Final Report

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TRAFFIC CHARACTERISTICS AND ESTIMATION OF TRAFFIC DELAYS AND USER COSTS AT INDIANA FREEWAY WORK ZONES

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CHAPTER 1. INTRODUCTION

It is often necessary to establish work zones on roadways for pavement and bridge repair and rehabilitation activities. It is common knowledge that each year the highway construction season starts with establishing work zones on roadways and that work zones cause traffic delays. Work zone is defined in the 1994 Highway Capacity Manual (TRB 1994) as "an area of highway in which maintenance and construction operations are taking place that impinge on the number of lanes available to moving traffic or affect the operational characteristics of traffic flowing through the area." In order to efficiently plan and schedule work zone operations, it is essential to know the traffic capacity values of work zones. A work zone reduces the available lanes for traffic and therefore causes vehicle deceleration and merging. When traffic flow is below the capacity of a work zone, traffic is delayed primarily by the reduced vehicle speed through the work zone. When traffic flow exceeds the work zone capacity, vehicle queues would form at the work zone and result in additional traffic delays. The 1994 Highway Capacity Manual provides typical capacity values of freeway work zones. As Dixon, Hummer and Lorscheider (1995) indicated these values were obtained using the traffic data on the roadways in Texas and they may not represent the work zone capacities of other states because of different freeway characteristics and driving behaviors. A previous study (Mommott and Conrad 1982) assumes that under congested traffic conditions vehicles travel through a work zone at a flow rate equal to the work zone capacity. This assumption is not accurate because during congestion at work zones traffic flow rates are mostly lower than the work zone capacities. Therefore, characteristics of work zone traffic flows and speeds during congestion are as essential as work zone capacity values in the assessment of work zone traffic delays and user costs.

Highway work zones reduce available lanes for traffic, form traffic bottlenecks, and cause traffic delay and congestion. The additional travel time and change of driving maneuvers at work zones result in excess costs to motorists in terms of the value of time, consumption of fuel and oil, and wearing of vehicle parts. Traffic delays at work zones reflect the work zone impact on traffic flows and are the basis for calculating excess user costs. The Indiana Department of Transportation has been using the user cost estimation model, QUEWZ (Memmott and Dudek 1982), to assess the work zone user costs. The model was developed by the Texas Transportation Institute using the traffic data collected from Texas highways. Because of different environments and driving behaviors, the Texas data may not accurately reflect Indiana's work zone conditions. It was therefore necessary to study the Indiana's work zone impact with Indiana's work zone traffic data. QUEWZ contains equations for estimating traffic delays caused by reduced vehicle speeds and vehicle queues at work zones. However, QUEWZ does not include the traffic delays incurred to motorists when vehicles decelerate from freeway speed to work zone speed before a work zone and when vehicles accelerate from work zone speed to freeway speed right after exiting the work zone. Furthermore, QUEWZ does not consider the vehicle queues caused by the stochastic nature of traffic.

This study was conducted to analyze the traffic flow characteristics of freeway work zones and the associated user costs based on the traffic data collected from Indiana freeways. The work zone capacities, vehicle speeds during congested and uncongested periods, and queue-discharge rates were first determined. Equations for estimating traffic delays at freeway work zones were also developed. Traffic delays at a work zone include the delays caused by vehicle deceleration when approaching the work zone, reduced vehicle speed through the work zone, time needed for vehicles to resume freeway speed after exiting the work zone, and vehicle queues at the work zone. Vehicle queues occur when traffic flow is higher than the traffic capacity of the work zone. Because of the randomness of traffic flow, vehicle queues may also form even when traffic flow is below the work zone capacity. The traffic delay equations were developed in this study for both when the arrival traffic flow is above the work zone capacity and when it is below the work zone capacity. In addition to the equations of work zone traffic delays, several equations of the characteristics of individual vehicle queues were also developed and presented in this report. These equations can be used to estimate the maximum and average queue lengths of a vehicle queue for a given time period, the time needed to clear individual vehicle in a vehicle queue, and the total and average traffic delays of a vehicle queue. Based on the traffic delay equations, a model was developed for calculating the excess user costs at work zones.

Traffic congestion occurs at a work zone when traffic flow exceeds the capacity of the work zone. Consequently, during congestion vehicles go through the work zone at reduced speeds and with fluctuated traffic flow rates. