM Simulation Evaluation

PART I: Basic Scenarios

For this section, an isolated intersection with four approaches having equal volume levels and similar turning movement percentages was coded in Synchro. The network was optimized in Synchro to obtain the signal timing plan. The optimized timing plan was then used in CORSIM for evaluation.

The evaluation of the system was done in a manner so as to account for most of the constraints that such a system could face. In order to do that, the following configurations were coded and subsequently used for evaluation:

TABLE 6.1 Basic Scenarios Evaluated

Volume (vph)	Speed Limit (mph)	Offsets (feet)
2000	30	200, 250, 300, 350, 400 & 450
2000	40	200, 250, 300, 350, 400 & 450
2000	50	200, 250, 300, 350, 400 & 450
2500	30	200, 250, 300, 350, 400 & 450
2500	40	200, 250, 300, 350, 400 & 450
2500	50	200, 250, 300, 350, 400 & 450
3000	30	200, 250, 300, 350, 400 & 450
3000	40	200, 250, 300, 350, 400 & 450

A. *Pre-signals on all four approaches*: Such a configuration was evaluated to study the performance of pre-signal equipped isolated intersections. Table 6.1 presents the scenarios that were created in CORSIM for evaluation.

B. *Pre-signals on three approaches:* This configuration was generated to account for the possibility of installing the system at intersections where one of the approaches may have a nearby upstream intersection. All the scenarios listed in Table 6.1 were again used with this configuration for evaluation.

C. *Pre-signal on two approaches:* This configuration was generated to account for the possibility of installing the system at intersections where two of the approaches may have a nearby upstream intersection. The scenarios from were used but with a small modification to the timing plan. For this configuration, the additional green time was allocated to the approaches without pre-signals, which is in contrast to the previous configuration where the additional green time was provided for the approaches with pre-signal.

D. *Pre-signal on one approach:* This configuration was generated to account for the possibility of installing the system at intersections where three of the approaches may have a nearby upstream intersection.

The offset distances for each of these configurations was varied from 200 feet to 450 feet in equal increments of 50 feet to determine the most suitable offset distance for the given set of conditions. Greater offset distances were intentionally left out as one would expect platoon dispersion to take a toll of the system's benefits. Moreover, a sensitivity analysis was carried out on speed limit to evaluate the impact of speed limit on the system's effectiveness. The speed limits used were 30 mph, 40 mph and 50 mph.

PART II: General Scenarios

The basic scenarios appear intrinsically similar to each other owing to the similarity of the input parameters. However, the intent of evaluating such scenarios was to determine if each of these configurations are able to yield significant benefits to warrant their installation.

Therefore another set of scenarios were coded for further evaluation. This set of simulations involved real world like scenarios with varying proportions of turning movements, truck volumes, total input volumes, speed limits, offset, number of lanes and a lag-lag type phasing sequence. It is important to note that unlike for the basic scenarios, this section involved evaluating various offset times between the main and pre-signal., Table 6.2, Table 6.3, Table 6.4, and Table 6.5 present a matrix describing all the proposed simulation scenarios to further evaluate the system under real life like conditions. It is important to be noted that all approaches had two exclusive through lanes, one exclusive left lane and on shared through-right lane.

TABLE 6.2 General Simulation Scenario: Case 1

VARIABLES	EB Approach			WB Approach			NB Approach			SB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	1565			2450			3120			1200		
Turning Movements (%)	6	92	2	8	84	8	5	94	1	10	82	8
Proportion of Heavy Vehicles (%)	5			10			15			20		
Speed Limit (mph)	30			35			40			50		
Offset (sec)	10			12			14			16		

TABLE 6.3 General Simulation Scenario: Case 2

VARIABLES	EB Approach			WB Approach			NB Approach			SB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	3115			1212			2568			1880		
Turning Movements (%)	14	82	4	6	92	2	13	77	10	9	88	3
Proportion of Heavy Vehicles (%)	5			35			20			15		
Speed Limit (mph)	50			40			35			30		
Offset (sec)	11			13			15			17		

TABLE 6.4 General Simulation Scenario: Case 3

VARIABLES	EB Approach			WB Approach			NB Approach			SB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	3118			1831			1540			2844		
Turning Movements (%)	10	87	3	7	88	5	10	85	5	9	86	5
Proportion of Heavy Vehicles (%)	8			22			11			17		
Speed Limit (mph)	40			35			40			50		
Offset (sec)	17			15			11			13		

TABLE 6.5 General Simulation Scenario: Case 4

VARIABLES	EB Approach			WB Approach			NB Approach			SB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	2500			1522			1710			3450		
Turning Movements (%)	12	85	3	7	90	3	9	87	4	11	81	8
Proportion of Heavy Vehicles (%)	9			30			21			14		
Speed Limit (mph)	40			35			30			50		
Offset (sec)	16			14			20			8		

It is to be noted that a lead-lead type signal phase was not coded as CORSIM does not provide the flexibility for the user to code a lead-lead phasing for the pre-signal because the waiting motorists at the pre-signal cannot be “pre-informed” about their movement at the dummy signal. A lag-lag type of phasing does bring into question the issue of yellow trap also. However, as the issue is related to the phasing strategy and not the system in question, it was excluded from the scope of this research.

Another practical concern which deserves some mention is the fact that while evaluating the system, this study concentrated more on offset distances rather than offset times. Using offset time as a way of evaluating is also a possibility, however, that would mean extensive fine tuning of offset distances to achieve perfect co-ordination. Since this exercise is time consuming and it was felt that the same results can be obtained by varying the offset distances, too much stress was not given on sensitivity analysis of offset times.

CHAPTER VII

DATA COLLECTION

Video data collected at three intersections in Miami, Florida were used to develop a model to predict the time taken by the n^{th} vehicle in queue to cross the stop bar. Since the back of the queue was not visible for the approach under study, queues in the opposing direction were observed to estimate the length of queue on the subject approach. The time to clear the stop bar was noted using a stop watch with the ability to split the time every time a vehicle crosses the stop bar.

The data set consisted of approaches with volumes of 1380 vph, 1707 vph, 1730 vph, 1735 vph and 1969 vph and heavy vehicle percentages of 2.3.2.6, 2.7, 2.9 and 6.3 %. Moreover, all approaches studied had three through lanes. In addition to volume and vehicle type, other factors which were considered were:

- Turning movement.
- Lane width.
- Vehicle position in a queue.

In addition to the data collected from the video tapes, some more data was collected in College Station at the intersection of University drive and College Main Street for validation of the model developed from the video data.

CHAPTER VIII

QUEUE DISCHARGE MODEL DEVELOPMENT

APPENDIX I presents some of the data that was used for data analysis. The variables, vehicle type (VT) and Movement Type (M) were expressed using dummy values. Vehicle Type 1 represents cars/SUVs, Vehicle type 2 represents trucks and vehicle type 3 represents buses and as far as movement type is concerned, through movements were represented by 0 while right turning movements were represented using 1.

Numerous combinations were tried in order to be able to fit a suitable curve. Each combination that was tried is listed below:

- Linear Model with all of the aforementioned variables.
- A two-regime model with the first regime describing the discharge phenomenon for the first 4 vehicles in queue and the second regime for the remainder vehicles in the queue. The first regime was expected to fit a non-linear curve and therefore, numerous combinations like, negative exponential, exponential, natural logs and logs to the base of 10 were tried. Moreover, in addition to the dependent variable T, Ts was also used. Ts can be defined as the time taken by a vehicle to cross the stop bar with the reference point in time being the crossing of leading vehicle. In simpler words, if the first vehicle takes 3 seconds and second vehicle takes 5 seconds, Ts values for the first and second vehicles would be 3 sec (3-0 seconds) and 2 sec (5 – 3 seconds) respectively. The second regime however, was expected to have a linear fit and hence no non-linear fits were tried.
- A quadratic model.
- A model in which, instead of T, R was used as the dependent variable and all combinations were tried again, viz., exponential, logarithmic & quadratic.

- A two-regime linear model. This model did not fare much better than the single regime linear model and hence was eliminated.

MS Excel was used to run multiple linear multiple regression analysis for all the expected fits described above. However, it was found that the linear model described the data much better than the non-linear models. The R-square value for the linear model was found to be 0.92 while that for the non-linear models varied between 0.04 and 0.091. This clearly indicates that the data follows a linear fit. As a double check, a scatter plot with position of vehicle (R) on the X-axis and Time (T) on the Y-axis was produced. Various forms of trend line were tried on the plot and that again proved that the data can be best described as being linear. It can be seen from Figure 8.1 through Figure 8.3 that the best possible fit can only be a linear line. It should be noted that only one independent variable, namely, R was used here since it is easily the most important one amongst others. Also, vehicle positions beyond the 10^{th} were deleted because limited data was available for such vehicles due to practical restrictions.

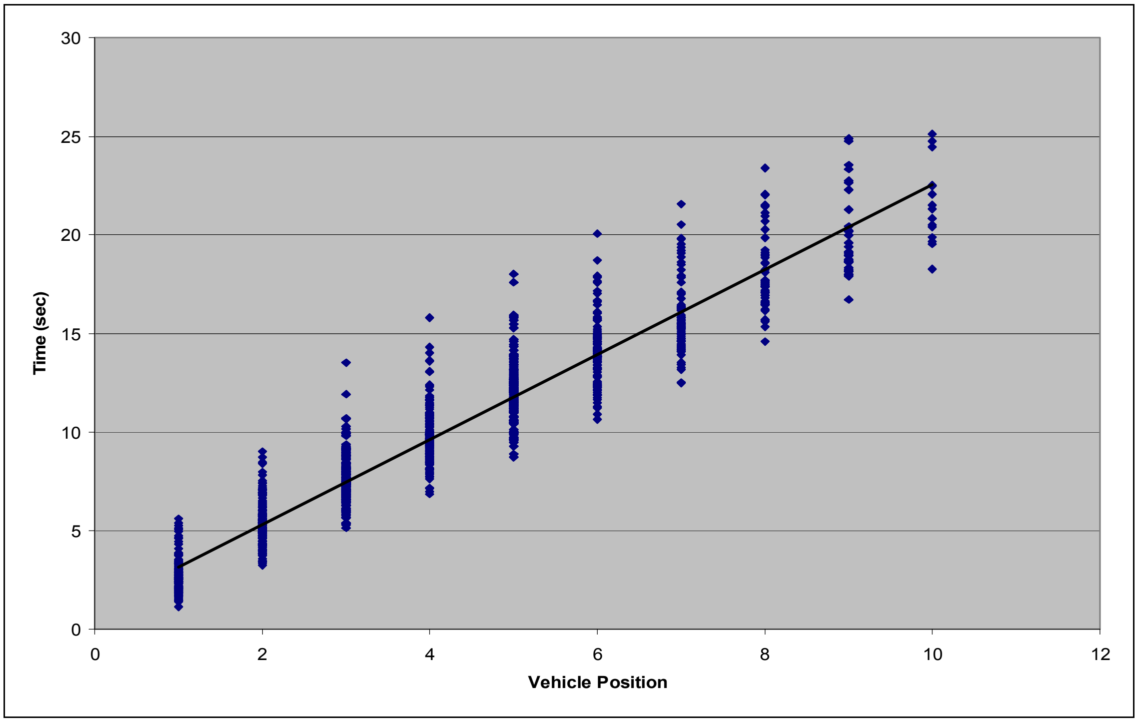

FIGURE 8.1 A Linear Trend Line Fitting the Data

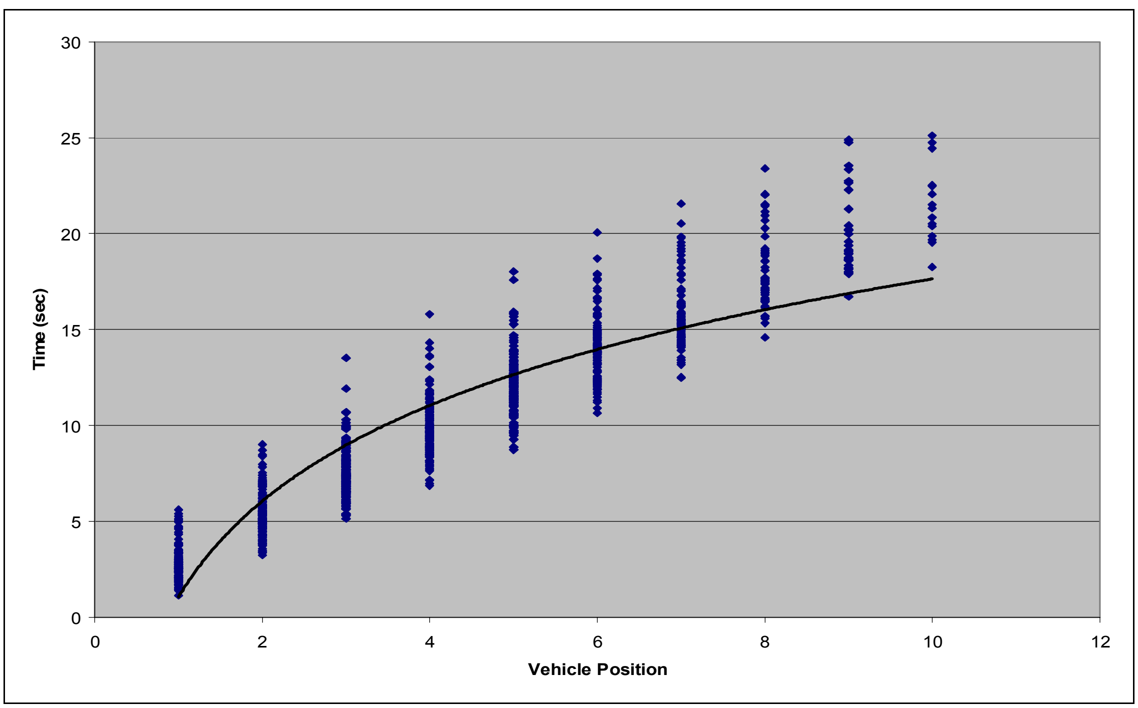

FIGURE 8.2 A Logarithmic Line Fitting the Data

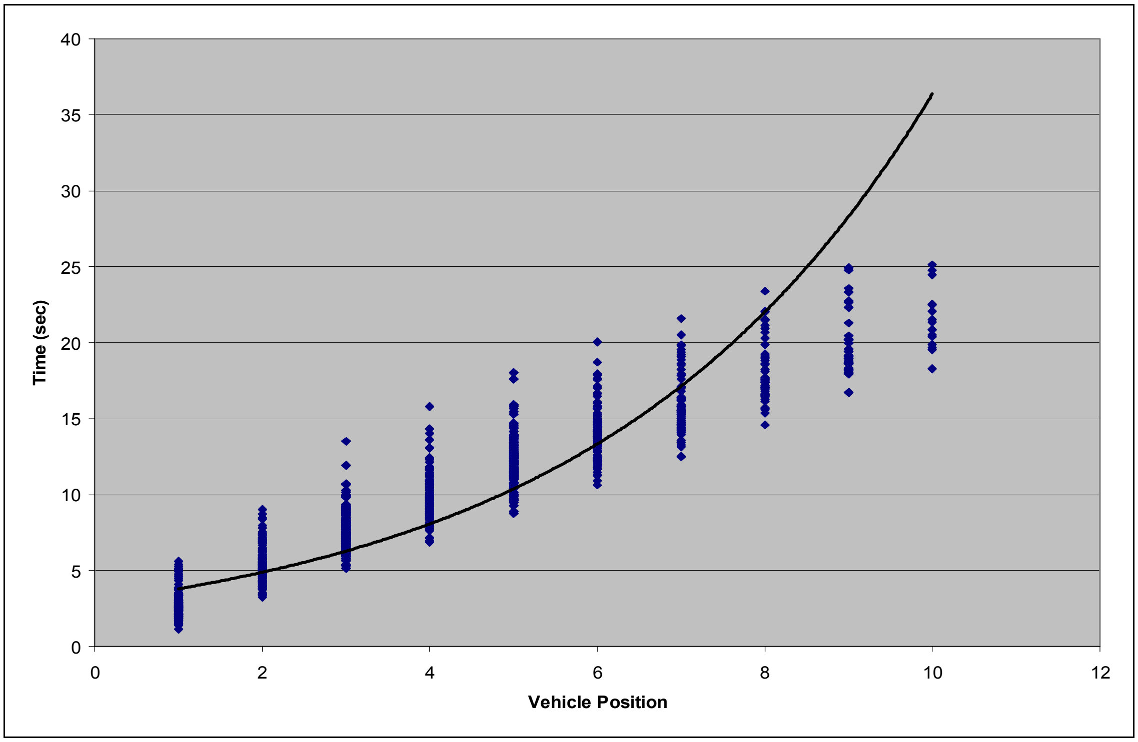

FIGURE 8.3 An Exponential Line Fitting the Data

The linear model which was eventually selected has been presented below:

$$T = 5.56+2.15R + 0.68M + 0.302Phv - 0.46W - 0.00022F + 0.27VT \quad (8)$$

where,

R= position/rank of the vehicle in queue. First vehicle will be ranked 1 and so on.

M= Dummy variable for movement type. Thru = 0 and Right = 1.

Phv = Proportion of Heavy Vehicles in percentage.

W = Width of Lane in feet.

F = Flow Rate in vph

VT = Dummy variable for vehicle type. Dummy value for Car, Bus and Truck were 1, 2 and 3 respectively.

It was found that the co-efficient for flow rate was not significantly different from 0 at

95 % confidence level because its p-value was found to be less than 0.05. This can be seen in Table 8.1. Usually under such circumstances, the variable is dropped out of the model and a reduced model is developed. However, in this case, it was felt that flow rate should feature in a queue discharge model according to traffic engineering judgments.

TABLE 8.1 Significance Testing for the Model

	Coefficients	Standard Error	t Stat	P-value
Intercept	5.55580	1.79	3.11	0.00
Vehicle Position	2.15160	0.02	114.82	0.00
Mvmt	0.67836	0.21	3.16	0.00
Phv	0.30208	0.13	2.38	0.02
Lane width	-0.46461	0.12	-3.88	0.00
Flow rate	-0.00022	0.00	-0.72	0.47
Vehicle Type	0.27079	0.05	4.96	0.00

The final check for this model's credibility was examination of the scatter and normal probability plots. It can be seen from Figure 8.4 that the residuals seem to have an average of 0 and no clear trend can be seen. It should be noted that lack of sufficient data restricts the model's use to only around 9 or 10 first vehicles in the queue. The sparsely spaced residual for the 9^{th} vehicle is a testimony to that. After studying the normal probability and scatter plot Figure 8.5, it can be said that the residuals are approximately normally distributed with a mean of 0. This augurs well for the model as it establishes its credibility in predicting the time required by a vehicle to cross the stop bar; from the onset of green. Although statistically the model seems to be sound, it also needs to checked for its ability to predict T that come close to observation made in real life. In other words, the model needs to be tested using real data.

A two-regime curve fit was also tried using numerous combinations. However, all the non-linear models were found to be describing the data points weakly. The linear two-regime model however came close but not as close as the single regime linear model. The R-square value for the two-regime linear model was approximately 0.90, a little less than that for the single regime model. Moreover, the scatter plot and the normal probability plot also looked much similar to those of the single regime model. In addition, Figure 8.1 clearly shows that even for the first 4 vehicles in queue, the relationship is linear contrary to prior expectations of it being non-linear in nature. It was therefore felt that the single regime model would be a more convenient model to use for the purpose of this research.

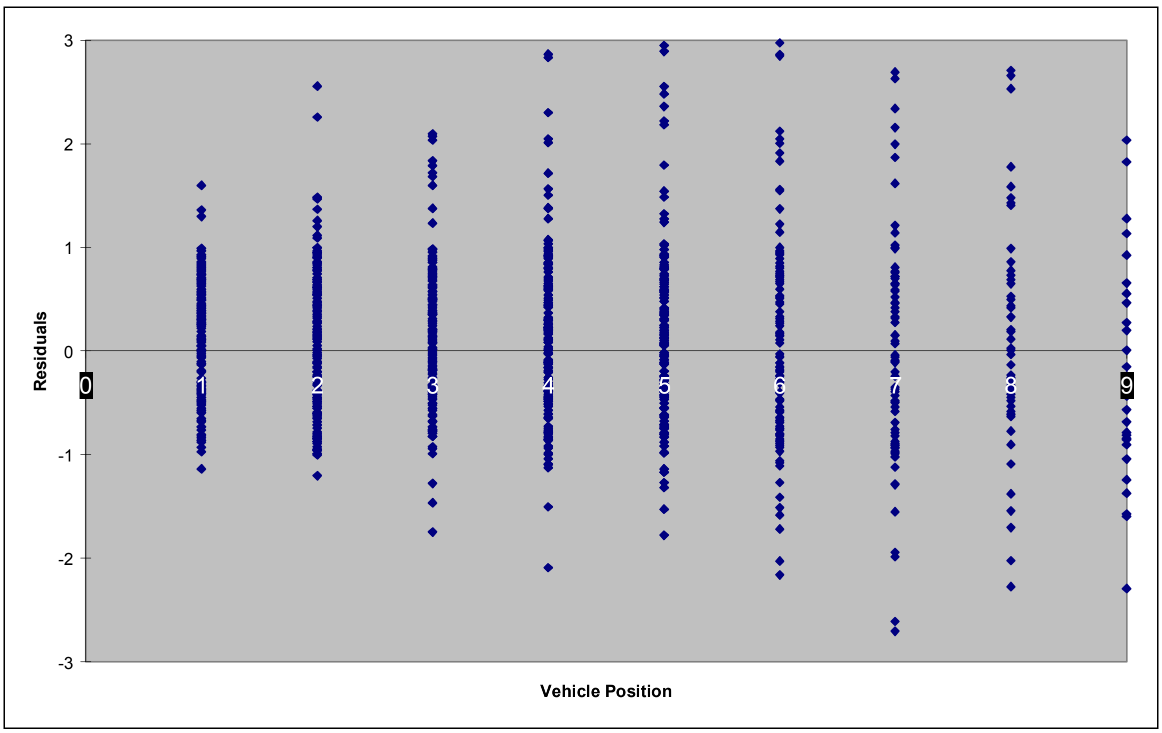

FIGURE 8.4 Residual Scatter Plot

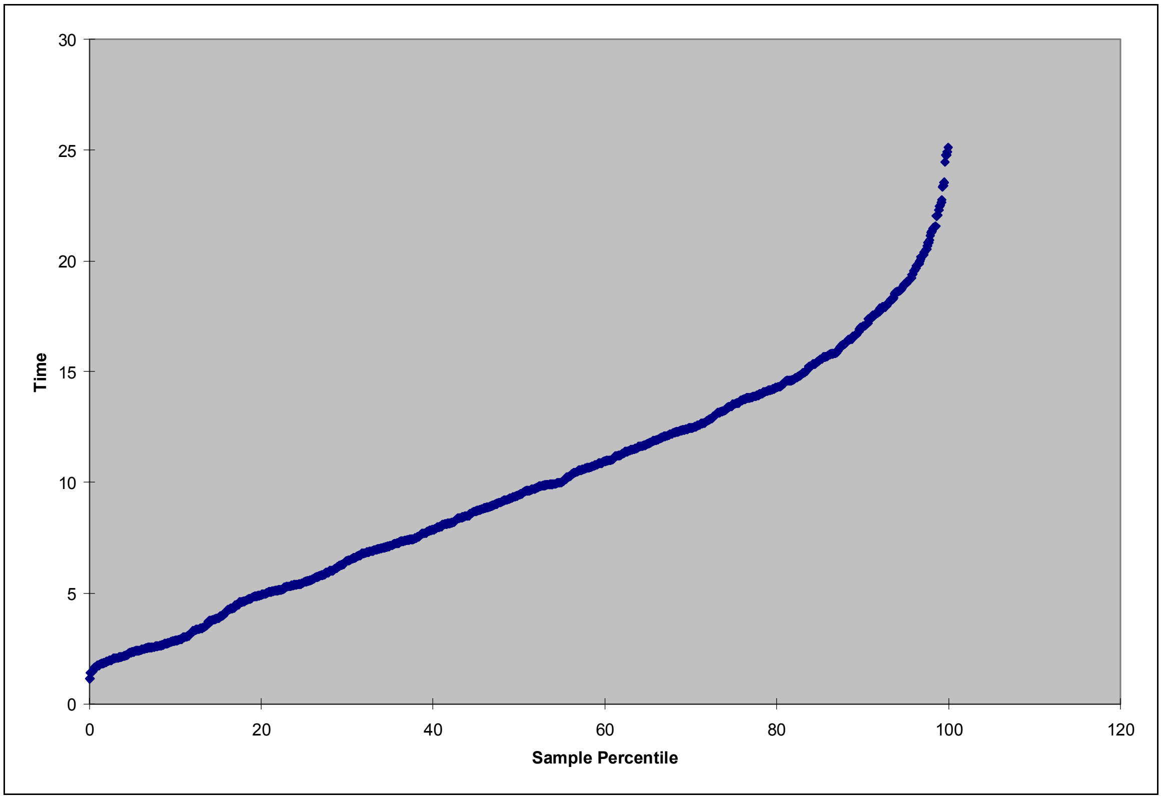

FIGURE 8.5 Normal Probability Plot

MODEL VALIDATION

As mentioned previously, real time data was collected in College Station at the intersection of College Main street and University drive. A total of 20 cycles were studied and time, 'T' for the EB vehicles noted. Appendix III presents a few sample tables where the observed and predicted values were compared and the error for each vehicle calculated. The errors over all the 20 cycles were then used to conduct a one sample t-test. The null and alternate hypothesis of the test is presented below:

Ho: Mean Error, $u = 0$

Ho: Mean Error, $u \neq 0$

$\alpha = 0.05$

The demo version of statistical software Minitab 14 was used to conduct the 1 sample t-test, the results of which are presented in Table 8.2. The P-value in this case was found to be greater than 0.05 and hence we fail to reject the null hypothesis.

TABLE 8.2 One-Sample t-test Output

Variable	N	Mean	StDev	SE Mean	95% CI	t-stat	P-value
C1	170	0.079	0.654	0.050	(-0.0196, 0.178)	1.58	0.115

Having passed the validation test also, the model seems fit to be used for evaluating the theoretical benefits of the proposed system.

CHAPTER IX

EVALUATION OF THEORETICAL BENEFITS

TRAVEL TIME SAVINGS

The linear model was conceived with the intention of being able to quantify travel time savings generated by the system. A more thorough evaluation of the system was done using CORSIM and is discussed later in the report. The model was used to calculate T for the first 10 vehicles in queue on four approaches of an isolated intersection. The site characteristics can be seen in Table 6.3. Using Equation 1, Table 9.1 was generated in MS Excel.

TABLE 9.1 Time to Cross Stop Bar for Base Case

BASE CASE				
RANK	EB	WB	NB	SB
1	3.93	5.75	7.56	9.37
2	6.07	7.89	10.34	11.51
3	8.21	10.03	11.84	13.65
4	10.99	12.81	14.62	15.79
5	12.49	14.95	16.12	17.93
6	14.63	16.45	18.26	20.07
7	16.77	18.59	20.40	22.21
8	18.91	20.73	22.54	24.35
9	21.05	22.87	24.68	26.49
10	23.83	25.65	27.46	29.27

Now, the time required to cross the stop bar by the same set of vehicles ***but*** moving at an approximately constant speed of 35 mph due to the pre-signal, was calculated. The assumptions made to calculate the time were:

1. Average space headway between two non-stationary vehicles = 100 feet. This value was obtained by multiplying the saturation headway (2 seconds corresponding to a saturation flow rate of 1800 pcphpl) by assumed speed limit (35 mph). It should be noted that this is a conservative estimate and hence the results too are expected to be on the lower side.
2. Average vehicle length = 20 feet
3. Cycle length = 100 seconds.
4. Uniform flow.
5. Vehicle Occupancy = 1.2 persons per vehicle (*14*).
6. Value of travel time savings = $ 8 per hour (*15*).
7. The front bumper of the first vehicle in queue is in line with the stop bar.
8. A vehicle was said to have crossed the stop bar when the rear of vehicle crossed the stop bar.
9. A total of 4 peak hours exist in a day.

Based on these assumptions, the position of each vehicle in the queue was calculated using the formula:

Distance from stop bar for vehicle ranked n = (distance of vehicle n-1) + Average spacing (10 feet) + Length of the Vehicle (20 feet)

= Distance of $(n-1)^{th}$ vehicle + 30 feet.

For example, for the second vehicle in queue will be placed at 20 + 30 = 50 feet from the stop bar while the first vehicle will be at 20 feet from the stop bar. The calculation of travel time savings can be seen if Appendix IV. A description of how each column in Appendix IV was calculated is presented below:

$$\text{Time to clear stop bar (sec)} = \frac{\text{Distance from stop bar}}{\text{Speed (in mph) x 1.47}}$$

$$\text{Savings in travel time (hours)} = [\text{Time to cross stop bar (Base Case, Table 9.1)} - \text{Time top bar (Alternative Case)}] \text{ x } \frac{1}{3600}$$

Value of Travel Time Savings (VOTTS) per day per vehicle ($) = Average Vehicle Occupancy (1.2 persons per car) **x** Average Value of Time ($ 8 per hour) x Savings in travel time (hours).

Daily Total Travel Time Savings = {[(Average (EB-VOTTS) x (Flow rate)] + [(Average (WB-VOTTS) x (Flow rate)] + [(Average (NB-VOTTS) x (Flow rate)] + [(Average (SB-VOTTS) x (Flow rate)]} x Number of cycles per hour (= 36) x Number of peak hour operations (=4)}

Therefore,

Daily Total Travel Time Savings ($ per day) = $ 325

The theoretical analysis suggests that the system can induce travel time savings worth $ 200 (approximately) a day which certainly is a significant number considering that setting up a signal costs anywhere between $100,000 and $ 200,000. However, this analysis was based on numerous assumptions and hence a conclusive statement can be made only on the basis of simulation results.

REDUCTION IN CONTROL DELAY USING HCM ANALYSIS

As mentioned in the “Methodology” section, Total Control Delay was the other MOE which had to be evaluated. It was therefore necessary to carry out a HCM capacity analysis with the intention of comparing control delay for the base case vis-à-vis the alternate case. In order to be able to study the impact of demand level, two sets of cases were considered.

1. With approach volumes of 500 vph, 1000 vph, 1500 vph and 2000 vph, and
2. With approach volumes of 2500 vph, 3000 vph, 3500 vph, and 4000 vph.

The intersection was considered to serve only the through and right movements and hence only a two-phase signal timing plan had to be designed. Since this is only a preliminary analysis, a basic two-phase operation was considered.

Since the site characteristics remained same for both the cases, the calculation of actual saturation flow rate would be common. The calculations are shown in Table 9.2.

TABLE 9.2 Determination of Actual Saturation Flow Rate

Lane Group	Si	N	f_w	f_{HV}	f_g	F_a	f_{bb}	f_{LU}	f_p	f_{RT}	f_{LT}	f_{Rpb}	f_{Lpb}	S
EB	1900	3	1	1	1	1	1	0.91	1	1	1	1	1	5085
WB	1900	3	1	1	1	1	1	0.91	1	1	1	1	1	5085
NB	1900	3	1	1	1	1	1	0.91	1	1	1	1	1	5085
SB	1900	3	1	1	1	1	1	0.91	1	1	1	1	1	5085

Si = Ideal saturation flow rate.

N = Number of Lanes in the Lane Group.

f_w = Lane Width Adjustment Factor.

f_{HV} = Heavy Vehicle Adjustment Factor.

f_g = Grade Adjustement Factor.

f_a = Area Type Adjustement Factor.

f_{bb} = Local bus blockage adjustment Factor.

f_{LU} = Lane Adjustment Utilization Factor.

f_p = Adjustment for Parking Conditions.

f_{RT} = Right-turn Adjustment Factor.

f_{LT} = Left-turn Adjustment Factor.

f_{Rpb} = Adjustment Factor for ped/bicycle interference with Right-turns.

f_{Lpb} = Adjustment Factor for ped/bicycle interference with Left-turns.

S = Actual Saturation Flow rate under existing conditions.

The minimum delay cycle length was calculated using the Webster's equation (*16*) while the delay terms were calculated using the standard equations for Uniform and Incremental delay (*16*). Delay due to pre-existing queue was assumed to be nil. The total control delay for each approach is shown in Table 9.3.

$$Co = \frac{1.5L+5}{1-\sum(V/S)} \tag{9}$$

$$d_1 = \frac{0.5C(1-g/C)^2}{1-[\min(1,X)g/C]} \tag{10}$$

$$d_2 = 900T[\,(X\text{-}1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}} \tag{11}$$

Total Control Delay (sec/veh) = d_1 . PF + d_2 (12)

Here, Arrival Type 3 was assumed and therefore P.F = 1

TABLE 9.3 Determination of Total Control Delay for the Base Case-1

Lane Group	V (vph)	S (vph)	V/S	C (sec)	g (sec)	c (vph)	X (v/c)	d_1 (s/veh)	d_2 (s/veh)	Total Control Delay (sec/veh)
EB	500	5085	0.088	65	19	1486	0.34	18	0.61	19
WB	1000	5085	**0.177**	65	19	1486	0.67	20	2.45	23
NB	1500	5085	0.265	65	38	2973	0.50	8	0.61	9
SB	2000	5085	**0.354**	65	38	2973	0.67	9	1.23	10
Sum of critical V/S			0.53							

Similarly, control delay for the proposed system was calculated. The proposed system was expected to provide a minimum of 2 seconds of extra green time for each approach without increasing the cycle length. It should be noted that this assumption of 2 seconds is on the conservative side since it is difficult to calculate precisely how much additional time can be provided. This in itself could be a topic for further research. The assumptions based on which this 2 sec value was described in the Study Design section. Based on this premise, the delay terms were calculated for the alternative case as shown in Table 9.4.

TABLE 9.4 Determination of Total Control Delay for the Alternative Case-1

Lane Group	V (vph)	S (vph)	V/S	C (sec)	g (sec)	c (vph)	X (v/c)	d_1 (s/veh)	d_2 (s/veh)	Total Control Delay (sec/veh)
EB	500	5085	0.088	65	21.5	1682	0.30	16	0.45	17
WB	1000	5085	0.177	65	21.5	1682	0.59	18	1.56	20
NB	1500	5085	0.265	65	40.5	3168	0.47	7	0.51	7
SB	2000	5085	0.354	65	40.5	3168	0.63	8	0.97	9

The next set of input volumes were then used to carry out the same analysis again. The analysis for the second set of volumes is presented in Table 9.5 and Table 9.6.

TABLE 9.5 Determination of Total Control Delay for the Base Case-2

Lane Group	V (vph)	S (vph)	V/S	C (sec)	g (sec)	c (vph)	X (v/c)	d_1 (s/veh)	d_2 (s/veh)	Total Control Delay (sec/veh)
EB	2500	5085	0.442	125	50	2040	1.23	37	106.11	144
WB	3000	5085	**0.531**	125	50	2040	1.47	37	214.55	252
NB	3500	5085	0.619	125	67	2720	1.29	29	132.00	161
SB	4000	5085	**0.708**	125	67	2720	1.47	29	213.87	243
Sum of critical V/S			1.29							

TABLE 9.6 Determination of Total Control Delay for the Alternative Case-2

Lane Group	V (vph)	S (vph)	V/S	C (sec)	g (sec)	c (vph)	X (v/c)	d_1 (s/veh)	d_2 (s/veh)	Total Control Delay (sec/veh)
EB	2500	5085	0.442	125	53	2142	1.17	36	81	117
WB	3000	5085	0.531	125	53	2142	1.40	36	183	219
NB	3500	5085	0.619	125	69	2821	1.24	28	111	139
SB	4000	5085	0.708	125	69	2821	1.42	28	190	218

A comparative analysis was carried out by combining the two cases. It can be seen from Figure 9.1 that considerable reduction in total control delay could be seen only for volumes greater than 2000 vph which in this case means a degree of saturation of 1. This finding supports our preliminary expectations that the system would be beneficial for oversaturated conditions. A paired t-test further confirmed that the reduction in control delay was statistically significant at 95% confidence level. The results of the paired t-test are presented in Table 9.7.

The analytical analysis of the system points towards the possibility of realizing benefits which would outdo the associated costs. However, to really be able to recommend or conclusively say that the system is worth the investment, simulation studies would be the best option. The next part of the data analysis section deals with evaluation of the system using CORSIM.

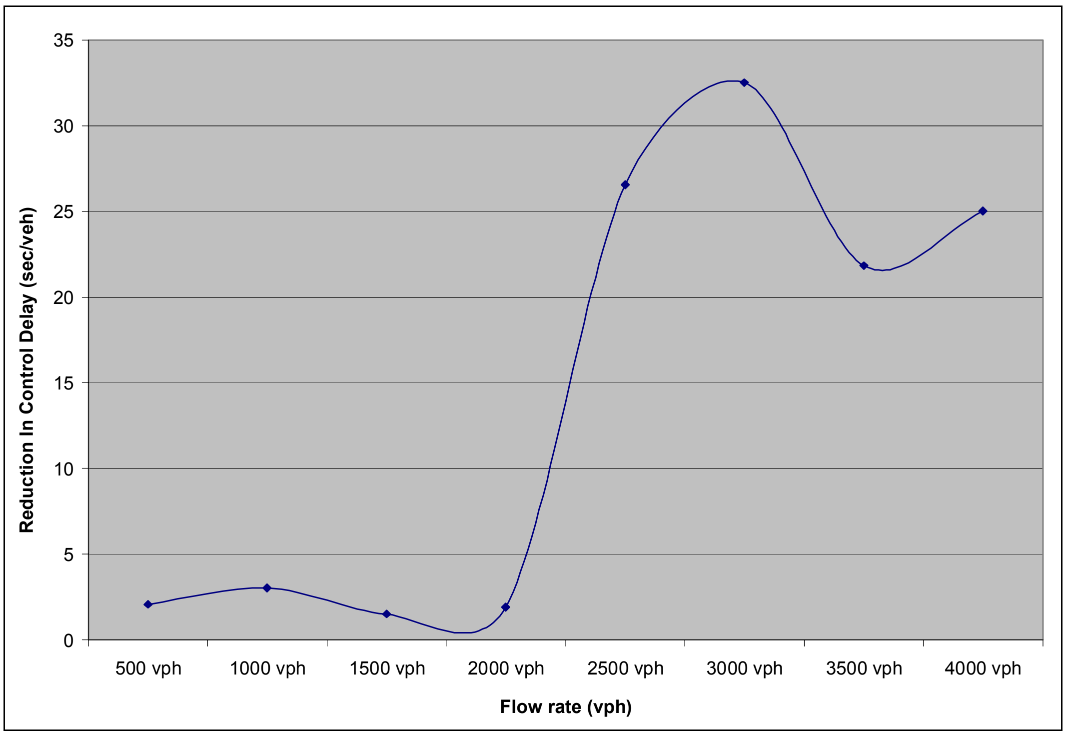

FIGURE 9.1 Reduction in Total Control Delay

TABLE 9.7 t-test: Paired Two Sample for Means

	Base Case	Alternate Case
Mean	107.49	93.19
Variance	11095.61	8577.37
Observations	8	8
Hypothesized Mean Difference	0	
Df	7	
t Stat	3.031	
P(T<=t) two-tail	0.019	
t Critical two-tail	2.365	

CHAPTER X

EVALUATION USING CORSIM

As described in the study methodology section, CORSIM was used to evaluate the proposed system using simulations of numerous scenarios classified as basic scenarios and general scenarios. This section presents the results of simulation and the subsequent analysis based on the simulation output.

BASIC SCENARIOS

Numerous pre-signal configurations were tried to determine the system's applicability under various restrictions. Each of the tried configuration and the need for the same are enlisted below:

1. *Pre-signals on all four approaches*: Such a configuration would be ideally suited for truly isolated intersections such that each approach is free from the influence of upstream intersections.
2. *Pre-signals on three approaches:* This configuration is ideally suited for intersections where one of the approaches may not be suitable for installation of Pre-signals because of its proximity to an upstream signal.
3. *Pre-signals on two approaches:* Such a configuration can be useful when two of the approaches may have the influence of nearby upstream thereby not satisfying the "isolated intersection" criteria.
4. *Pre-signals on one approach:* This type of configuration would be useful when only one of the approaches to the intersection can be classified as an approach to an isolated intersection.

The network was coded based on the scenarios presented in Table 6.1. A summary of the results and analysis is presented herewith:

A. *Pre-signals on all four approaches*: In this section, the system was tested under various scenarios and each scenario with its respective analysis is presented below:

i. *Input Volume = 2000 vph and Speed Limits of 30 mph, 40 mph and 50 mph*

Total Travel Time (veh-mins)

The average total travel time was calculated by averaging the total travel time over all the four approaches. For all the scenarios, the system performs rather poorly as can be seen from Figure 10.1. The figure illustrates the vast difference in total travel time for all offset distances in comparison with the baseline case.

Total Control Delay

Figure 10.2 illustrates that the system does not generate any benefits and induces additional control delay; almost twice that observed for the base case. Changing speed limits was also found to have had no effect as the additional control delay for each speed limit was approximately the same with respect to the baseline case.

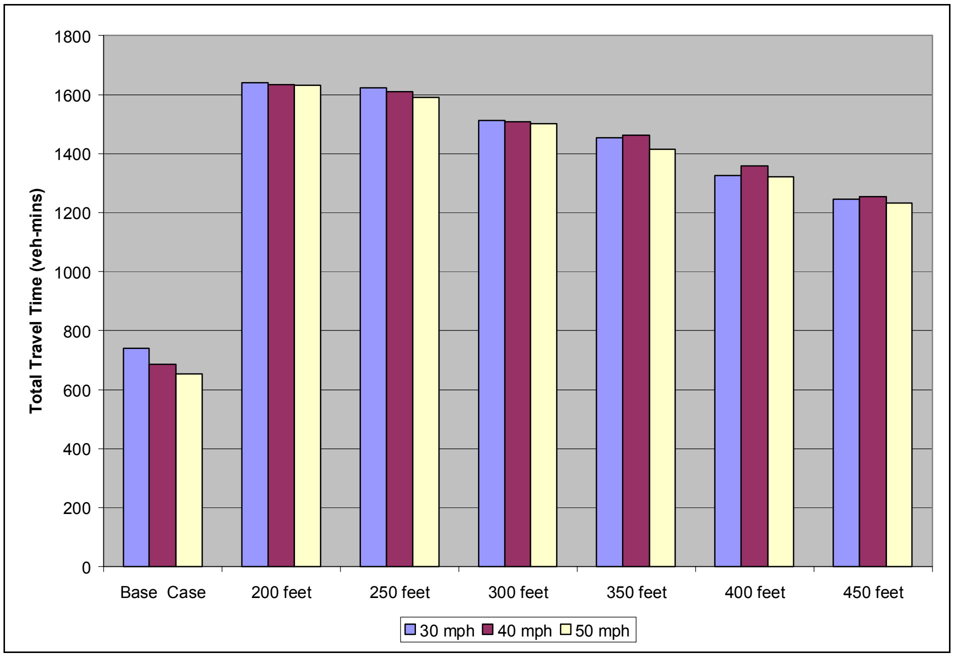

FIGURE 10.1 Total Travel Time for Case Ai

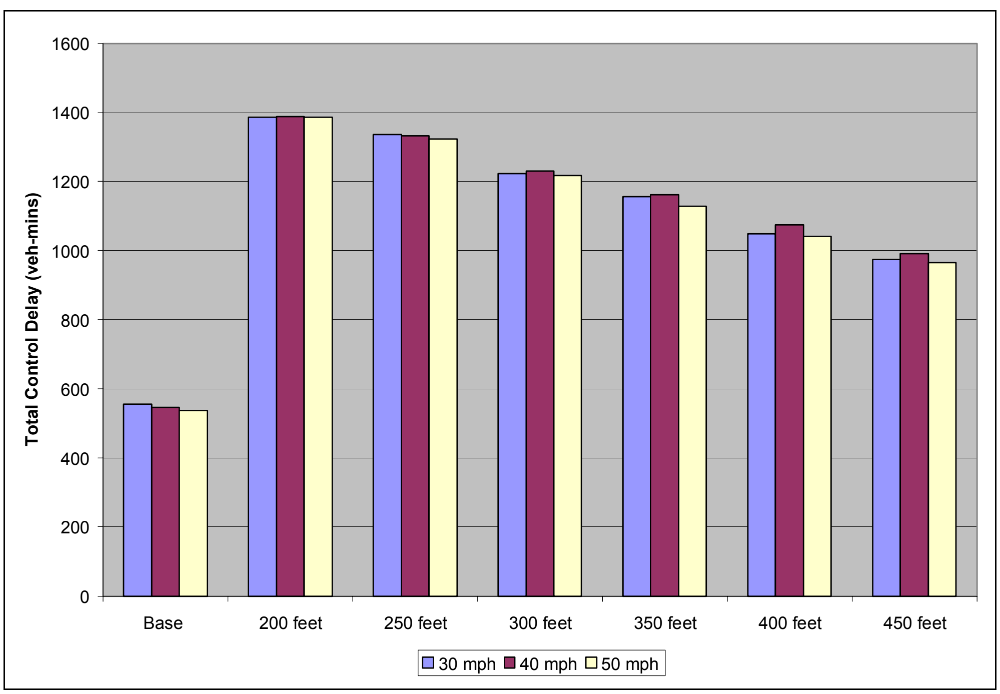

FIGURE 10.2 Total Control Delay for Case Ai

ii. *Input Volume = 2500 vph & Speed Limits of 30 mph, 40 mph and 50 mph.*

Total Travel Time (veh-mins)

For the 30 mph speed limit, only pre-signals with offset distances of 350 feet, 400 feet and 450 feet yielded travel time savings in comparison with the baseline case.

For the 40 mph speed limit, pre-signals with offsets ranging from 250 feet thru 450 feet were found to have generated travel time benefits. However, the benefits for 250 feet were negligible and hence the real benefits can be said to have been realized only beyond 300 feet with the maximum occurring for the 400 feet offset.

Except for the 450 feet offset, all other options failed to generate any travel time savings for the 50 mph speed limit. Even the benefits generated by the 450 feet offset were marginal and hence it appears that this speed limit does not work out in favor of the system for the given set of parameters. The only plausible reason that can explain this trend is the steep speed gradient between the front row and last row vehicles. This speed gradient is expected to be larger at higher speed limits because the former with no obstructions ahead of them are expected to attain final velocity much quicker compared with the latter. This probably means that the last row vehicles end up reaching the main signal during the red at the main intersection and then waiting for an additional cycle.

Observing Figure 10.3 closely leads us to believe that the 40 mph speed limit is the best amongst the given set of speed limits thereby contributing in a negative way to the total control delay and total travel time.

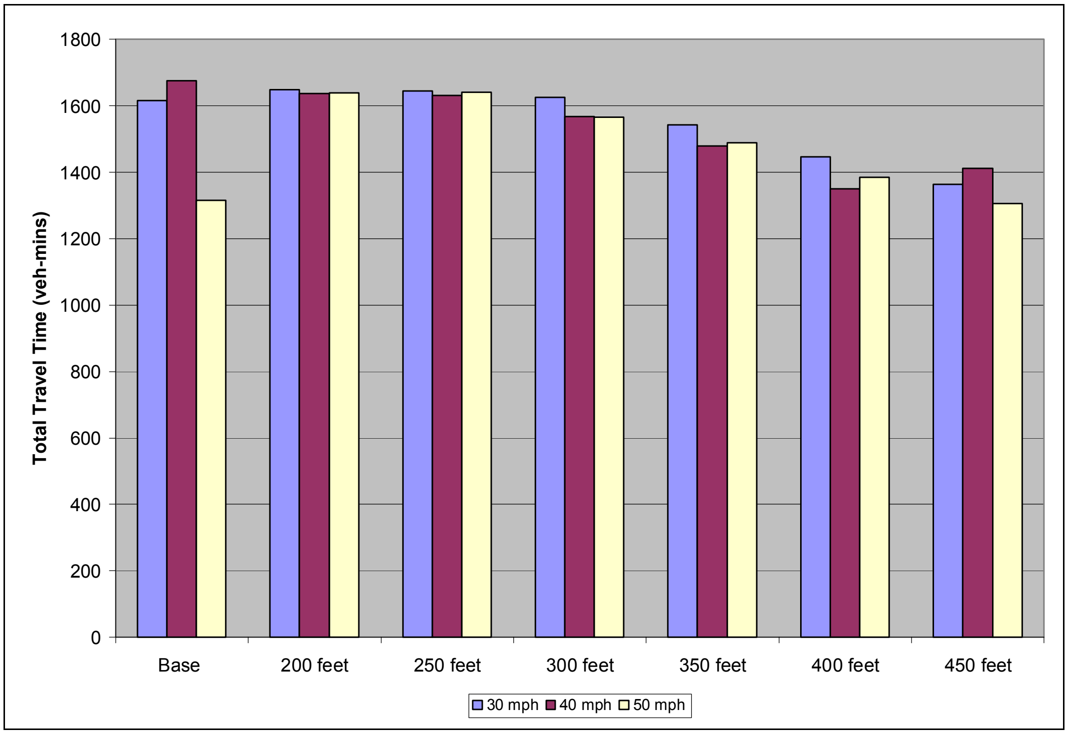

FIGURE 10.3 Total Travel Time for Case A-ii

Total Control Delay (veh-mins)

Figure 10.4 illustrates the change in total control delay with respect to the base line case for various offset distances and speed limits. The 30 mph speed limit was observed to have generated benefits for offsets greater than 200 feet and the benefits increase linearly from thereon up to a maximum corresponding to an offset distance of 450 feet. The 40 mph speed limit was found to have the maximum impact as far as reduction in control delay is concerned. The maximum in this case was observed for offset distances of 400 feet and 450 feet with both generating approximately the same reduction in total control delay. The weakest link in this case was the 50 mph speed limit as the benefits were observed only for offsets greater than 200 feet. Although a trend line for this speed limit would appear similar to the trend line for the 30 mph speed limit, it is important to note that the latter was able to generate greater benefits. A one way

ANOVA was carried out to determine if the influence of speed limits (treatments) on the total control delay was significant using a significance level of 0.05. It was found that the calculated value of F-statistic (0.22) was less than the tabulated value of F-statistic (3.68) indicating that the difference between the treatments (speed limits) is not significant at 95% confidence level.

From Figure 10.4 it can be seen that lowest total control delay (1214 veh-mins) for the base line case was for the 50 mph speed limit. Using this value as the base value, the change in total control delay affected by each speed limit was calculated and is presented in Table 10.1. ANOVA on Table 10.1 gave the exact same results as described above. Hence it can be said at 95 % confidence level that the reduction in total control delay is not significantly different for the various speed limits tried.

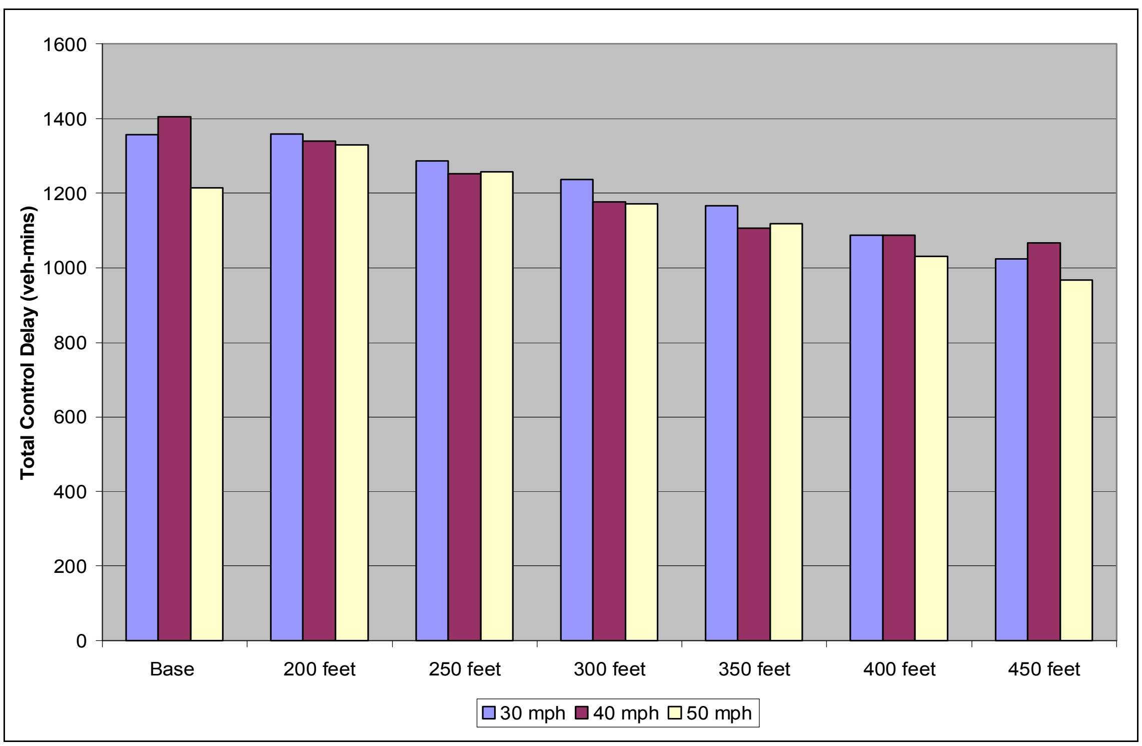

FIGURE 10.4 Total Control Delay for Case A-ii

TABLE 10.1 Reduction in Total Control Delay for Case A-ii

Offset Distance →	200 feet	250 feet	300 feet	350 feet	400 feet	450 feet
30 mph	-145	-73	-23	47	128	192
40 mph	-126	-39	39	109	128	147
50 mph	-116	-43	42	97	185	247

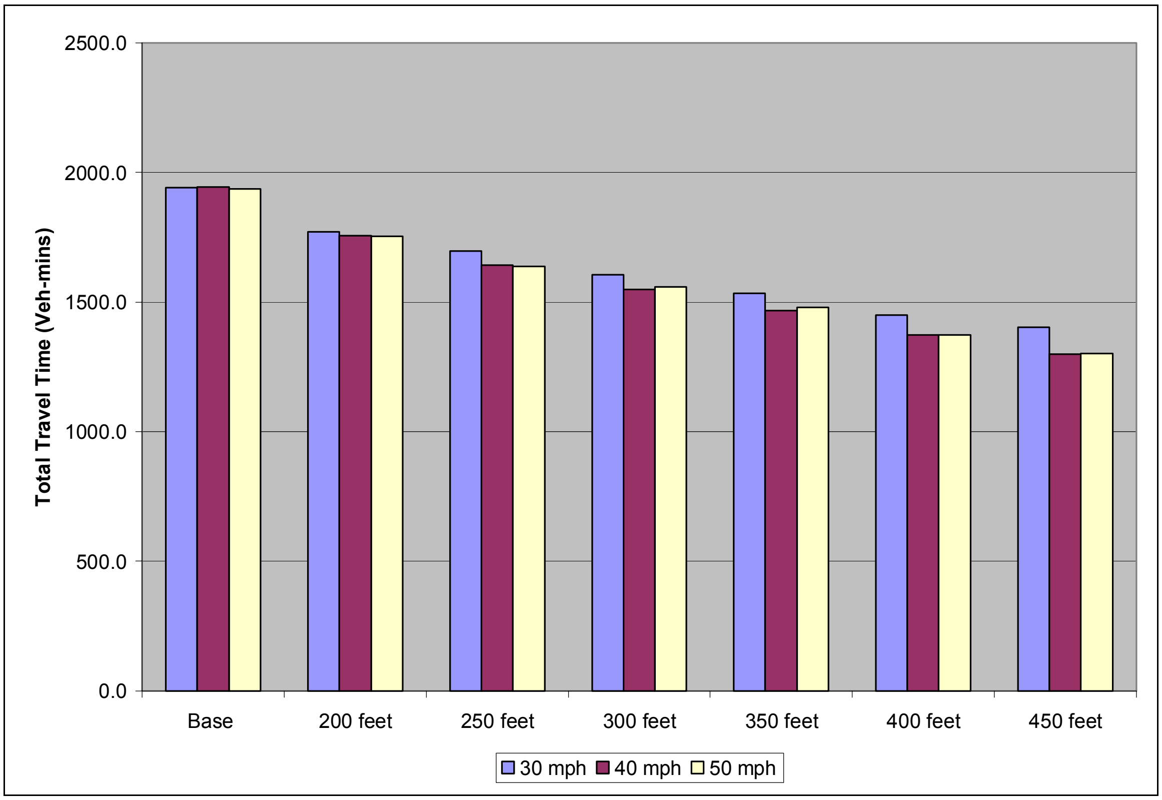

FIGURE 10.5 Total Travel Time for Case A-iii

iii. *Input Volume = 3000 vph and speed limits of 30 mph, 40 mph and 50 mph*

Total Travel Time (veh-mins)

For all the speed limits, the benefits linearly increased as the offset distance was increased. It was observed that the 450 feet offset yielded maximum travel time benefits as can be seen in Figure 10.5. It can be said that the most optimum speed limit was 40 mph followed by 50 mph and 30 mph in that order of effectiveness.

Total Control Delay (veh-mins)

An increase in offset distance was found to have increased the benefits with the maximum benefits being realized for the 450 feet offset distance (Figure 10.6). Yet again, the 50 mph speed limit was found to have the least total control delay for the 450 feet offset distance. However, an ANOVA done on the total control suggested that the treatments (speed limits) induced total control delays, which were less than the base case, but cannot be said to be significantly different from each other. The calculated F-statistic (0.33) was less than the F-critical (3.68) and hence it can be said that the treatments resulted in values not significantly different from each other at 95% confidence level. Table 10.2 presents the actual reduction in total control delay with respect to the baseline value of 1359 veh-mins (40 mph speed limit). An ANOVA again suggested that reduction in total control delay due to the treatments was not significantly different with F-calculated (0.07) being much less than F-tabulated (3.68). Hence, it can be said that all the speed limits generated benefits which were significantly different from each other.

TABLE 10.2 Reduction in Total Control Delay for Case A-iii

Offset Distance →	200 feet	250 feet	300 feet	350 feet	400 feet	450 feet
30 mph	4	84	144	197	268	293
40 mph	24	109	196	248	342	284
50 mph	31	113	183	260	345	406

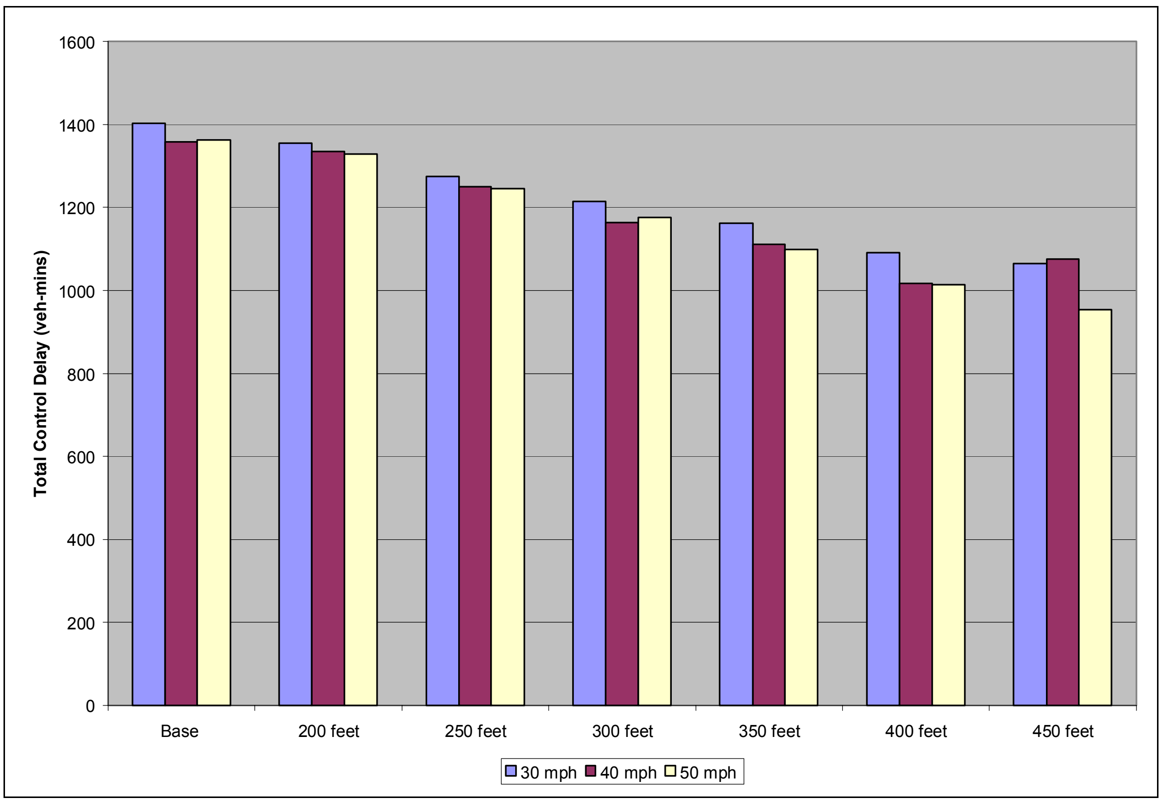

FIGURE 10.6 Total Control Delay for Case A-iii

Summary for Case A

The results obtained from simulation of all the scenarios have been presented in Table 10.3. A close examination of the results indicates that travel time benefits were realized only for 5 scenarios, viz., Scenarios 4, 5, 7, 8 and 9 (Refer Appendix II). This points out that the scenarios with input volumes of 2000 vph did not have any travel time savings suggesting that the system would work for oversaturated conditions, in this case above 2500 vph. Even with the 2500 vph traffic volume, the 50 mph speed limit failed to generate any considerable benefits. The maximum savings which were in excess of 600 veh-minutes for a 15 minute analysis period were observed to be associated with the 450 feet offset for Scenarios 8 and 9. As a result of thee findings, it can be said that the most favorable scenario for the system was Scenario 8. These savings can be transformed into a monetary value. Assuming that the average vehicle occupancy of 1.2 (7) and an average value of time of $ 8 (8), we can get the dollar value for the savings. It is to be noted that both these assumptions have been made on the conservative side and hence the estimates also will be conservative thereby in some way counteracting the possibility of some overestimation made by the CORSIM model.

For the most optimum combination, the travel time savings = 643 vehicle-minutes

Total Savings per day ($) = (Savings/15-min period) x 4 x (Average Vehicle Occupancy) x (Average Value of travel time savings) x (average peak hours per day)

$$= \frac{643}{60} \text{ x } 4 \text{ x } 1.2 \text{ x } 8 \text{ x } 4 = \$\ 1647$$

The system thus has the capability of saving approximately $ 1650 per approach per day, which by any standards is a significant amount. However, it may not always be possible to provide 450-feet offset and hence other feasible option also need to be identified. Table 10.4 provides dollar value of savings for each of the scenarios where benefits were observed.

TABLE 10.3 Summary for the "Pre-Signal on All Four Approaches" Case

Offset Distances →	200 feet	250 feet	300 feet	350 feet	400 feet	450 feet
SCENARIO 1	-899.84	-882.62	-773.04	-714.79	-587.21	-506.37
SCENARIO 2	-947.07	-923.13	-822.28	-775.84	-923.13	-567.81
SCENARIO 3	-978.53	-937.03	-846.92	-760.95	-668.13	-578.25
SCENARIO 4	-136.73	-63.68	-9.37	74.15	168.68	251.10
SCENARIO 5	-84.43	20.36	108.08	196.85	324.66	264.22
SCENARIO 6	-426.41	-350.52	-251.04	-174.99	-70.16	10.02
SCENARIO 7	169.98	245.61	336.51	408.50	491.40	540.40
SCENARIO 8	188.11	299.85	395.92	475.00	570.24	643.47
SCENARIO 9	184.33	298.93	378.64	457.76	564.77	634.25

Table 10.4 Travel Time Savings ($) for Case A

Offset Distances →	200 Feet	250 Feet	300 Feet	350 feet	400 Feet	450 feet
SCENARIO 4	-350	-163	-24	190	432	643
SCENARIO 5	-216	52	277	504	831	676
SCENARIO 6	-1092	-897	-643	-448	-180	26
SCENARIO 7	435	629	861	1046	1258	1383
SCENARIO 8	482	768	1014	1216	1460	**<u>1647</u>**
SCENARIO 9	472	765	969	1172	1446	1624

It should be noted that a full blown cost benefit was not done but based on common traffic engineering knowledge, the cost of installing a signal system was assumed to be around $ 100,000 to $ 200,000. This assumption was used to get a rough estimate of system's cost effectiveness.

B. *Pre-signals on three approaches*: For this configuration, the WB approach was coded without a pre-signal while all other approaches had pre-signals and the scenarios from 10 thru 18 (refer Appendix 1) were coded in CORSIM for evaluation.

i. *Input volume = 2000 vph and speed limits of 30 mph, 40 mph and 50 mph*

Total Travel Time (veh-mins)

Consistent with the observations made in the previous configuration (CASE A-i); scenarios evaluated for this case also fail to generate any travel time benefits. This can be seen clearly from Figure 10.7. In accordance with per prior expectations, low demand level can be attributed for these negative benefits.

Total Control Delay

Similar to the observation made in the evaluation of total travel time, the system induces large delays in comparison with the baseline case Figure 10.8. Since the results do not show any benefits, further evaluation of the scenario using ANOVA has been left out as the results will have no significant meaning.

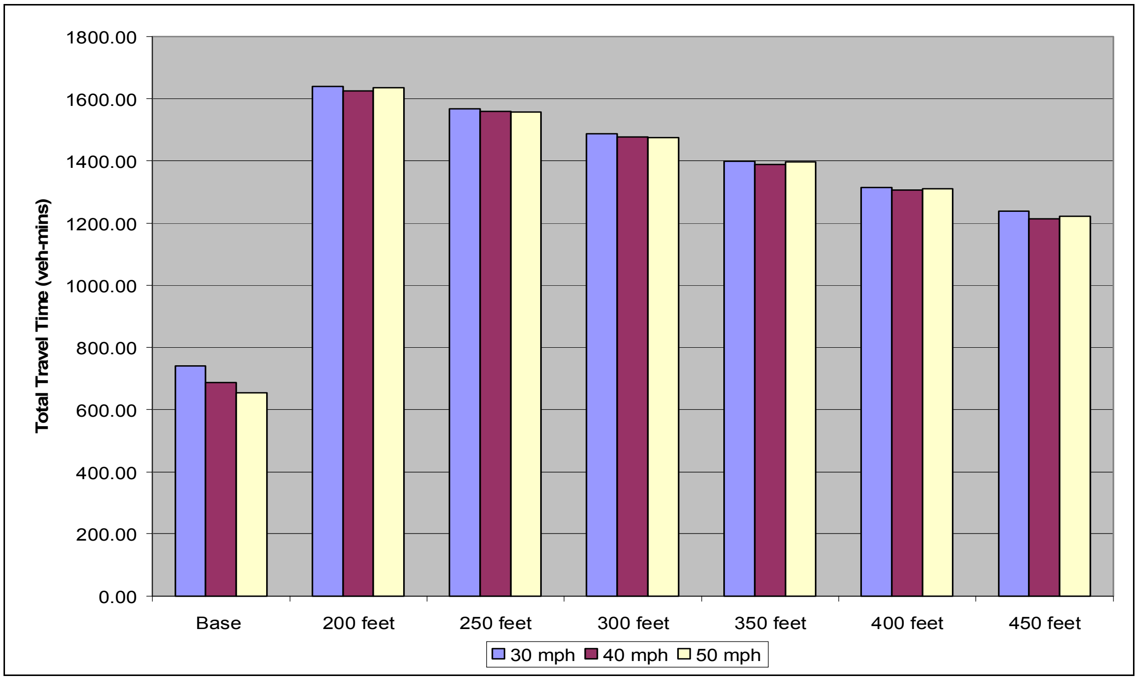

FIGURE 10.7 Total Travel Time for Case B-i

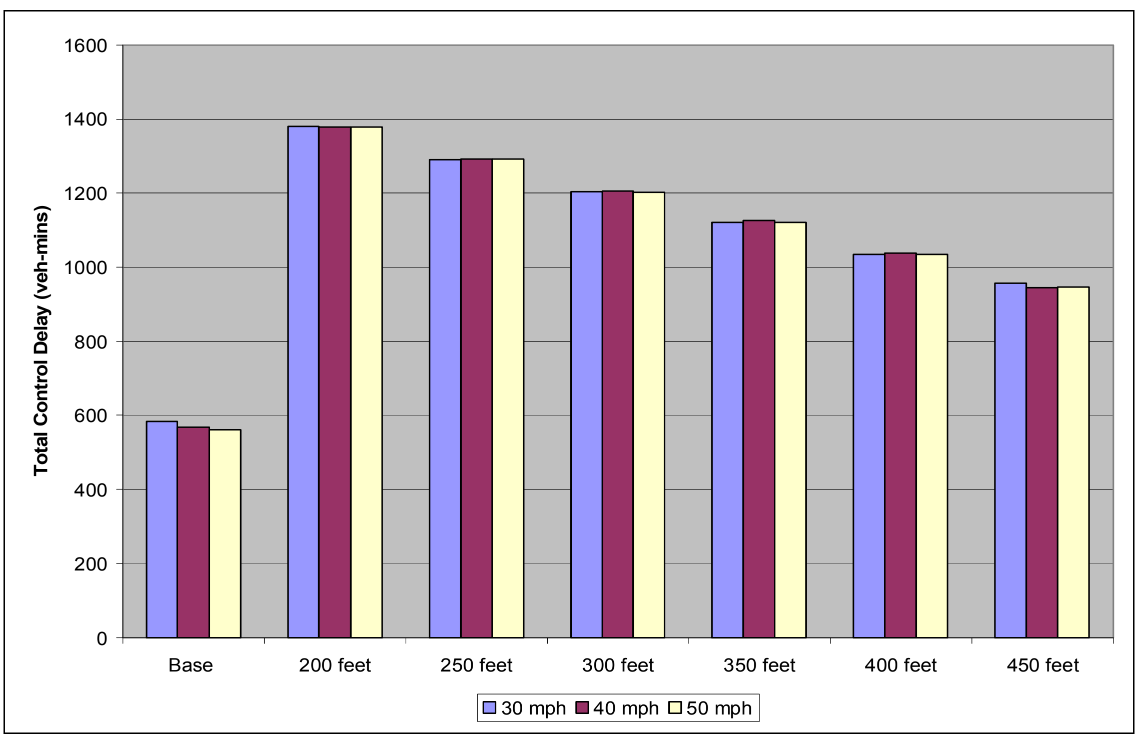

FIGURE 10.8 Total Control Delay for Case B-i

ii. *Input volume = 2500 vph and speed limits of 30 mph, 40 mph and 50 mph*

Total Travel Time (veh-mins)

For the 30 mph and 40 mph speed limits, the average travel time calculated over the three approaches with pre-signals was found to be less than that of the baseline case for offsets greater than 250 feet with 40 mph speed limit being the better of the two. The maximum travel time savings were observed for the 450 feet offset as can be observed from Figure 10.9.

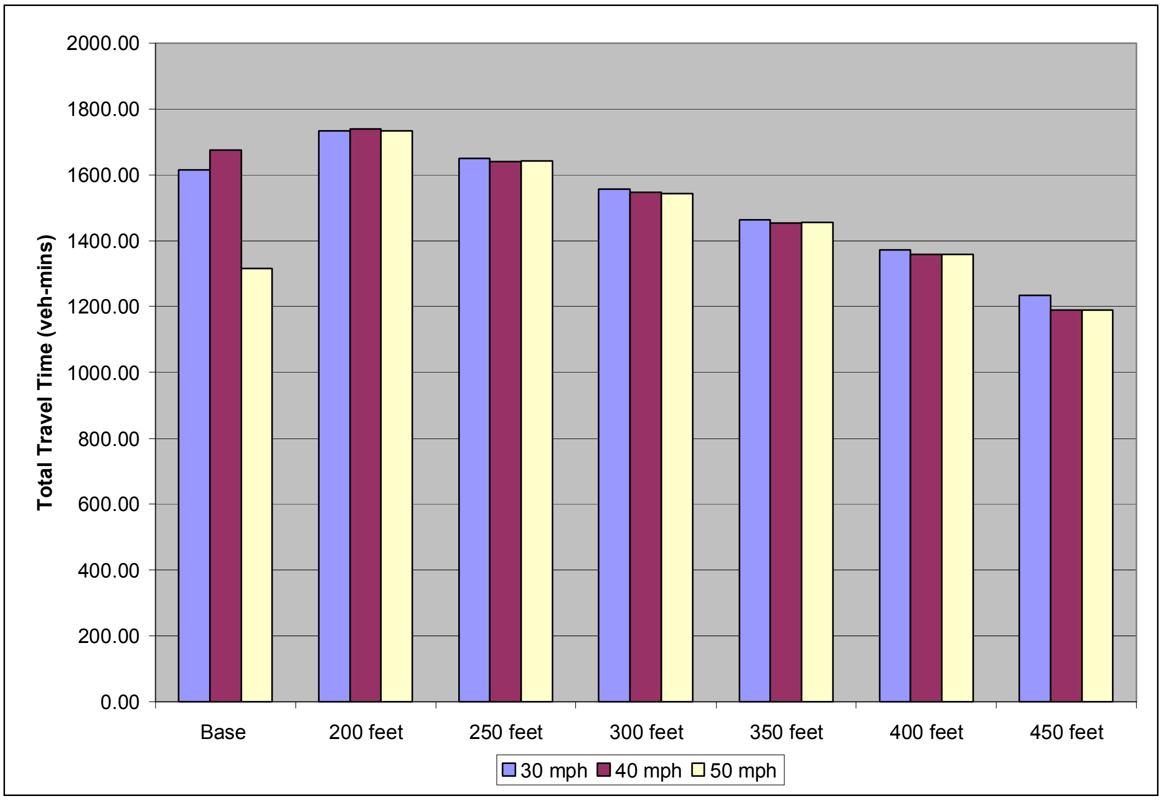

FIGURE 10.9 Total Travel Time for Case B-ii

Total Control Delay (veh-mins)

The trend line in Figure 10.10 for the 30 mph and 50 mph speed limits appears similar as the total control delay shoots up a little for the 250 feet offset distance and then decreases almost as a linear function of the offset distance. The 40 mph speed limit's trend line however appears linear from the beginning as it reaches a minimum value for the 450 feet offset distance. The greatest drop in total control delay can therefore be attributed to the 40 mph speed limit (Table 10.5).

ANOVA was used to determine if the treatments (variation in speed limits) caused any significant changed. The statistical test resulted in the conclusion that the treatments were not significantly different from each other at 95 % confidence level as the calculated F-statistic (0.97) was less than the tabulated F-statistic (3.68). ANOVA was also used to evaluate the magnitude by which total control delay drops for each speed limit. Here again, the calculated F-statistic (0.97) was less than the tabulated F-statistic (3.68) hence suggesting that the treatments are not significantly different from each other at 95 % confidence level. Figure 10.10 presents the reduction in total control delay associated with each speed limit.

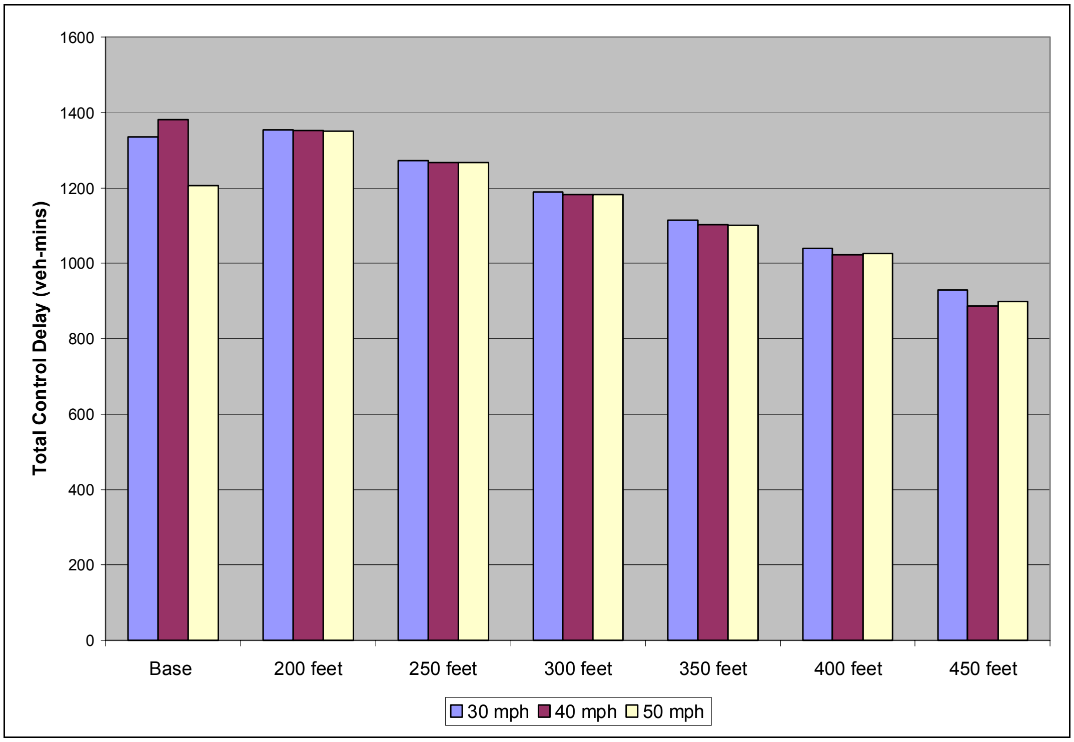

FIGURE 10.10 Total Control Delay for Case B-ii

TABLE 10.5 Reductions in Total Control Delay for Case B-ii

Offset Distance →	200 feet	250 feet	300 feet	350 feet	400 feet	450 feet
30 mph	-148	-66	17	92	167	277
40 mph	-146	-61	24	104	183	320
50 mph	-144	-61	24	105	180	308

iii. *Input Volume = 3000 vph and speed limits of 30 mph, 40 mph and 50 mph*

Total Travel Time (veh-mins)

As the input volume was increased from 2500 vph to 3000 vph, a sudden change was observed in that all offset distances were found to have been generating travel time savings as can be seen in Figure 10.11. For the set of conditions given above, the system's performance was observed to be reasonable. Moreover, in this case also, the maximum benefits were observed for the 450 feet offset distance.

As observed previously, the benefits in this case also tend to increase with the offset distance while the travel time on the WB approach as expected remains almost the same yielding low negative benefits.

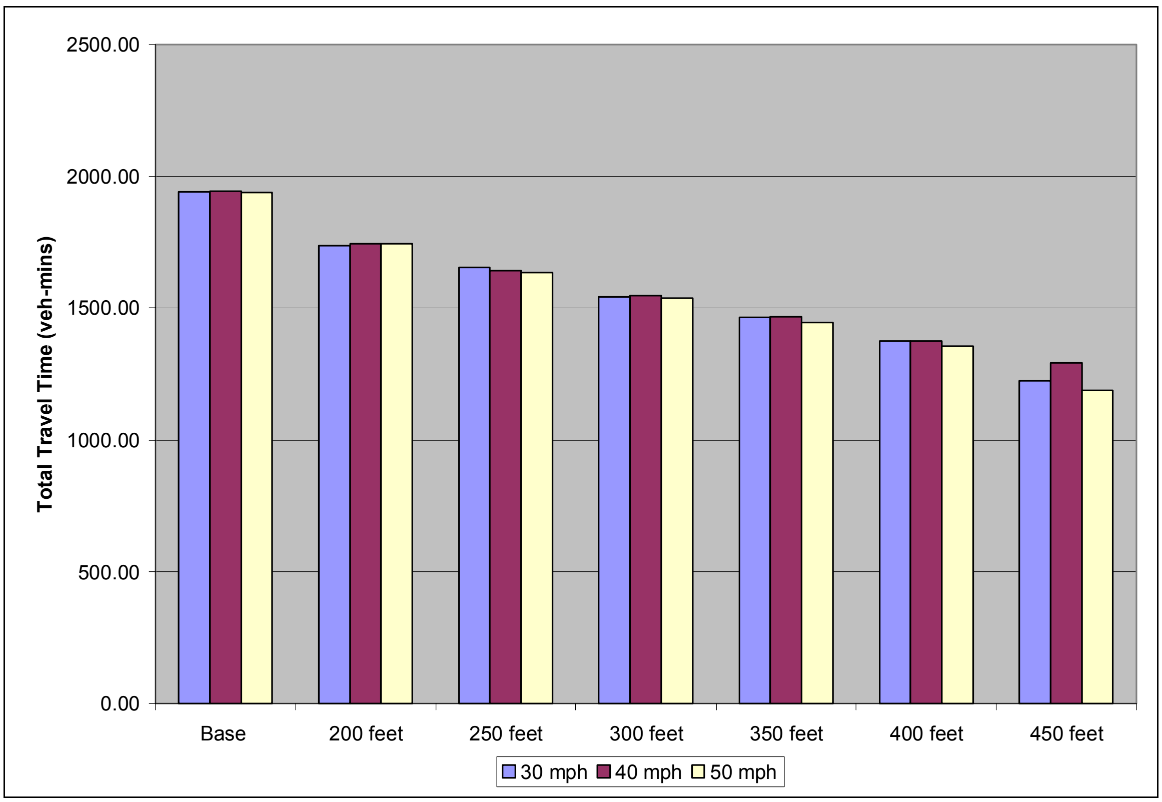

FIGURE 10.11 Total Travel Time for Case B-iii

Total Control Delay (veh-mins)

The offset distance when increased yet again resulted in a reduction in the total control delay for all three speed limits. The reduction was observed to be almost linear for the 30 mph speed limit starting from the 200 feet offset. For the 40 mph and 50 mph speed limits however, the delay increased marginally for the 200 feet offset and from thereon dipped below the base line scenarios' delay to reach the minimum at 450 feet as shown in Figure 10.12. An ANOVA on the total control delay (veh-mins) suggested that the treatments do not yield any significantly different results at 95% confidence level. The F-statistic was found to be 0.016 for both, the absolute total control delay and the reduction in total control delay. This value was much lesser than F-critical (3.68) thereby indicating that the three treatments were not significantly different from each other. This can be seen from Table 10.6, which tabulates the actual reduction in control delay with respect to the baseline case's least total control delay case (i.e. 50 mph).

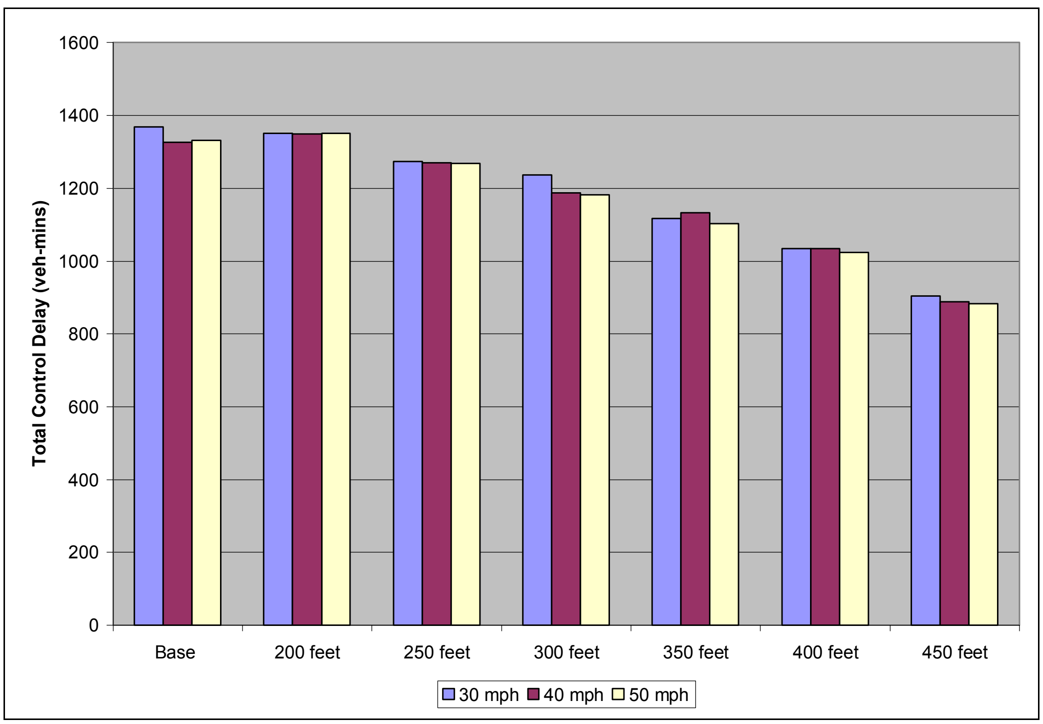

FIGURE 10.12 Total Control Delay for Case B-iii

TABLE 10.6 Reduction in Total Control Delay for Case B-iii

	200 feet	250 feet	300 feet	350 feet	400 feet	450 feet
30 mph	-25	51	89	210	292	421
40 mph	-22	56	138	193	292	438
50 mph	-26	58	144	222	301	441

Summary for Case B

Table 10.7 provides the actual travel time savings associated with each configuration of the proposed system. It is difficult to pick a particular trend but it can be said that benefits can be seen for all the offset distances tried for the cases where input volume was 3000 vph. Moreover, benefits can be seen for some of the offsets in Scenarios 13 thru 15 also. On the contrary, none of the offset distance or speed restriction options were able to generate any travel time benefits for the 2000 vph demand level and hence it can be said that the system with three pre-signalized approaches works well only for volumes above 2500 vph. Table 10.8 presents the monetary value of travel time savings per day (assuming peak period operations only) based on the assumption made previously. The minimum positive benefits were observed for Scenario 14 with an offset of 250 feet, which were of the order of approximately $5000 per day. The maximum however was well over $ 100,000 per day. It was again observed that the 50 mph speed limit for the 2500 vph demand level, generated benefits only for the 450 feet offset distance. The reason can be expected to be the same as described previously.

Since this configuration had one approach which was not provided with a pre-signal, it is bound to have incurred travel time losses. The real benefits of the system therefore would be the net benefits obtained by deducting the travel time losses incurred on the

WB approach from the benefits realized on the other approaches. The net benefits are shown in Table 10.9. It can be seen that despite the losses incurred on the WB approach, the net benefits still have a positive sign for the conducive set of site characteristics.

TABLE 10.7 Summary for the "Pre-Signal on Three Approaches" Case

Offset Distance →	200 Feet	250 Feet	300 Feet	350 feet	400 feet	450 feet
SCENARIO 10	-900.82	-827.65	-747.20	-658.92	-574.56	-498.06
SCENARIO 11	-938.19	-872.77	-791.26	-702.92	-619.68	-527.82
SCENARIO 12	-981.29	-902.68	-821.05	-743.38	-656.06	-569.15
SCENARIO 13	-119.05	-36.02	58.15	151.63	244.25	381.04
SCENARIO 14	-64.50	33.80	127.53	220.39	316.80	485.95
SCENARIO 15	-419.41	-327.27	-227.68	-140.72	-44.76	124.68
SCENARIO 16	205.10	287.89	398.84	479.13	565.43	717.82
SCENARIO 17	198.49	301.08	395.24	476.43	567.94	651.67
SCENARIO 18	194.16	302.68	399.18	492.24	581.89	749.61

TABLE 10.8 Travel Time Savings ($) for Case B

Offset Distance →	200 feet	250 feet	300 Feet	350 feet	400 feet	450 feet
SCENARIO 10	-2306	-2119	-1913	-1687	-1471	-1275
SCENARIO 11	-2402	-2234	-2026	-1799	-1586	-1351
SCENARIO 12	-2512	-2311	-2102	-1903	-1680	-1457
SCENARIO 13	-305	-92	149	388	625	975
SCENARIO 14	-165	87	326	564	811	1244
SCENARIO 15	-1074	-838	-583	-360	-115	319
SCENARIO 16	525	737	1021	1227	1448	1838
SCENARIO 17	508	771	1012	1220	1454	1668
SCENARIO 18	497	775	1022	1260	1490	1919

TABLE 10.9 Net Travel Time Savings ($) for Case B

Offset Distance →	200 Feet	250 Feet	300 Feet	350 feet	400 feet	450 feet
SCENARIO 10	-5183	-4990	-4814	-4397	-4111	-4038
SCENARIO 11	-5356	-5223	-5050	-4791	-4450	-4142
SCENARIO 12	-5204	-4801	-4383	-3986	-3538	-3094
SCENARIO 13	-1129	-830	-547	-326	-126	502
SCENARIO 14	-1873	-1575	-1246	-1003	-766	-90
SCENARIO 15	-2010	-1759	-1477	-1248	-940	-301
SCENARIO 16	185	433	878	1212	1381	1793
SCENARIO 17	400	778	1035	1445	1707	1950
SCENARIO 18	347	769	987	1397	1652	2042

C. Pre-signals on two approaches

For this configuration, a little change has been made compared to the configuration evaluated in B. Instead of providing extra green to the approaches with pre-signals, the additional time was allocated to the approaches without pre-signals (EB and WB) to see if and how benefits get transferred. It was therefore important to also evaluate how the system would perform if the additional green was transferred to two of the approaches without pre-signals. To exactly determine the extent of benefits and associated negative benefits on the approaches with pre-signals (but without additional green), each MOE was evaluated with respect to both type of approaches, viz., with and without pre-signals. The approaches without pre-signals were provided with lead-lead type phasing while the approaches with pre-signals had split phasing.

i. Input Volume = 2000 vph and speed limit of 30 mph, 40 mph and 50 mph

Total Travel Time (veh-mins) and Total Control Delay (veh-mins) for approaches without pre-signals (EB and WB)

It can be observed from Figure 10.13 and Figure 10.14 that travel time and control delay associated with the approaches under study was higher for the proposed alternative as compared with the corresponding approaches in the baseline case. The increase in travel time and total control delay for the approaches without pre-signals was actually expected because of the phasing sequence. For this scenario, the NB and SB had split phasing while EB & WB had lead-lead type of phasing. Due to the former two approaches, the phase for the EB and WB was reduced in duration thereby resulting in inefficient operation.

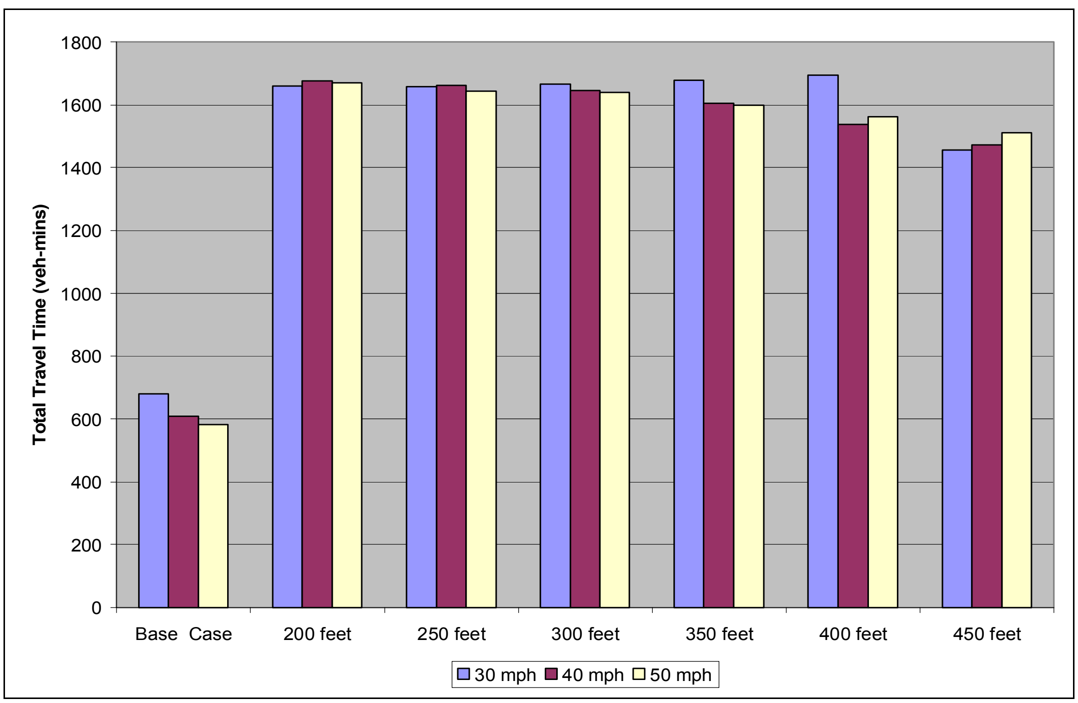

FIGURE 10.13 Total Travel Time for the EB and WB approaches

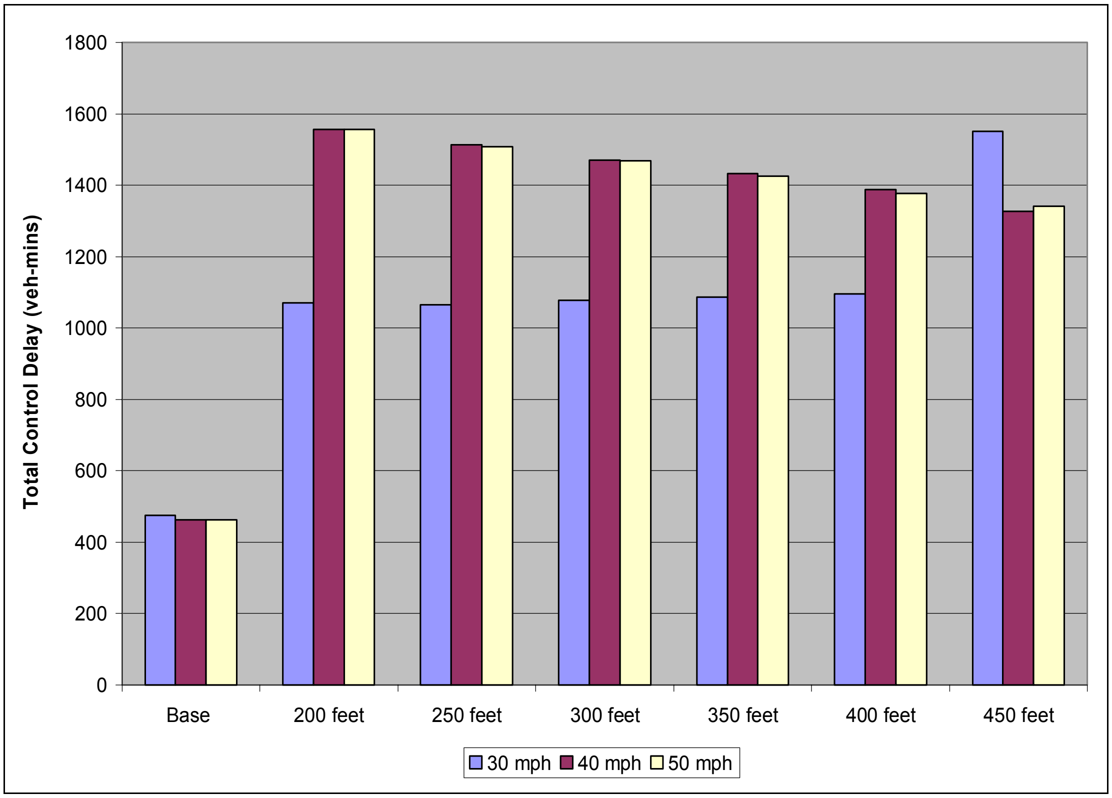

FIGURE 10.14 Total Control Delay for EB and WB approaches

Total Travel Time (veh-mins) and Total Control Delay (veh-mins) for approaches with pre-signals (NB and SB)

The travel time and total control delay on approaches with pre-signals also did not show any benefits as has been the case for this demand level (Figure 10.15 and Figure 10.16). Since both, approaches with & without pre-signals, do not show any benefits, further evaluation of the travel time savings was not conducted.

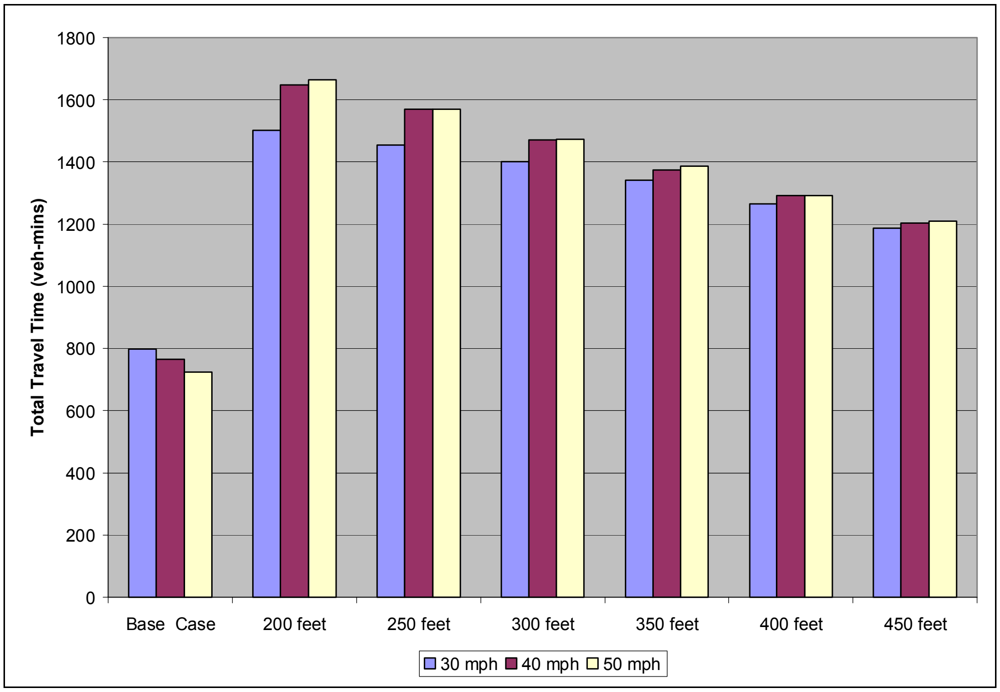

FIGURE 10.15 Total Travel Time for NB and SB approaches in Scenario C-i

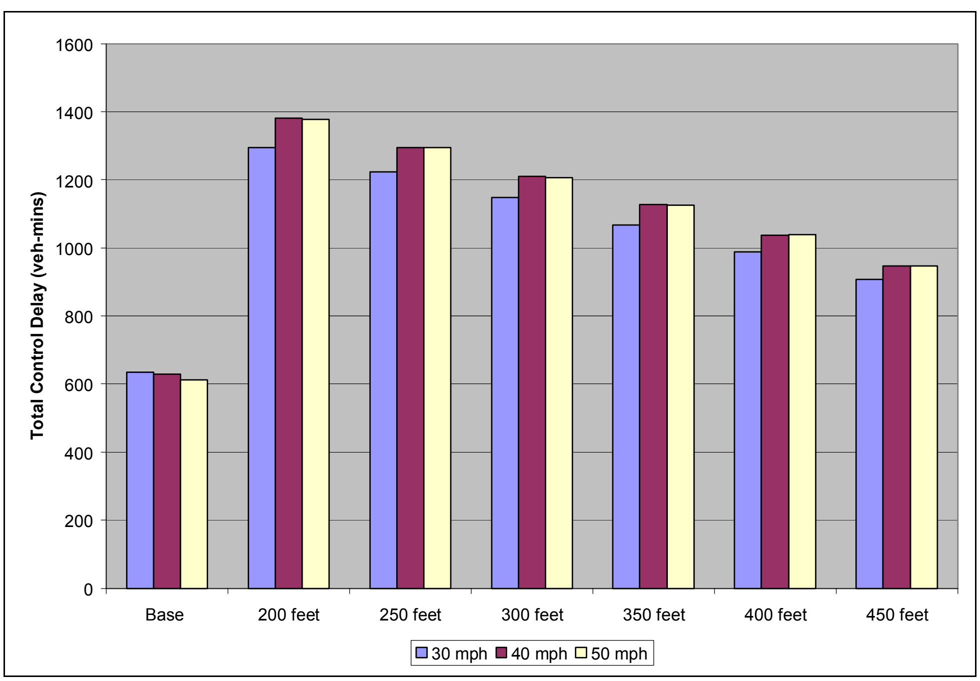

FIGURE 10.16 Total Control Delay for NB and SB approaches in Scenario C-i

ii. Input Volume = 2500 vph and speed limit of 30 mph, 40 mph and 50 mph.

Total Travel Time (veh-mins) and Total Control Delay (veh-mins) for approaches without pre-signals (EB and WB)

Increasing the volume to 2500 also did not help the cause much as both the MOEs were observed to be showing negative benefits for this approach (Figure 10.17 and Figure 10.18). The reason yet again remains the same as mentioned previously, i.e. the difference in phasing sequence between the baseline case and the proposed alternative. Since no benefits were realized, no statistical tests were conducted to evaluate any trends.

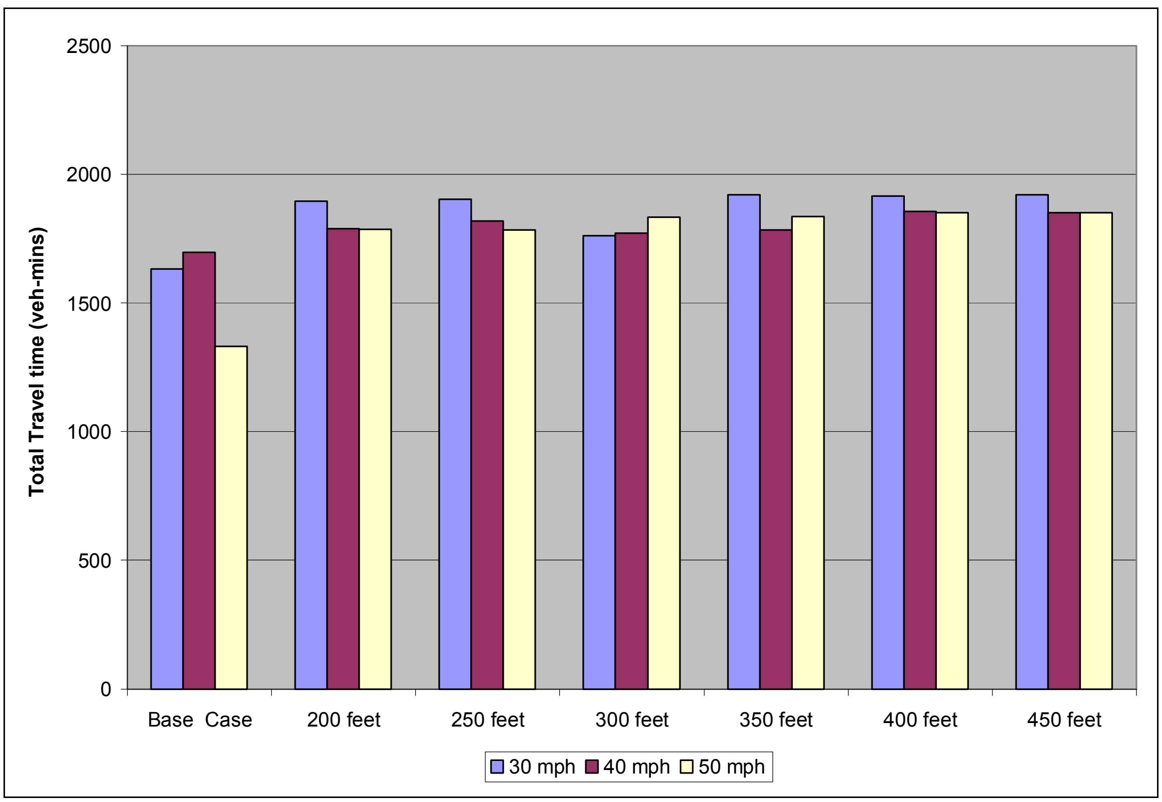

FIGURE 10.17 Total Travel Time for the EB and WB approaches in Scenario C-ii

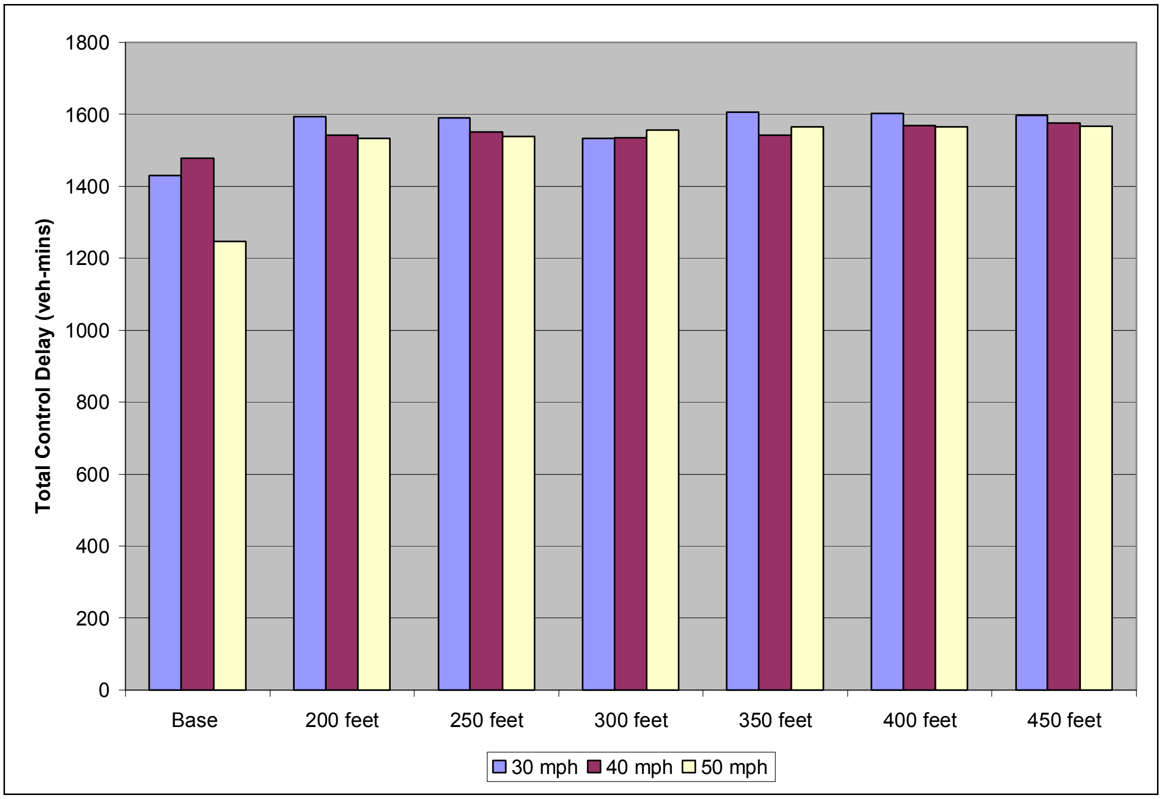

FIGURE 10.18 Total Control Delay for EB and WB approaches in Scenario C-ii

Total Travel Time (veh-mins) and Total Control Delay (veh-mins) for approaches with pre-signals (NB and SB)

Figure 10.19 illustrates the impact of offset distances and speed limits on the reduction in travel time. Benefits with the 30 mph speed limit were observed for offsets greater than 250 feet. For the 40 mph speed limit, benefits were observed for offsets greater than 200 feet while the corresponding offset distance for the 50 mph speed limit was 450 feet. None of the other options yielded any benefits for the 50 mph speed limit.

The proposed system was also able to bring about a reduction in the total control delay (Figure 10.20). The benefits for the 30 mph and 40 mph speed limits were observed for offsets greater than 200 feet while the corresponding offset distance for the 50 mph speed limits was 300 feet. For case C-ii, it has been observed that the 50 mph speed limit has been the most inefficient of the three. Moreover, the benefits were observed only for the higher offset distances. The reason behind this could probably be associated with the way travel time is calculated in CORSIM. For the base case, the backed up queue may be increasing the travel time. In the alternative scenarios however, there are two components of the total travel time, viz., travel time upstream of the pre-signal and travel time across the offset distance. For higher offset distances, even in the presence of queues, the vehicles reach the back of the pre-signal queue earlier as the link is shorter. Moreover, the latter section is traversed at higher speed hence resulting is lower overall travel time. The speed gradient/differential may be one factor which may be influencing it further. These effects were not investigated deeper as this section looks at the macroscopic results. However, microscopic evaluation can be carried out in future studies on this subject.

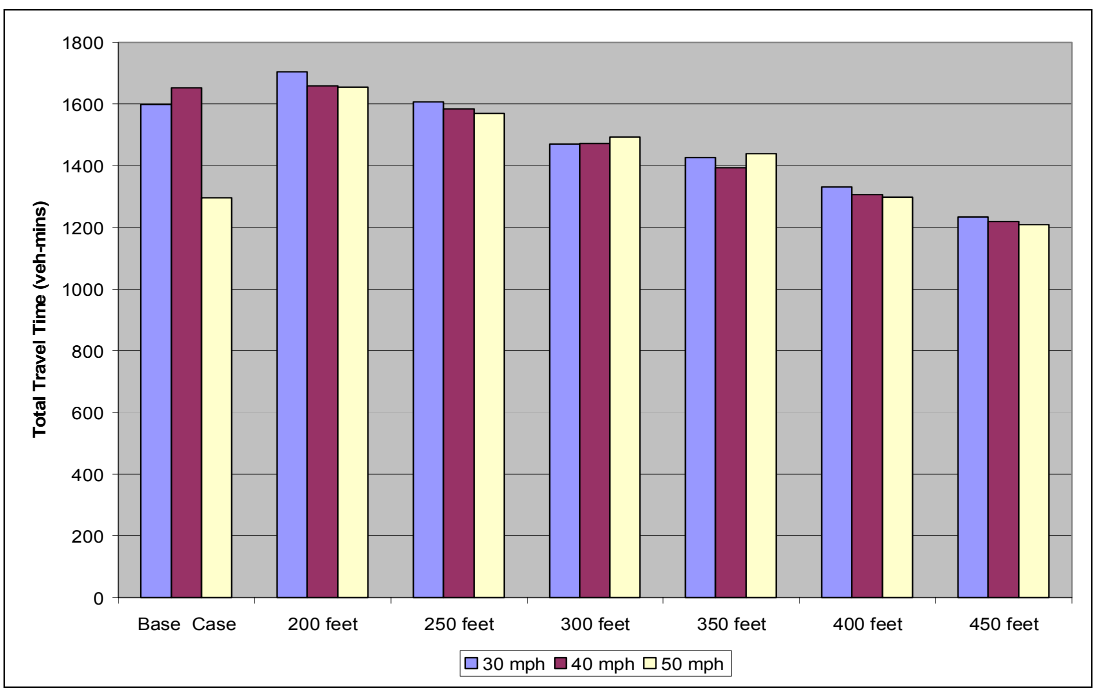

FIGURE 10.19 Total Travel Time for the NB and SB approaches in Scenario C-ii

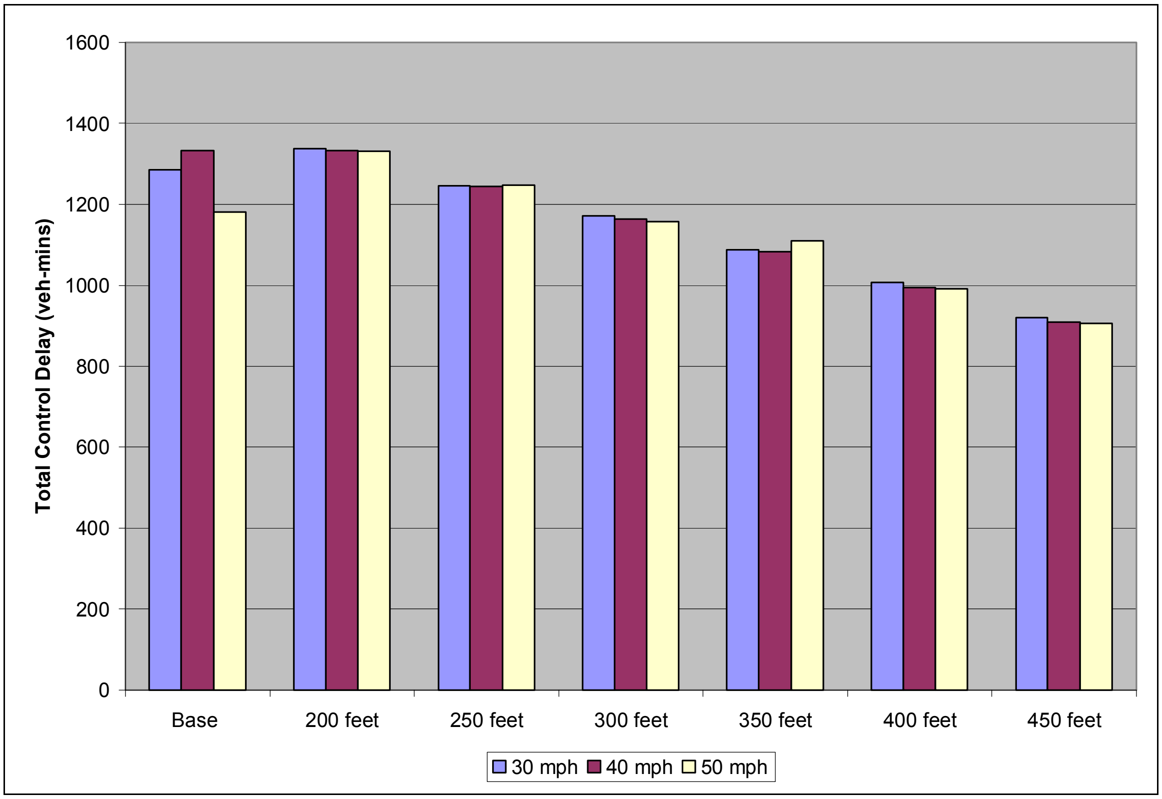

FIGURE 10.20 Total Control Delay for NB and SB approaches in Scenario C-ii

An ANOVA was conducted to assess the influence of speed limit on the reduction in total control delay. The reduction in total control delay was calculated with reference to the 50 mph speed limit for the base case. For both, the actual values of total control delay, and, the reduction with respect to the base case, the F-calculated was found to be less than F-critical suggesting that the speed limits did not significantly impact the total control delay at 95% confidence level.

iii. Input Volume = 3000 vph and speed limit of 30mph, 40 mph and 50 mph

Total Travel Time (veh-mins) and Total Control Delay (veh-mins) for approaches without pre-signals (EB and WB)

Figure 10.21 and Figure 10.22 yet again show that the EB and WB approach failed by a small margin. The reason yet again is the timing plan used for the two cases.

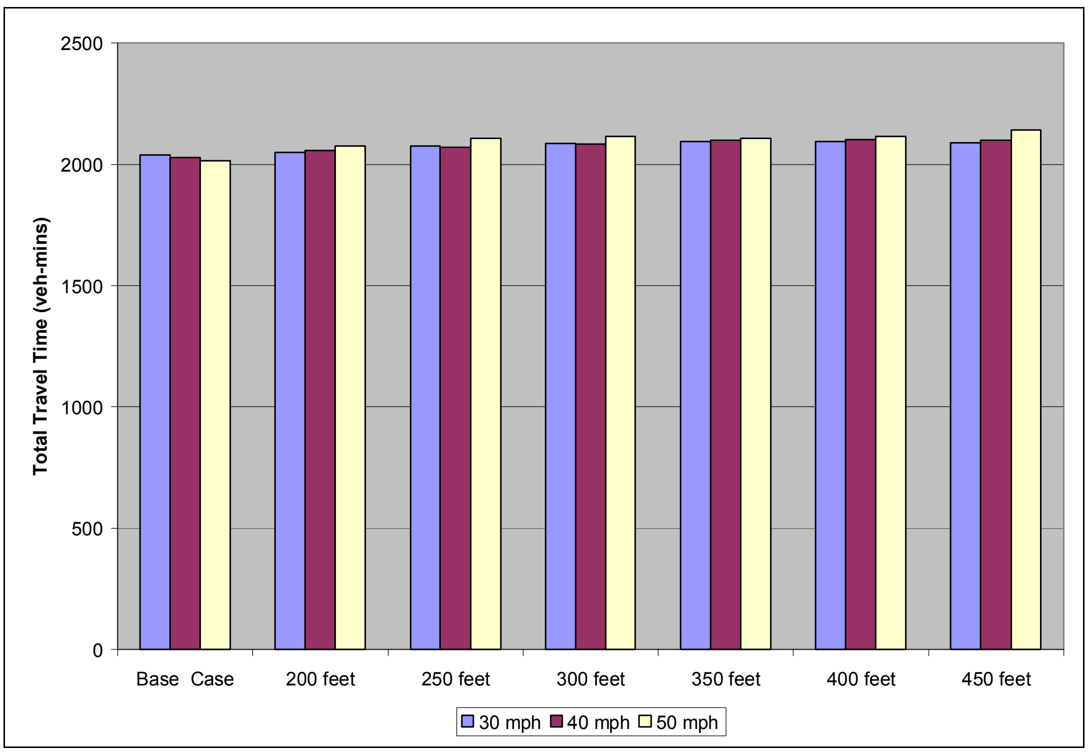

FIGURE 10.21 Total Travel Time for the EB and WB approaches in Scenario C-iii

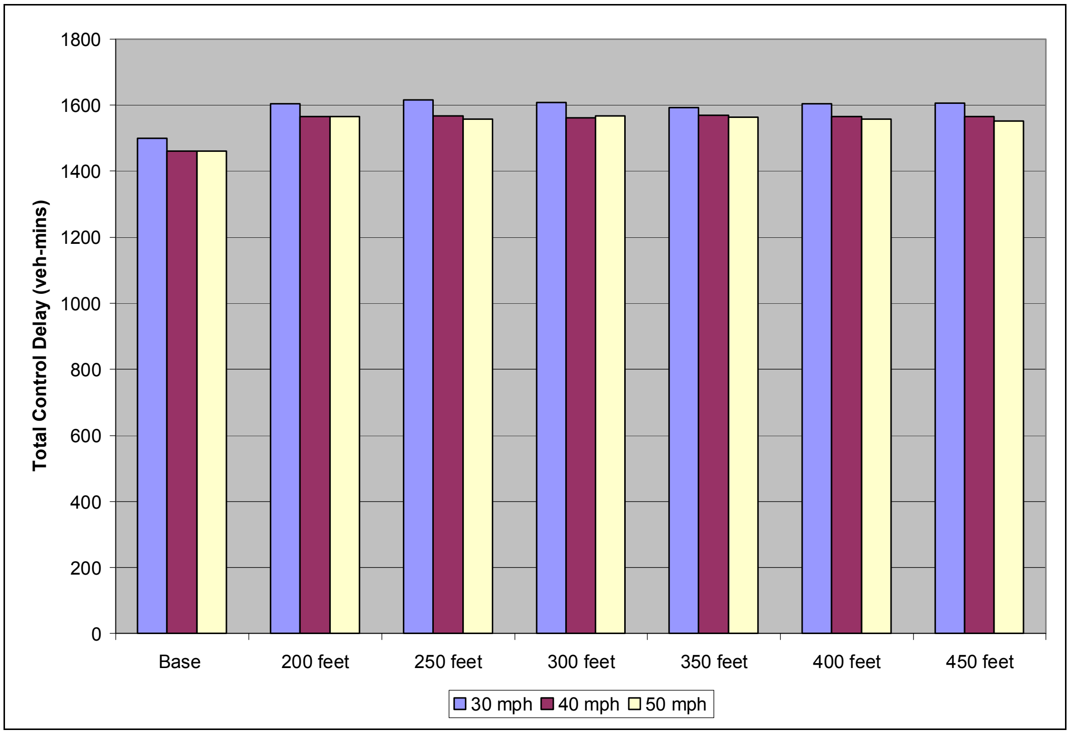

FIGURE 10.22 Total Control Delay for EB and WB approaches in Scenario C-iii

Total Travel Time (veh-mins) and Total Control Delay (veh-mins) for approaches with pre-signals (NB and SB)

Figure 10.23 illustrates that for a demand level of 3000 vph, all the speed limits and all the offset distances yielded benefits. These benefits have been converted into their monetary value in the subsequent section.

Except for the 200 feet offset distance, all other offsets resulted in a reduction in the total control delay (Figure 10.24). ANOVA was used to assess the influence of speed limits on the total control delay, and the reduction in total control delay. The reduction in total control delay was calculated with the baseline case's 40 mph speed limit as reference because it yielded the lowest total control delay. The F-calculated (0.017) was found to be significantly lower than the F-critical

(3.68) thereby conclusively proving that the treatments are not significantly different from each other in terms of reduction in total control delay.

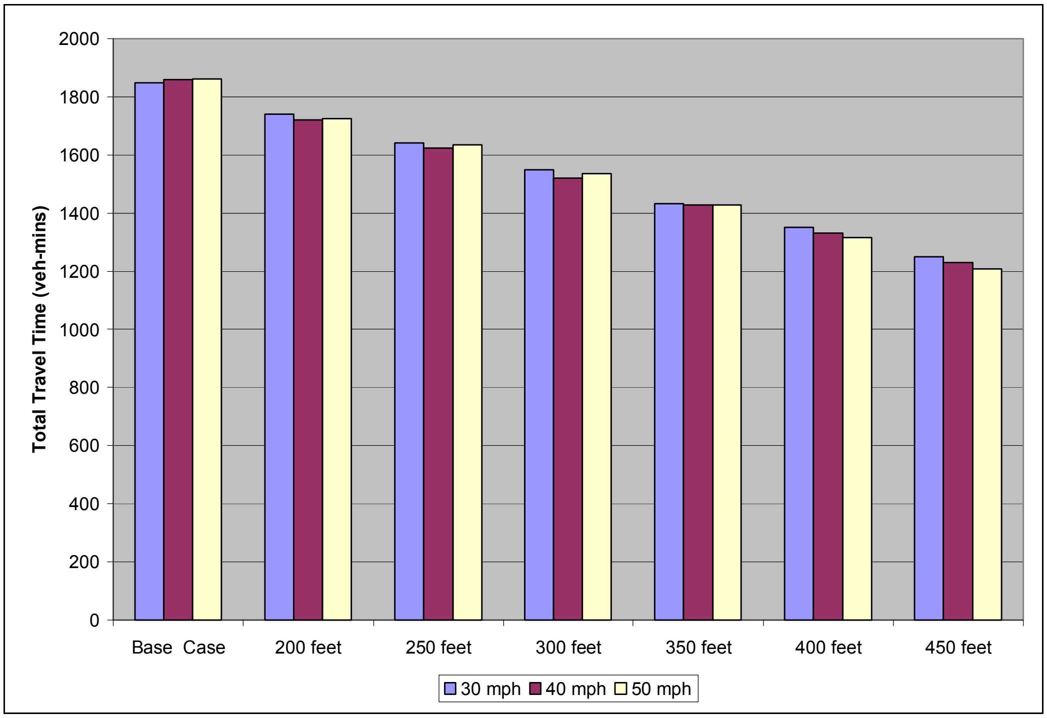

FIGURE 10.23 Total Travel Time for the NB and SB approaches in Scenario C-iii

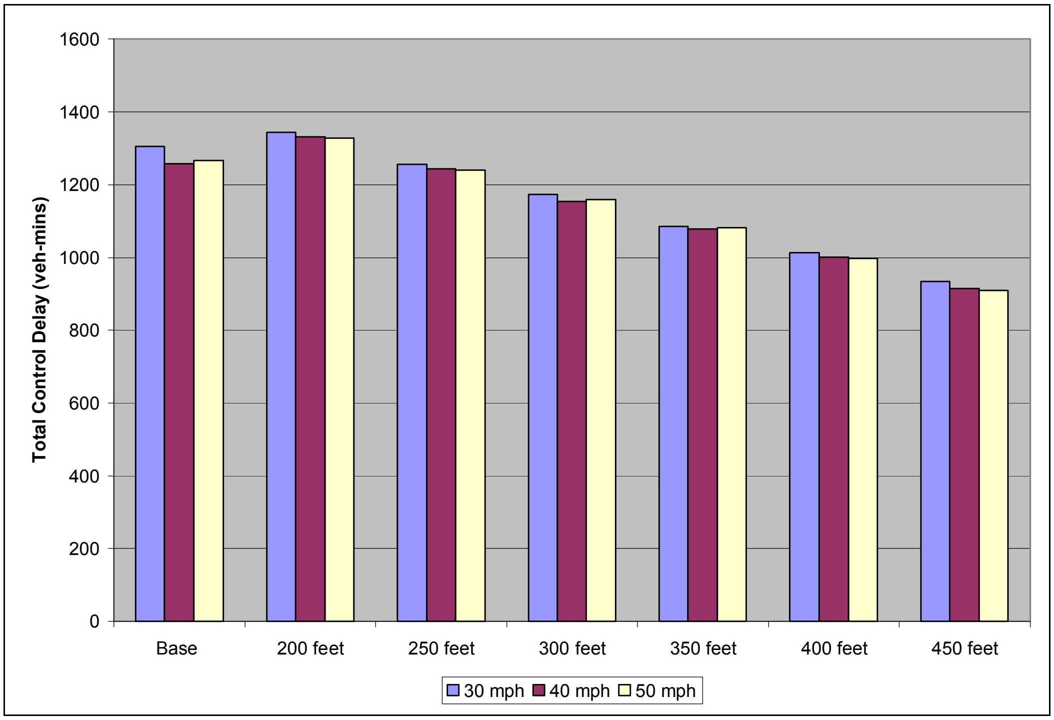

FIGURE 10.24 Total Control Delay for NB and SB approaches in Scenario C-iii

Summary for Case C

The analysis for the previous two configurations (A and B) was repeated to estimate the monetary value of travel time benefits that can be expected from this type of a configuration. Table 10.10 presents travel time savings for approaches with pre-signals (NB and SB) while Table 10.11 presents travel time savings for the approaches without pre-signals (EB and WB). It can be seen that the Table 10.11 does not have any positive values. This could influence the reader to think that the system would incur losses to motorists and hence is not suitable with this configuration. It is therefore important to calculate the net benefits which can be defined as the sum of benefits (negative or positive) obtained from Table 10.10 and Table 10.11. It is to be noted that the table contains positive benefits only for scenarios 22, 23, 26, 27 and 28. Table 10.12 suggests that for the tabulated scenarios, the system as a whole is beneficial only for scenarios 23, 26, 27 and 28 and hence such a configuration should not be ruled out completely. Different phase plan may well result in more benefits for low demand levels also.

TABLE 10.10 Travel Time Savings for Approaches with Pre-Signals ($)-Case C

Offset Distance →	200 Feet	250 Feet	300 Feet	350 feet	400 feet	450 Feet
SCENARIO 19	-1797	-1677	-1543	-1390	-1196	-995
SCENARIO 20	-2262	-2064	-1811	-1560	-1348	-1124
SCENARIO 21	-2405	-2163	-1914	-1693	-1452	-1243
SCENARIO 22	-269	-22	327	439	684	931
SCENARIO 23	-17	173	458	661	885	1106
SCENARIO 24	-916	-700	-500	-364	-4	226
SCENARIO 25	276	528	764	1061	1274	1529
SCENARIO 26	355	608	868	1105	1356	1613
SCENARIO 27	347	576	830	1105	1397	1671

TABLE 10.11 Travel Time Savings for Approaches without Pre-Signals ($)-Case C

Offset Distance →	200 Feet	250 feet	300 Feet	350 feet	400 feet	450 Feet
SCENARIO 19	-5019	-5010	-5052	-5109	-5191	-3970
SCENARIO 20	-5470	-5399	-5314	-5106	-4763	-4432
SCENARIO 21	-5576	-5441	-5419	-5206	-5014	-4757
SCENARIO 22	-1350	-1387	-658	-1466	-1448	-1470
SCENARIO 23	-462	-614	-375	-443	-812	-783
SCENARIO 24	-2318	-2300	-2559	-2580	-2655	-2656
SCENARIO 25	-59	-192	-250	-285	-287	-259
SCENARIO 26	-145	-219	-278	-367	-376	-368
SCENARIO 27	-320	-475	-507	-480	-521	-642

TABLE 10.12 Net Travel Time Savings ($) for Case C

Offset Distance →	200 Feet	250 feet	300 Feet	350 feet	400 feet	450 feet
SCENARIO 19	-6817	-6687	-6595	-6499	-6387	-4965
SCENARIO 20	-7732	-7463	-7125	-6665	-6110	-5556
SCENARIO 21	-7981	-7604	-7334	-6898	-6466	-6000
SCENARIO 22	-1619	-1408	-332	-1028	-764	-539
SCENARIO 23	-478	-441	83	219	73	323
SCENARIO 24	-3234	-3000	-3059	-2944	-2659	-2429
SCENARIO 25	216	336	514	776	986	1269
SCENARIO 26	210	388	590	738	980	1245
SCENARIO 27	26	102	323	625	876	1029

D. Pre-signals on one approach

The SB approach was coded with a pre-signal for this configuration with the intent of evaluating the feasibility of having only one approach with a pre-signal.

i. Input Volume = 2000 vph and speed limits of 30 mph, 40 mph and 50 mph.

   Total Travel Time (veh-mins)

   It can be observed from Figure 10.25 that the pre-signal could not generate any travel time benefits. With the evaluation of this configuration, it is quite clear that the system was not able to generate enough benefits for a low input volume of 2000 vph.

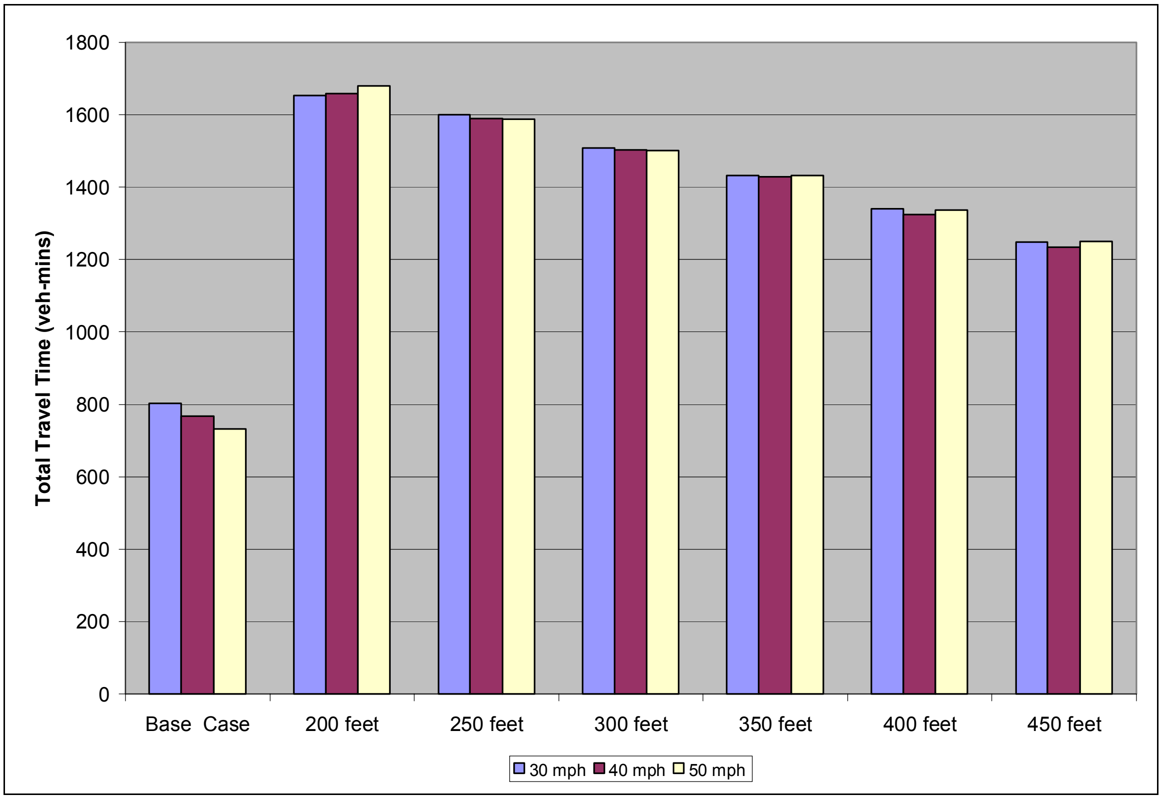

FIGURE 10.25 Total Travel Time for Case D-i

Total Control Delay (veh-mins)

The total control delay also exhibits a trend similar to one observed for total travel time (Figure 10.26). It can now be said that the system may not after all be an option for this volume level, assuming that CORSIM models the queue discharge characteristic accurately, which actually is not the case because of the way in which drivers react to a signal head irrespective of the indication. No statistical test was required as the system was not generating benefits which will naturally push the B/C ration below 1.

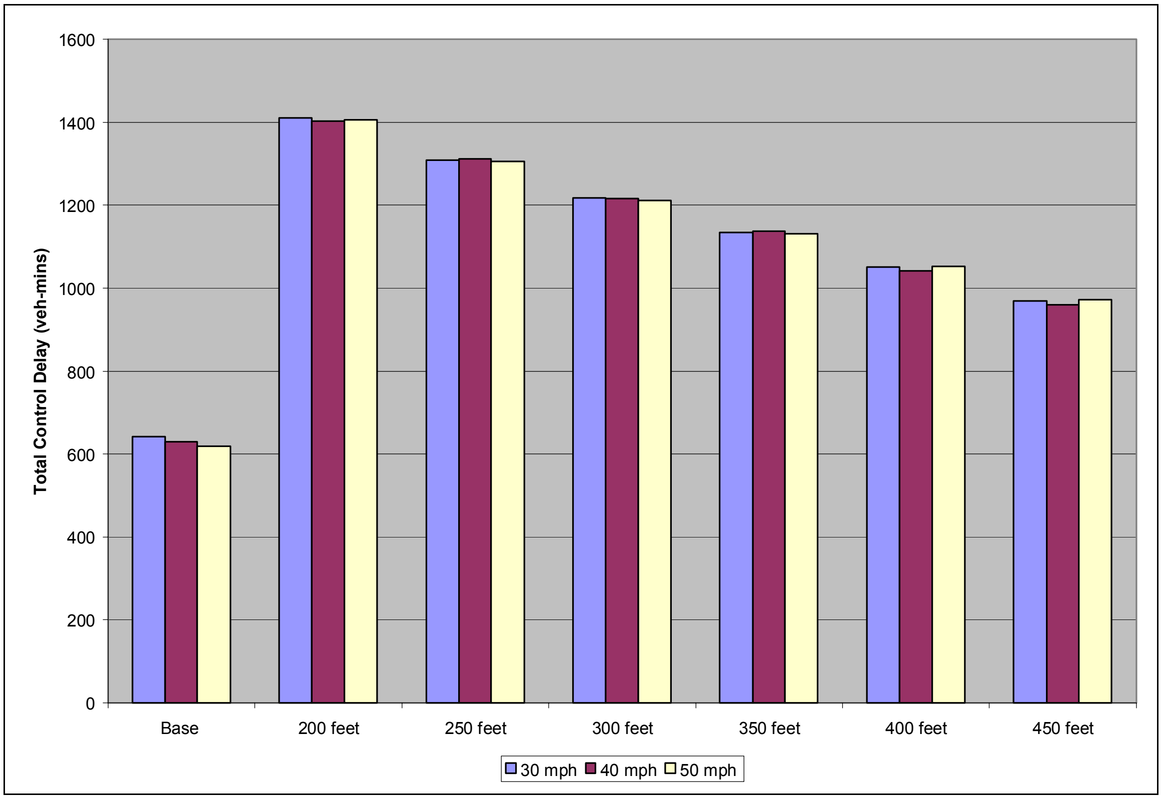

FIGURE 10.26 Total Control Delay for Case D-i

ii. Input Volume = 2500 vph and speed limits of 30 mph, 40 mph and 50 mph.

Total Travel Time (veh-mins)

Figure 10.27 illustrates that the 30 mph and 40 mph speed limits were able to generate travel time savings for offsets greater than 250 feet. The 50 mph speed limit however failed to yield any benefits. A more detailed analysis of travel time is done in economic evaluation section.

Total Control Delay (veh-mins)

Figure 10.28 presents the variation in total control delay with respect to the base line case. It can be seen that for the 30 mph and 40 mph speed limits the benefits start showing up after from the 250 feet offset while the benefits for 50 mph speed limit were realized only for the 450 feet offset. However, an ANOVA on these results yet again suggests that the difference in benefits was not significantly different from each other at 95% confidence level as F-calculated (0.014) for both the actual total control delay and the reduction in total control delay was less than F-critical (3.68). The reduction in total control delay is presented below in Table 10.13.

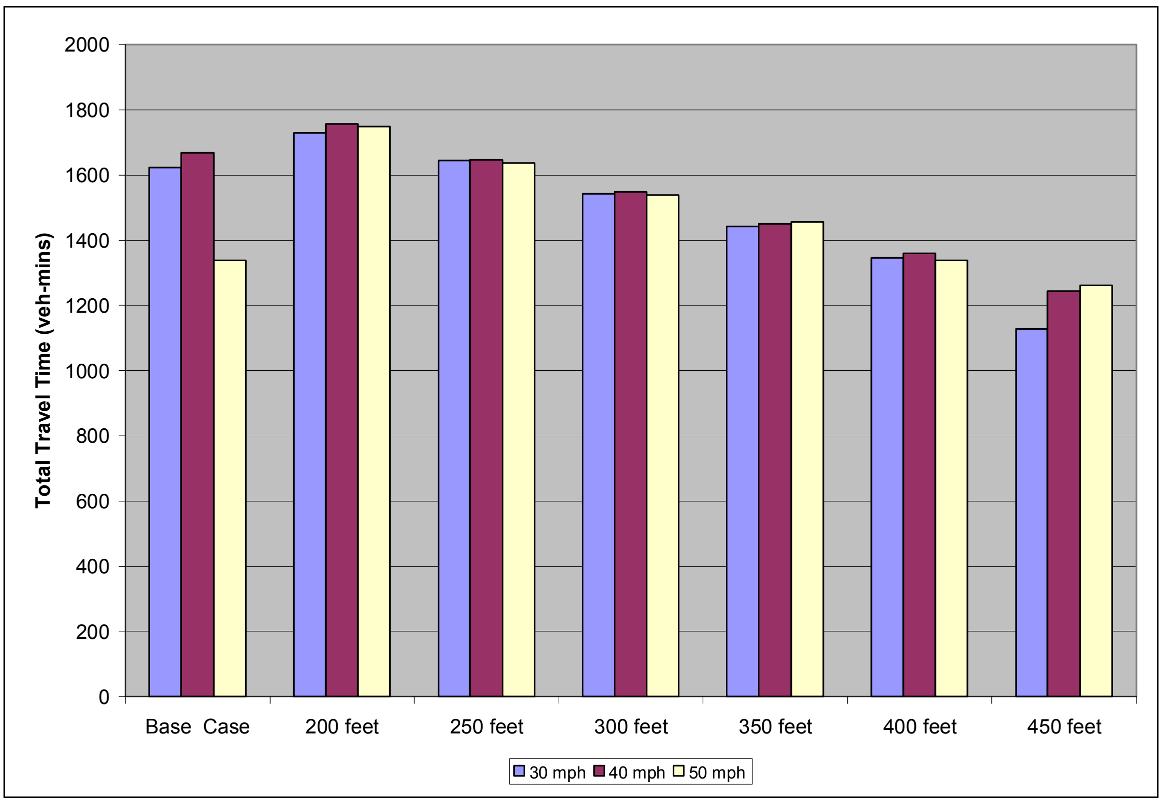

FIGURE 10.27 Total Travel Time for Case D-ii

TABLE 10.13 Reduction in Total Control Delay for Case D-ii

Offset Distance →	200 feet	250 feet	300 feet	350 feet	400 feet	450 feet
30 mph	-153	-65	18	96	179	355
40 mph	-145	-63	21	106	179	264
50 mph	-160	-67	27	98	188	251

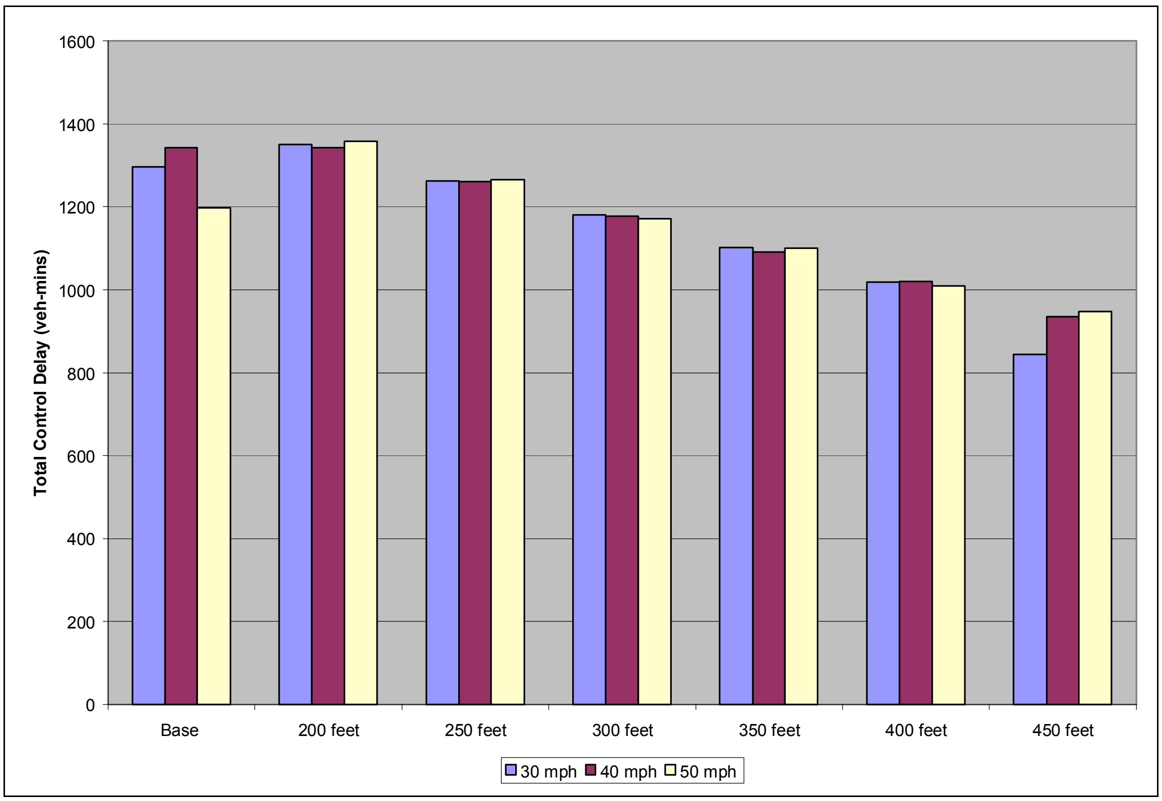

FIGURE 10.28 Total Control Delay for Case D-ii

iii. Input Volume = 3000 vph and speed limits of 30 mph, 40 mph and 50 mph.

Total Travel Time (veh-mins)

As the Figure 10.29 illustrates, this scenario saw comprehensive travel time benefits irrespective of the offset distance. However, maximum benefits were realized for the 450 feet offset for all speed limits. The 40 mph and 50 mph speed limits were quite close in terms of travel time benefits and were better than the 30 mph speed limit.

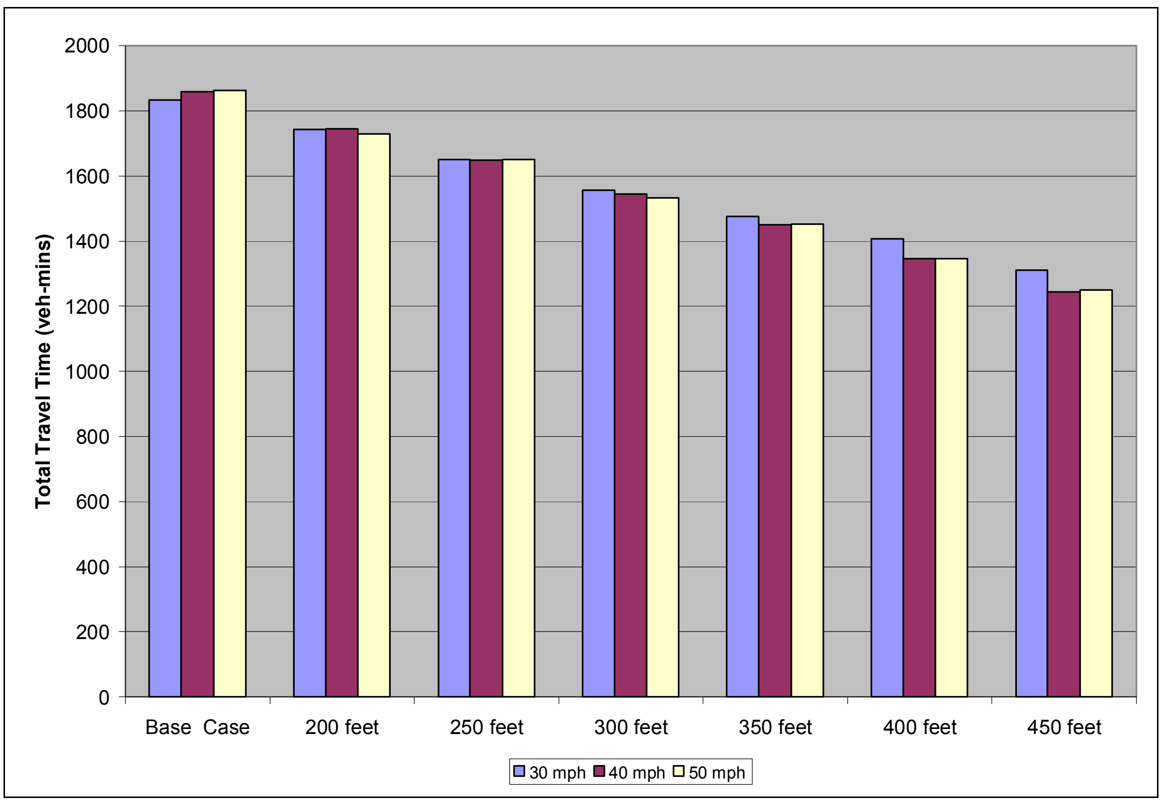

FIGURE 10.29 Total Travel Time for Case D-iii

Total Control Delay (veh-mins)

Control delay for these scenarios was larger than the base case for the 200 feet offset (Figure 10.30). All the offsets greater than 200 feet yielded benefits with the 40 mph speed limit giving maximum reduction in control delay for an offset distance of 450 feet. ANOVA for this case also indicated that changing speed limits does not generate benefits which are significantly different from each other. The F-calculated was found to be 0.94 while F-critical was found to be 3.68.

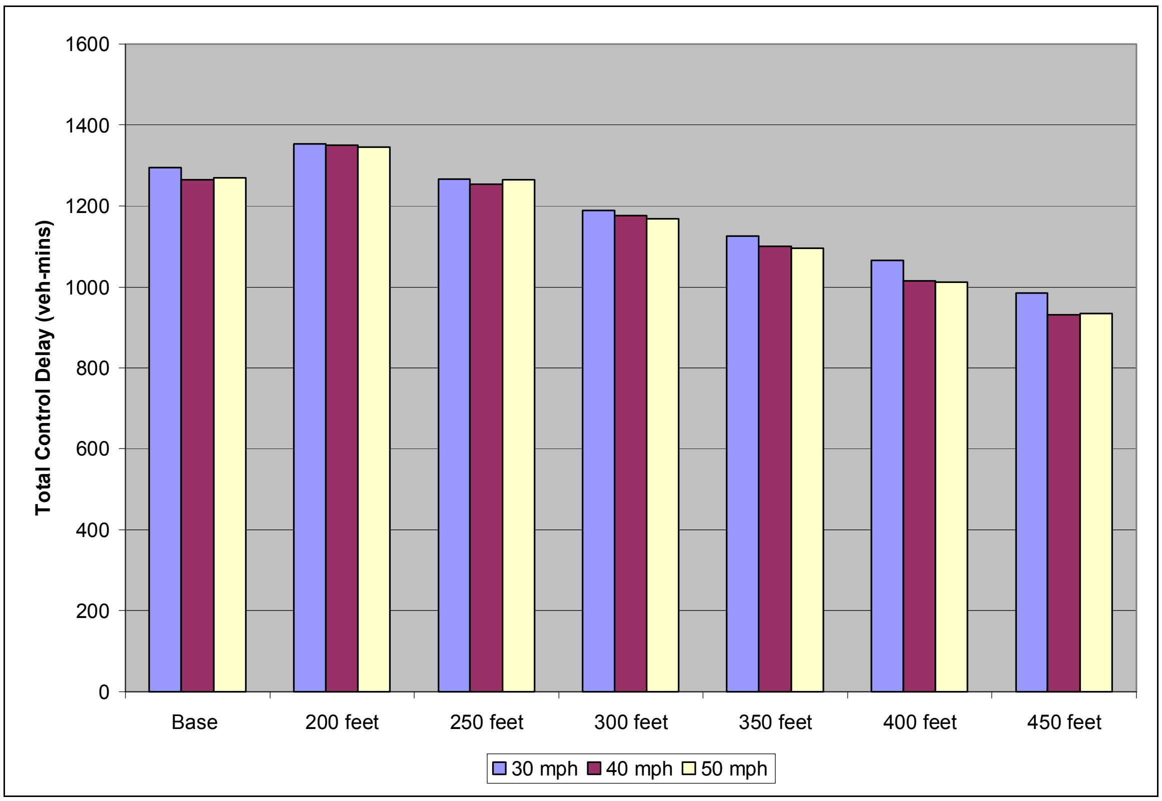

FIGURE 10.30 Total Control Delay for Case D-iii

Summary for Case D

The total travel time benefits (negative or positive) for the three approaches without the pre-signals are tabulated in Table 10.14. Similarly, Table 10.15 presents the travel time savings for the SB approach (with pre-signal). It can be seen that the approaches without the pre-signal suffered heavy travel time losses while the approach with the pre-signal benefited from it. The disbenefits on approaches without pre-signal can be attributed to the fact that the base case had lead-lead type signal phasing while the pre-signal case had split phasing. It therefore becomes important to consider the net benefits rather than just benefits on approaches with pre-signals. The net benefits have been presented in Table 10.16. The Table indicates that the benefits observed were reduced heavily due to the losses on the three approaches. The decision to go in for this configuration would then boil down to the weighted priority given to each approach. However, it is clearly visible that for most of the cases, the system as a whole is a success.

TABLE 10.14 Travel Time Savings for Approaches without Pre-Signal ($)- Case D

Offset Distance →	200 Feet	250 Feet	300 feet	350 feet	400 feet	450 Feet
SCENARIO 28	-7228	-7621	-7461	-7646	-7296	-7537
SCENARIO 29	-8001	-8251	-8073	-7892	-7933	-7788
SCENARIO 30	-8368	-8342	-8277	-8164	-7999	-8052
SCENARIO 31	-1584	-1145	-658	-187	339	1042
SCENARIO 32	-2705	-2609	-2679	-2619	-2662	-2670
SCENARIO 33	-5589	-5357	-5500	-5556	-5290	-5370
SCENARIO 34	-550	-609	-539	-519	-423	-485
SCENARIO 35	-607	-499	-602	-614	-568	-610
SCENARIO 36	-575	-751	-478	-768	-654	-716

TABLE 10.15 Travel Time Savings for Approach with Pre-Signal ($) - Case D

Offset Distance →	200 feet	250 feet	300 Feet	350 Feet	400 feet	450 feet
SCENARIO 28	-2178	-2040	-1804	-1611	-1374	-1141
SCENARIO 29	-2278	-2102	-1882	-1692	-1424	-1194
SCENARIO 30	-2421	-2187	-1964	-1791	-1545	-1323
SCENARIO 31	-267	-53	210	464	712	1264
SCENARIO 32	-224	53	306	558	790	1085
SCENARIO 33	-1051	-762	-512	-299	3	197
SCENARIO 34	233	468	710	917	1094	1337
SCENARIO 35	290	537	805	1044	1312	1573
SCENARIO 36	339	540	846	1051	1324	1567

TABLE 10.16 Net Travel Time Savings ($) for Case D

Offset Distance →	200 feet	250 Feet	300 Feet	350 feet	400 feet	450 feet
SCENARIO 28	-9405	-9661	-9265	-9257	-8669	-8678
SCENARIO 29	-10279	-10353	-9954	-9584	-9357	-8982
SCENARIO 30	-10789	-10529	-10241	-9955	-9544	-9374
SCENARIO 31	-1851	-1198	-448	277	1051	2306
SCENARIO 32	-2930	-2555	-2373	-2062	-1872	-1585
SCENARIO 33	-6640	-6119	-6012	-5855	-5287	-5174
SCENARIO 34	-317	-141	172	398	672	852
SCENARIO 35	-317	39	203	430	743	963
SCENARIO 36	-236	-211	368	283	670	852

GENERAL SCENARIOS

The previous section dealt with basic and non-realistic scenarios. The final test of the system therefore has to be under more real life like scenarios. In this section, four cases were coded in CORSIM for evaluation purpose.

Case 1

The input parameters presented in Table 10.17 were used to code an intersection in CORSIM. The timing plan for the intersection was generated and optimized using Synchro before transferring the timing plan to CORSIM.

As decided based on prior expectations, total travel time and total control delay were used to compare and quantify the benefits of the system with respect to the baseline case. Table 10.18 presents the results of the simulation runs for the baseline case and the system under evaluation.

TABLE 10.17 Input Parameters for Case 1

VARIABLES	EB Approach			WB Approach			SB Approach			NB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	1565			2450			1200			3120		
Turning Movements (%)	6	92	2	8	84	8	10	82	8	5	94	1
Turning Movements (vph)	94	1440	31	196	2058	196	120	984	96	156	2933	31
Proportion of Heavy Vehicles (%)	5			10			20			15		
Speed Limit (mph)	30			35			50			40		
Offset Time (sec)	10			12			16			14		
Offset distance (feet)	204			286			544			381		

TABLE 10.18 Results for Case 1

MOE	BASELINE CASE				CASE 1			
	EB	WB	SB	NB	EB	WB	SB	NB
Total Travel Time (veh-mins)	510	1584	262	1504	492	1379	301	1107
Total Control Delay (veh-mins)	346	1337	183	988	320	1045	193	741

As done previously, the savings in travel time were converted into its monetary value to get a clear idea on the amount of savings that can be expected from such a system for the given parameters.

Total Savings = (Savings/15-min period) x 4 x (Average Vehicle Occupancy) x (Average Value of travel time savings) x (average peak hours per day)

= {(18 + 205 – 39 + 397) *4 * 1.2 * 8 * 4 / 60} = $ 1485 per day.

The savings realized for this scenario appear to be quite significant. It is quite commonly assumed that the cost of installing a signal varies from $100,000 to $200,000. The benefits thus appear to be exceeding the costs by a long way.

All approaches, except that south bound approach showed benefits as far as travel time savings were concerned. The reason behind the systems failure cannot be attributed entirely to the low demand level. It is actually a combination of low demand & low speed limit added together with a proportion of heavy vehicles which can be termed as a little high than the average. These factors probably led to the failure of the proposed system for the SB approach. The presence of heavy vehicles adversely affected the speed gradient between the leading vehicles and following vehicles in the queue.

CORSIM was configured to run 10 simulations of 15 minutes each. The output file contained the records of delay for each interval of 1 minute. The increase in control delay from the first minute to the 15th minute for the baseline case was compared with the same for the corresponding approach with pre-signal. A paired t-test was used for this evaluation and the results of the statistical tests are illustrated in Table 10.19.

The statistical test thus suggests that the system was able to yield significantly lower control delay vis-à-vis the baseline case for the EB, WB and NB approaches. For the SB approach however, the control delay was significantly higher as compared to the baseline case. Figure 10.31 illustrates the benefits realized on each approach.

TABLE 10.19 Results of Paired t-test for Each Approach for Case 1

Approach	t-calculated	t-critical	Statistical Comments	General Comments
EB	4.7485064	1.7613	Significantly different	Beneficial
WB	5.4760582		Significantly different	Beneficial
SB	-3.1601202		Significantly different	Not Beneficial
NB	8.2860814		Significantly different	Beneficial

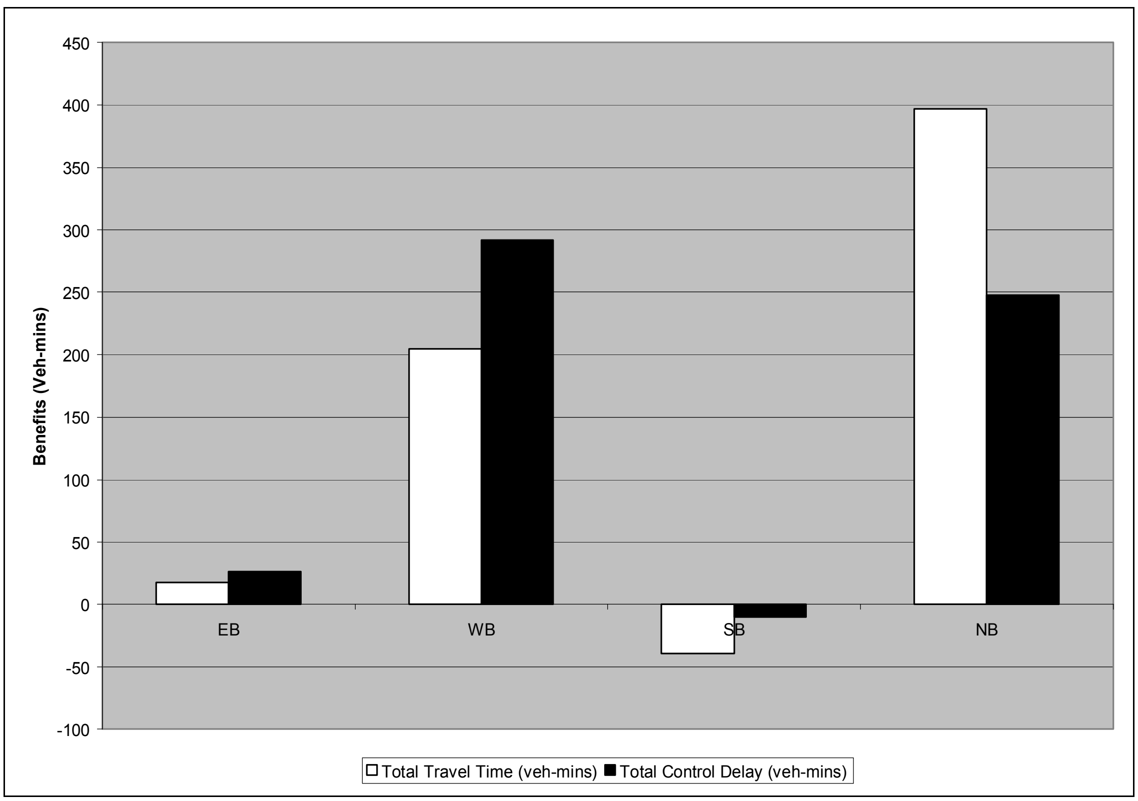

FIGURE 10.31 Reduction in Travel Time and Control Delay for Case 1

Case 2

Table 10.20 describes the parameters used in coding CASE 2. The timing plan was generated and optimized using SYCHRO before transferring it to CORSIM for the actual evaluation of the system.

TABLE 10.20 Input Parameters for Case 2

VARIABLES	EB Approach			WB Approach			SB Approach			NB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	3115			1212			1880			2568		
Turning Movements (%)	14	82	4	6	92	2	9	88	3	13	77	10
Turning Movements (%)	219	1283	63	147	2254	49	108	1056	36	406	2402	312
Proportion of Heavy Vehicles (%)	5			35			15			20		
Speed Limit (mph)	50			40			30			35		
Offset Time (sec)	11			13			17			15		
Offset Distance (feet)	374			354			347			357		

The results obtained from CORSIM have been tabulated in Table 10.21 and the benefits have been presented in a graphical form in Figure 10.32.

TABLE 10.21 Results for Case 2

MOE	BASELINE CASE				CASE 2			
	EB	WB	SB	NB	EB	WB	SB	NB
Total Travel Time (veh-mins)	1914	318	1235	1455	1314	330	1156	1109
Total Control Delay (veh-mins)	1442	225	1142	1083	897	211	910	751

Based on values presented in Table 10.21 above, the monetary value of travel time savings was found to be $ 2594 per day. This figure again is a substantial one and thus strongly suggests that the system would be extremely beneficial for this scenario.

The monetary value of travel time savings was found to be $ 2594 per day. This figure again is a substantial one and thus strongly suggests that the system would be extremely beneficial for this scenario.

It can be seen from Figure 10.32 that the WB approach failed to generate any significant benefits. Looking at the input parameters, it is difficult to identify any specific reason for this case but it appears that the high left turning traffic compounded with a low speed limit and considerable heavy vehicles in the traffic adversely impacted the efficient performance of the system. However, the negative benefits for the WB approach are miniscule compared to the real benefits observed for other three approaches.

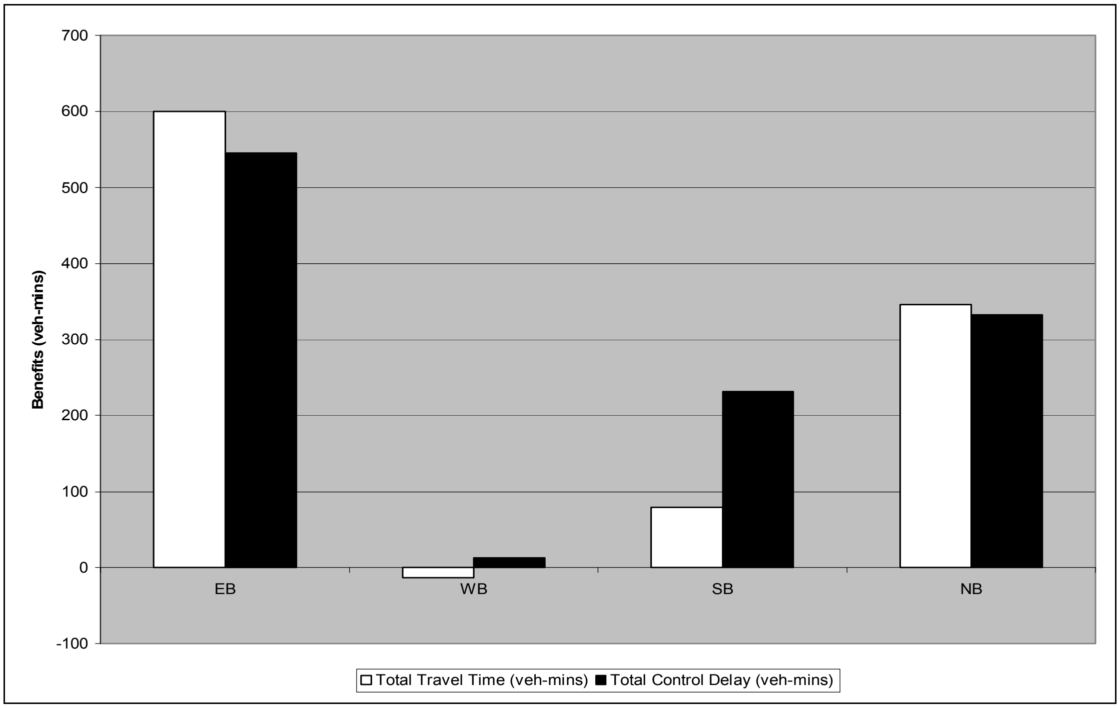

FIGURE 10.32 Reduction in Travel Time and Control Delay for Case 2

A paired t-test was yet again used to compare the change in control delay with each analysis interval in CORSIM simulation. The results of the analysis are presented in Table 10.22 below.

TABLE 10.22 Results of Paired t-test for Each Approach for Case 2

	t-calculated	t-critical	Statistical Comments	General Benefits
EB	7.70	1.76	Significantly Different	Beneficial
WB	6.35		Significantly Different	Not Beneficial
SB	-6.48		Significantly Different	Beneficial
NB	6.93		Significantly Different	Beneficial

The paired t-test suggested that the total control delay for the case 2 approaches (EB, SB and NB) was significantly lower than for the base case approaches. For the WB approach however, the control delay was significantly greater than the control delay for the baseline scenario's WB approach.

Case 3

The input variables for this case are presented in Table 10.23. The CORSIM output was reduced and compiled to compare the aforementioned MOEs.

TABLE 10.23 Input Parameters for Case 3

VARIABLES	EB Approach			WB Approach			SB Approach			NB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	3118			1831			1540			2844		
Turning Movements (%)	10	87	3	7	88	5	9.5	85	5.5	9	86	6
Turning Movements (%)	312	2712	94	128	1611	92	146	1309	85	256	2432	156
Proportion of Heavy Vehicles (%)	8			22			11			17		
Speed Limit (mph)	40			35			40			50		
Offset Time (sec)	17			15			11			13		

The observed benefits for the system were restricted only to EB, WB and SB approaches. In addition, even the WB approach did not have benefits as much as the EB and the NB approaches.

A close examination of Table 10.23 indicates that the reason could well be associated with the demand levels. The WB and SB approaches had low demand as compared to the EB and NB approaches (Table 10.24). This factor certainly seems to have played a big part is the system not generating benefits. Figure 10.33 also provides a hint that probably an input volume between 1540 and 1830 would generate no benefits and no losses, which could be roughly considered as the break-even point for the system. As done previously, the monetary value of travel time savings was calculated and the result was not against expectations owing to the large benefits on two of the approaches. The total travel time savings were found to be $ 2007 per day.

TABLE 10.24 Results for Case 3

MOE	BASELINE CASE				CASE 3			
	EB	WB	SB	NB	EB	WB	SB	NB
Total Travel Time (veh-mins)	1824	730	462	1494	1314	711	577	1125
Total Control Delay (veh-mins)	1327	585	340	1125	902	513	382	774

Evaluation of the other MOE, viz., total control delay was done with the help of a paired t-test for each approach. Table 10.25 presents the summary of the paired t-test study. The control delay for the EB, WB, and NB approaches in the proposed alternative were found to be significantly lower than the control delay for the corresponding approaches in the baseline case. For the SB approach however, the total control delay was significantly higher for the proposed alternative in comparison with the control delay for corresponding approach in the base case.

TABLE 10.25 Results of Paired t-test for Each Approach for Case 3

Approach	t-calculated	t-critical	Comments
EB	7.74	1.76	Significantly Different
WB	5.76		Significantly Different
SB	-7.17		Significantly Different
NB	6.75		Significantly Different

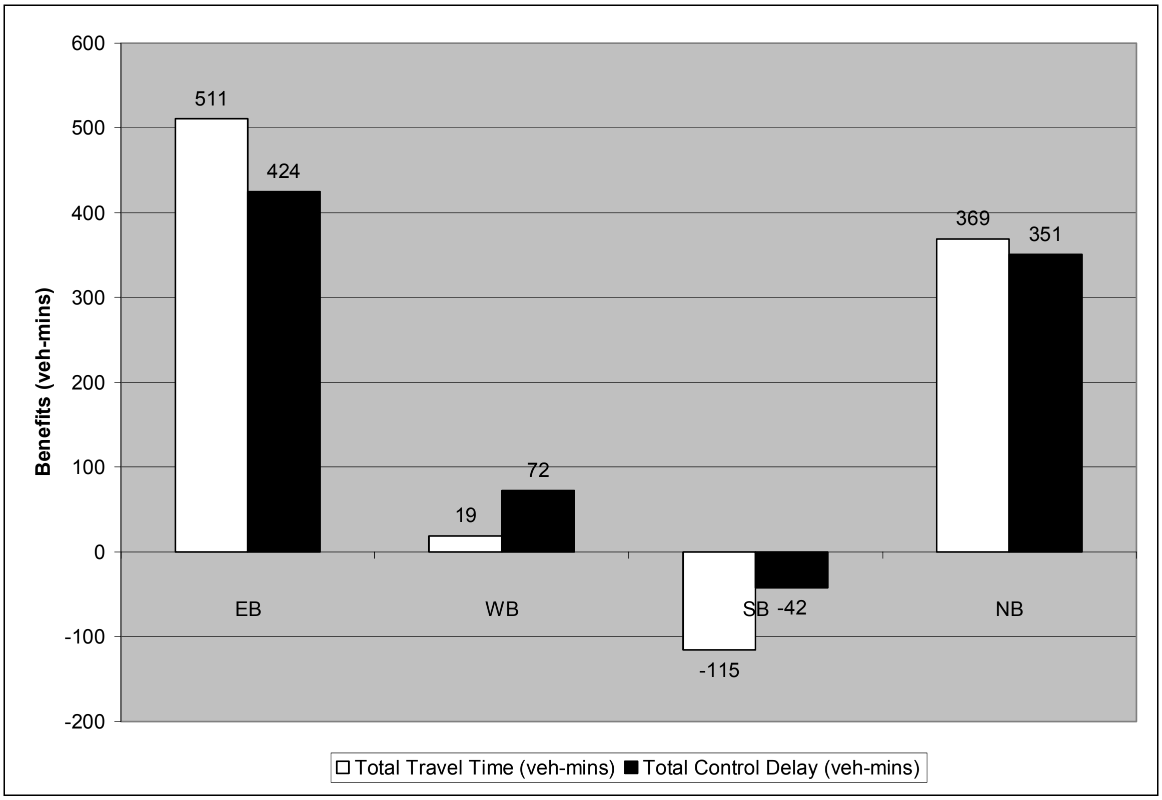

FIGURE 10.33 Reduction in Travel Time and Control Delay for Case 3

Case 4

Table 10.26 presents the input parameters for this scenario and the results of the simulation are presented in Table 10.27.

TABLE 10.26 Input Parameters for Case 4

VARIABLES	EB Approach			WB Approach			SB Approach			NB Approach		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Total Volume (vph)	2500			1522			1710			3450		
Turning Movements (%)	12	85	3	7	90	3	9	87	4	11	81	8
Turning Movements (%)	300	2125	75	107	1369	46	158	1479	73	380	2795	276
Proportion of Heavy Vehicles (%)	9			30			21			14		
Speed Limit (mph)	40			35			30			50		
Offset Time (sec)	16			14			20			8		

TABLE 10.27 Results for Case 4

MOE	BASELINE CASE				CASE 4			
	EB	WB	SB	NB	EB	WB	SB	NB
Total Travel Time (veh-mins)	1683	608	610	1578	1230	704	633	1342
Total Control Delay (veh-mins)	1407	491	443	1017	886	562	436	957

This configuration had two approaches on which travel time savings were not observed. A careful look at Table 10.26 suggests that the reason yet again was low demand. Consistent with the observation that the breakeven point lies somewhere been 1500 vph and 1800 vph, it can be seen from Figure 10.34 that travel time savings for the SB approach were close to 0 which suggests that the given volume is close to the breakeven point. Despite these losses on WB and SB approaches, the net value of travel time savings was found to be equal to $ 1459 per day.

The results from the t-test for evaluating total control delay have been listed in Table 10.28 below. From the table it can be said that the total control delay on EB, SB and NB approaches was significantly lower than that for the corresponding approaches of the baseline case. However, the total control delay on the WB approach of the proposed alternative was significantly higher than that for the baseline case.

TABLE 10.28 Results of Paired t-test for Each Approach for Case 4

Approach	t-calculated	t-critical	Comments
EB	6.72	1.76	Significantly Different
WB	-4.89		Significantly Different
SB	8.73		Significantly Different
NB	9.9		Significantly Different

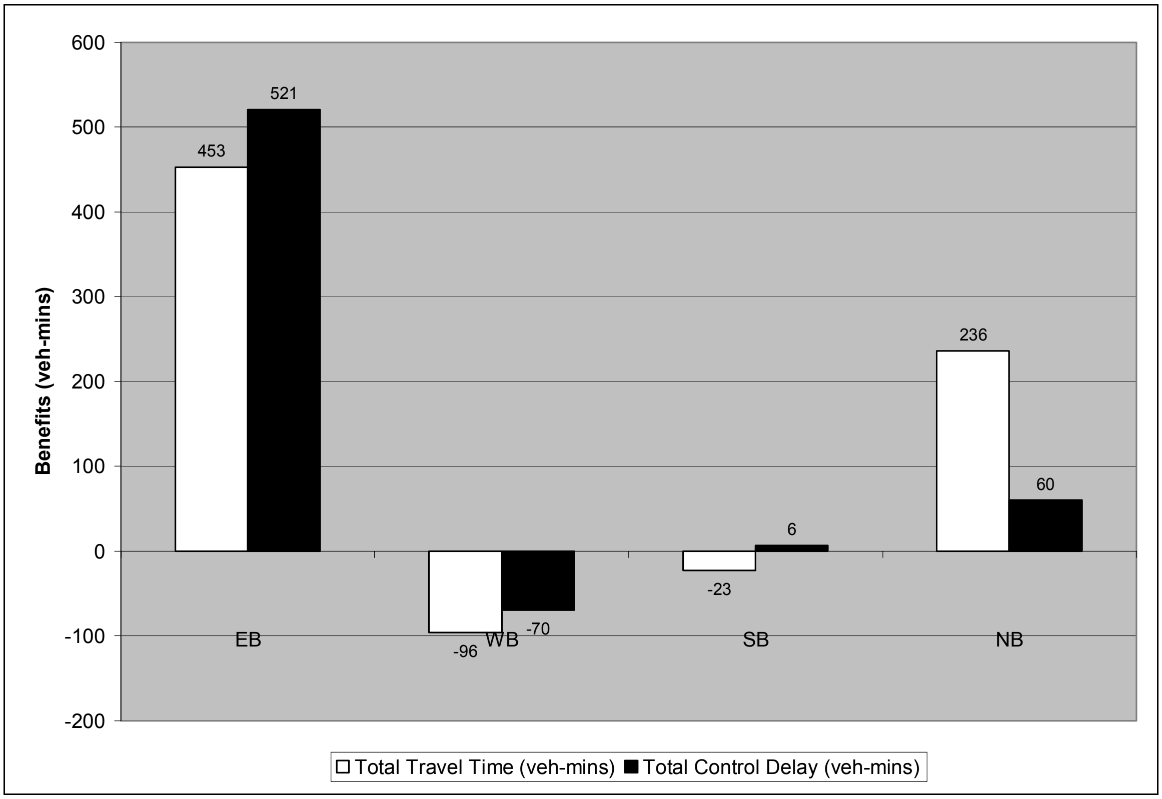

FIGURE 10.34 Reduction in Travel Time and Control Delay for Case 4

Sensitivity Analysis

One of the expectations prior to the commencement of this research study was the notion that the system would be useful for oversaturated intersections. The evaluation of various scenarios (both basic and general) during the course of this study has strongly pointed towards this hypothesis to be true. Figure 10.35 illustrates the impact of demand level (or input volume) on the benefits. Although there were a few aberrations in the general trend, it is quite clear from the figure that real benefits start showing for volumes greater than 1800 vph. The evaluation of basic scenarios had indicated that the breakpoint input volume was approximately 2500 vph. However, that section was restricted to the use of split phasing while in this section, a lag-lag type of phasing sequence was followed.

For an input volume of 1565 vph, marginal benefits can be seen. To understand the reason, it is important to look at other factors. For this approach, the percentage of heavy vehicles was low and the left turning traffic also was less. The combination of these factors probably led to this small change in the trend of benefits vis-à-vis demand level.

Sensitivity analysis of other parameters such as offset distance, offsets times, heavy vehicle proportion, and speed limits failed to give any conclusive evidence of their respective impacts on the realization of benefits and hence are not discussed in this report.

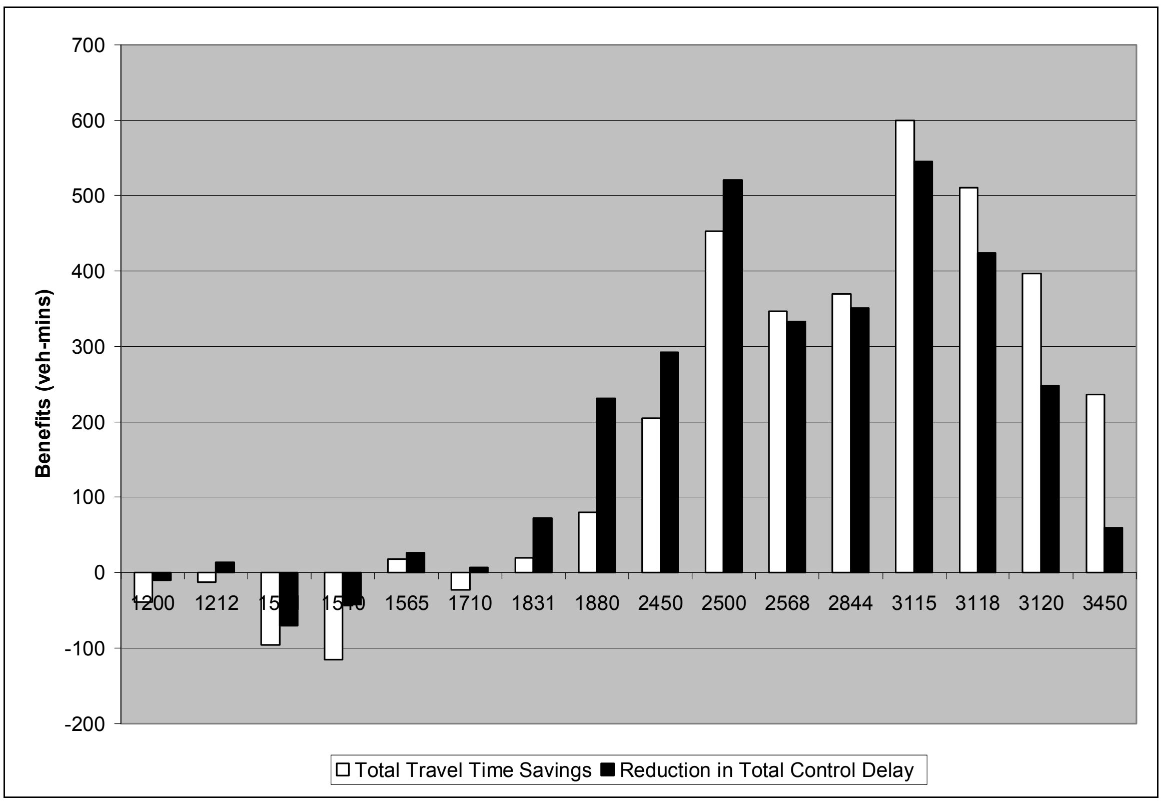

FIGURE 10.35 Impact of Demand on Benefits

Summary

The cases where an approach failed to generate any benefits were short listed and are presented in Table 10.29. It is important to note that the wide difference in the proportion of heavy vehicles between the first two rows (volumes of 1540 vph and 1521 vph) in the Table 10.29 resulted in negative benefits being more for the approach with a lower proportion of heavy vehicles. The reason behind this could be difference in left turning traffic. To validate this premise, TRAFVU simulations were observed and it was observed that left turning traffic would block the through moving vehicles thereby, on occasions, creating a queue at the main intersection. A sensitivity analysis of proportion of turning movements therefore could be a useful analysis but is beyond the scope of this research. Also, hundreds of other combinations can be evaluated but have been intentionally left out due to practical limitations.

TABLE 10.29 Short Listed Cases

Volume (vph)	Speed (mph)	Phv (%)	Offset Distance (feet)	% Left movement	% Through movement	% Right movement	Total Travel Time Savings	Reduction in Total Control Delay
1540	40	11	299	10	85	5	-115	-42
1521	35	30	333	7	90	3	-96	-70
1200	50	20	446	10	82	8	-39	-10
1710	30	21	408	9	87	4	-23	6
1212	40	35	354	6	92	2	-13	13

CHAPTER XI

SUMMARY

FINDINGS

Queue discharge modeling and signal optimization will continue to remain related to each other and the advancement in the former will greatly aid the advancement in the latter. This research study explored one amongst a gamut of possibilities. The findings of this study have been summarized in this section:

- There is no consistency in the models developed so far to describe the queue discharge process. Each model, depending upon where it was developed reveals a new finding. It therefore appears that queue discharge may be highly location specific and models can only be used to obtain rough estimates.
- The queue discharge phenomenon can be described by a linear model with reasonable accuracy. Although literature suggests non-linear models to describe this phenomenon, none of those models took the cumulative time. These models instead depended on either headways between successive vehicles or saturation flows at various points during the green interval.
- An attempt to try and fit the data using, logarithmic & exponential (both negative and positive) indicated that these curves had a poor fit and hence explaining the corresponding low R-square values.
- It was found that the relation between 'T' and 'R' comes close to the rule of thumb used commonly by traffic engineers. The rule of thumb used by traffic engineers to calculate 'T' is, $T= 4 + 2R$ whereas the data collected for this study suggests that the equation should be, $T = 1 + 2.15R$. The advancement is automobile technology can be a possible reason for this reduction in the coefficient from 4 to 1.

- The linear regression model developed in this study had a high R-square value. Moreover, it can be seen from the scatter plot and the normal probability plot that the residuals have a mean of 0 and are approximately normally distributed.
- Unlike other queue discharge models, the model developed in this study takes into account other factors like discharge rate, movement type (through or right), lane width, vehicle type, proportion of heavy vehicles in traffic which are known to have a significant impact on the queue discharge process.
- Using the linear model, it was found that the proposed system has the ability to yield high travel time savings. The theoretical figure of savings was found to be approximately \$ 325 per day which by any standard is a big amount considering that the costs associated with setting up a new signal system varies between \$100,000 and \$200,000.
- A HCM capacity analysis also suggested that the system would yield statistically significant benefits. However, it is important to qualify that statement by limiting the practically significant benefits to oversaturated conditions. It was found that the reduction in control delay were considerable for input volumes greater than 2000 ($v/c \approx 1$).
- This was further confirmed by analysis of simulation results. CORSIM analysis for configuration type A (system on all four approaches) suggested that significant benefits start showing up for volumes equal to and/or greater than 2500 vph ($v/c \approx 1.79$). For this volume level, only the 40 mph and 30 mph speed limits were able to generate significant benefits. However, increasing the volume to 3000 vph ($v/c \approx 2.23$), resulted in realization of benefits for all speed limits and all offset distances. This configuration yielded positive benefits as high as \$ 2500 a day.
- For configuration Type B also, a volume level of 3000 vph generated benefits for all offset distances. For the 2000 vph volume level however, none of the offsets or speed limits were able to generate any significant benefits. This reconfirms the fact that the system will be useful only under oversaturated conditions. It was

found that despite the losses incurred on the approach without the pre-signal, the overall travel time savings ($) had a positive sign.

- For Configuration type C and D also, the absence of pre-signals did not result in travel time losses.
- It was found that changing speed limits did not make any significant difference in the change in total travel control delay. However, it was found that the 40 mph speed limit consistently gave more benefits compared the other two speed limits.
- The general scenarios indicated that the breakeven point for the system was approximately 1800 vph (v/c < 1). This indicates that the performance of the system depends on how the signals are timed because the basic scenarios had only split phasing while the general scenarios had lag-lag type of phasing sequence.
- It was also found that a high proportion of left turning movements coupled with a high proportion of heavy vehicles in the traffic had the capability to cause the system to fail. This is another indication that a proper phasing sequence is of prime importance for the system to perform at its best.
- Based on the analysis of the general scenario cases, no clear evidence could be found on the relation between offset distance, offset times, heavy vehicle proportion, or speed limits, and the system benefits. This aspect however deserves a further detailed investigation.
- The TRAFVU animations indicated that the vehicles departing from the pre-signals tend to slow down on nearing the main signal. This can be attributed to the way CORSIM codes driver behavior. Every driver in CORSIM therefore slows down thereby limiting the benefits seen in this study. However, it can also be argued that probably in real life too, motorists may look at the main signal from a far off location (but downstream of the pre-signal) and start decelerating. The results therefore can be assumed to be close to what should be expected in real life also.

LIMITATIONS

The author realizes that this study has its fair share of limitations as it was impossible to cover every minute detail of the topic covered in this research. It is recommended that the following limitations should be kept in mind while using the results of this study:

- The data used for the regression model is not exactly normal, but is only approximately normal. The model was primarily developed for evaluating the approximate and theoretical benefits of the proposed benefits.
- It should be noted that the model was developed using data which lacked sufficient data points for vehicles placed beyond the 10^{th} vehicle in queue. This was a result of the angles of the video camera used for collecting data.
- The data set also lacked a good representation of the right turning vehicles, buses, trucks and had no representation of the left turning movements.
- The model also suffers from the limitation of not taking into account the impact of leading vehicles on the following vehicles. For example, if the leading vehicle was a right turning vehicle, the time required by the following vehicle to clear the queue will be more compared to if the leading vehicle was a through moving vehicle. The same logic is also applicable to the presence of heavy vehicles in the queue. Moreover, the turning movement distribution is also an important factor which does not feature in the model. However, relevant data could not be collected due to practical limitation resulting in these shortcomings in the model.
- Since the cameras were places at an angle, some personal errors in fixing the stop bar location, might have unknowingly skewed the data.
- The basic scenarios had exactly similar conditions on all approaches which is highly abstract in nature. This was however done to get the average values for the MOEs.
- The general scenarios did not contain an extensive sensitivity analysis to evaluate the impact of offsets, speed limits, proportion of left turners and heavy vehicles

in the traffic. It is almost a certainty that these factors will play an important role in the performance of the proposed system.

- It was not possible to code a lead-lead phasing scheme for the Pre-Signal in CORSIM and hence this phasing sequence was not included in the research. One can expect that results could be different from what they are now.
- The system in itself too has a few limitations. As this concept results in queuing to take place upstream of the actual intersection, it may cause interference with the efficient operation of the upstream intersection. The system's applicability is therefore restricted to isolated intersections only.
- The analysis suggested that this system can only be termed useful under oversaturated conditions.
- There can also be many arguments against the system on the basis of safety. A typical case would be of a vehicle approaching the pre-signal during the yellow indication. Such a motorist may be tempted to accelerate and cross the pre-signal with the knowledge that clearing this signal would most likely enable him/her to clear the main intersection as well. Such practical cases need greater investigation that that done in this research leaving the scope for further studies on this topic.
- The other major safety concern with the system is the lack of a fail-safe option. In the eventuality of main signal failing to turn green in time, some motorists, familiar with the system may not be able to decelerate and stop in time thereby creating dilemma zone issues.
- The system's compatibility with actuated control systems was not investigated in this research. Under actuated control systems, it is expected that the benefits may not be as high as those seen in this study.

RECOMMENDATIONS FOR FUTURE RESEARCH

The realization of the limitations of this study and the system has created numerous opportunities for future research on related topics. Some of the possibilities include:

- Much more can be done to improve the model developed for in this research study Future studies should include the impact of left turning movements in addition to collecting data so has to have a good representation of trucks, buses and right turners. Moreover, impact of driver familiarity, duration of cycle length and impact of downstream intersections can also be considered.
- If other variables except for rank 'R' were ignored, it is still possible to observe a two-regime model with the first regime being non-linear and the second regime being linear in nature. Future studies may investigate this aspect also as it may give the model a different perspective.
- It is recommended that the system be evaluated using a lead-lead phasing sequence in VISSIM to be able to better understand the performance of this system.
- Sensitivity analysis should be carried out to study the impact of various offset distances by keeping other factors constant. Similarly, impact of heavy vehicle percentage in traffic, proportion of left turning and right turning movements should also be studied.
- More complex studies involving the microscopic analysis of how motorists would react to the onset of yellow at the pre-signal could also be a worthwhile research study. Such a study could consider the presence of various vehicle types at various positions and speeds during and prior to the onset of yellow.
- A detailed benefit cost analysis can also be considered to involve all minute costs and benefits of the system.

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APPENDIX I

Time	Vehicle Position	Mvmt	Vehicle Type	Flow rate	Phv	Lane width
2.41	1	0	1	1969	2.9	11
3.97	2	0	1	1969	2.9	11
5.93	3	0	1	1969	2.9	11
8.77	4	0	1	1969	2.9	11
11.55	5	1	1	1969	2.9	11
13.16	6	1	1	1969	2.9	11
1.81	1	0	1	1969	2.9	11
3.83	2	0	1	1969	2.9	11
7.32	3	0	1	1969	2.9	11
9.27	4	0	1	1969	2.9	11
11.03	5	0	1	1969	2.9	11
12.58	6	0	1	1969	2.9	11
3.21	1	0	1	1969	2.9	11
5.53	2	0	1	1969	2.9	11
7.59	3	0	1	1969	2.9	11
9.69	4	0	1	1969	2.9	11
11.99	5	0	1	1969	2.9	11
3.01	1	0	1	1969	2.9	11
5.07	2	0	1	1969	2.9	11
8.68	3	0	1	1969	2.9	11
10.83	4	1	1	1969	2.9	11
12.36	5	0	1	1969	2.9	11
2.39	1	0	1	1969	2.9	11

Time	Vehicle Position	Mvmt	Vehicle Type	Flow rate	Phv	Lane width
5.3	2	0	1	1969	2.9	11
7.5	3	0	1	1969	2.9	11
10.74	4	0	1	1969	2.9	11
12.36	5	0	1	1969	2.9	11
14.58	6	0	1	1969	2.9	11
1.73	1	0	1	1969	2.9	11
4.06	2	0	1	1969	2.9	11
7.49	3	0	1	1969	2.9	11
10.25	4	0	1	1969	2.9	11
11.63	5	0	1	1969	2.9	11
13.99	6	0	1	1969	2.9	11
1.91	1	0	1	1969	2.9	11
4.3	2	0	1	1969	2.9	11
7.02	3	0	1	1969	2.9	11
9.91	4	0	1	1969	2.9	11
11.22	5	0	1	1969	2.9	11
13.21	6	0	1	1969	2.9	11
2.49	1	0	1	1969	2.9	11
4.93	2	1	1	1969	2.9	11
6.9	3	0	1	1969	2.9	11
8.99	4	0	1	1969	2.9	11
13.94	5	0	1	1969	2.9	11
17.65	6	0	2	1969	2.9	11
20.52	7	0	3	1969	2.9	11
22.02	8	0	1	1969	2.9	11
24.91	9	0	1	1969	2.9	11

Time	Vehicle Position	Mvmt	Vehicle Type	Flow rate	Phv	Lane width
1.97	1	0	1	1969	2.9	11
4.81	2	0	1	1969	2.9	11
7.23	3	0	1	1969	2.9	11
9.6	4	0	1	1969	2.9	11
10.78	5	0	1	1969	2.9	11
12	6	0	1	1969	2.9	11
5.13	1	0	1	1969	2.9	11
8.48	2	0	2	1969	2.9	11
10.66	3	1	1	1969	2.9	11
12.14	4	0	1	1969	2.9	11
15.27	5	0	1	1969	2.9	11
1.83	1	0	1	1969	2.9	11
4.72	2	0	1	1969	2.9	11
7.48	3	0	1	1969	2.9	11
9.92	4	0	1	1969	2.9	11
11.71	5	0	1	1969	2.9	11
2.69	1	0	1	1969	2.9	11
5.12	2	0	1	1969	2.9	11
7.87	3	0	1	1969	2.9	11
9.55	4	0	1	1969	2.9	11
11.09	5	0	1	1969	2.9	11
12.87	6	1	1	1969	2.9	11
14.09	7	0	1	1969	2.9	11
15.66	8	0	1	1969	2.9	11

APPENDIX II

	Input Volume (vph)	Speed Limit (mph)
SCENARIO 1	2000	30
SCENARIO 2	2000	30
SCENARIO 3	2000	30
SCENARIO 4	2500	40
SCENARIO 5	2500	40
SCENARIO 6	2500	40
SCENARIO 7	3000	50
SCENARIO 8	3000	50
SCENARIO 9	3000	50
SCENARIO 10	2000	30
SCENARIO 11	2000	30
SCENARIO 12	2000	30
SCENARIO 13	2500	40

	Input Volume (vph)	Speed Limit (mph)
SCENARIO 14	2500	40
SCENARIO 15	2500	40
SCENARIO 16	3000	50
SCENARIO 17	3000	50
SCENARIO 18	3000	50
SCENARIO 19	2000	30
SCENARIO 20	2000	30
SCENARIO 21	2000	30
SCENARIO 22	2500	40
SCENARIO 23	2500	40
SCENARIO 24	2500	40
SCENARIO 25	3000	50
SCENARIO 26	3000	50
SCENARIO 27	3000	50

APPENDIX III

Time (Observed)	Vehicle Position	Mvmt	Phv	Lane width	Vehicle Type	Flow rate	T-predicted	Error
2.05	1	0	0	11	1	1703	2.54534	-0.49534
4.19	2	0	0	11	1	1703	4.69534	-0.50534
6.33	3	0	0	11	1	1703	6.84534	-0.51534
7.93	4	0	0	11	1	1703	8.99534	-1.06534
10.04	5	0	0	11	1	1703	11.14534	-1.10534
12.8	6	0	0	11	1	1703	13.29534	-0.49534
15.62	7	0	0	11	1	1703	15.44534	0.17466
18.64	8	0	0	11	1	1703	17.59534	1.04466
19.93	9	0	0	11	1	1703	19.74534	0.18466

Time	Vehicle Position	Mvmt	Phv	Lane width	Vehicle Type	Flow rate	T-predicted	Error
3.35	1	0	0	11	1	1703	2.4858	0.8642
5.53	2	0	0	11	1	1703	4.6258	0.9042
7.73	3	0	0	11	1	1703	6.7658	0.9642
9.31	4	0	0	11	1	1703	8.9058	0.4042
11.43	5	0	0	11	1	1703	11.0458	0.3842
13.29	6	0	0	11	1	1703	13.1858	0.1042
15.1	7	0	0	11	1	1703	15.3258	-0.2258
17.1	8	0	0	11	1	1703	17.4658	-0.3658
18.67	9	0	0	11	1	1703	19.6058	-0.9358
20.86	10	0	0	11	1	1703	21.7458	-0.8858
23.44	11	0	0	11	1	1703	23.8858	-0.4458

Time	Vehicle Position	Mvmt	Phv	Lane width	Vehicle Type	Flow rate	T-predicted	Error
2.19	1	0	0	11	1	1703	2.4858	-0.2958
4.37	2	0	0	11	1	1703	4.6258	-0.2558
6.41	3	0	0	11	1	1703	6.7658	-0.3558
8.33	4	0	0	11	1	1703	8.9058	-0.5758
10.62	5	0	0	11	1	1703	11.0458	-0.4258
13.61	6	0	0	11	1	1703	13.1858	0.4242
15.79	7	0	0	11	1	1703	15.3258	0.4642
18.37	8	0	0	11	1	1703	17.4658	0.9042
20.37	9	0	0	11	1	1703	19.6058	0.7642

APPENDIX IV

EB Approach Travel Time Savings

Rank	Distance from stop bar	Time to clear stop bar	Savings in travel time (hours)	VOTTS per day per vehicle ($)
1	20	0.5	0.00097	0.00986
2	140	3.2	0.00081	0.00822
3	260	5.9	0.00064	0.00657
4	380	8.6	0.00066	0.00674
5	500	11.3	0.00032	0.00328
6	620	14.1	0.00016	0.00163
7	740	16.8	0.00000	-0.00002
8	860	19.5	-0.00016	-0.00166
9	980	22.2	-0.00032	-0.00331
10	1100	24.9	-0.00031	-0.00314
			TOTAL	0.02817

WB Approach Travel Time Savings

Rank	Distance from stop bar	Time to clear stop bar	Savings in travel time (sec)	VOTTS per day per vehicle ($)
1	20	0.5	0.00147	0.01499
2	140	3.2	0.00131	0.01335
3	260	5.9	0.00115	0.01170
4	380	8.6	0.00116	0.01187
5	500	11.3	0.00100	0.01022
6	620	14.1	0.00066	0.00676
7	740	16.8	0.00050	0.00512
8	860	19.5	0.00034	0.00347
9	980	22.2	0.00018	0.00182
10	1100	24.9	0.00020	0.00199
			TOTAL	0.08129

NB Approach Travel Time Savings

Rank	Distance from stop bar	Time to clear stop bar	Savings in travel time (sec)	VOTTS per day per vehicle ($)
1	20	0.5	0.00197	0.02013
2	140	3.2	0.00199	0.02029
3	260	5.9	0.00165	0.01683
4	380	8.6	0.00167	0.01700
5	500	11.3	0.00133	0.01354
6	620	14.1	0.00117	0.01189
7	740	16.8	0.00100	0.01025
8	860	19.5	0.00084	0.00860
9	980	22.2	0.00068	0.00695
10	1100	24.9	0.00070	0.00712
			TOTAL	0.13261

SB Approach Travel Time Savings

Rank	Distance from stop bar	Time to clear stop bar	Savings in travel time (sec)	VOTTS per day per vehicle ($)
1	20	0.5	0.00248	0.02526
2	140	3.2	0.00231	0.02361
3	260	5.9	0.00215	0.02196
4	380	8.6	0.00199	0.02032
5	500	11.3	0.00183	0.01867
6	620	14.1	0.00167	0.01702
7	740	16.8	0.00151	0.01538
8	860	19.5	0.00135	0.01373
9	980	22.2	0.00118	0.01209
10	1100	24.9	0.00120	0.01225
			TOTAL	0.18029

VITA

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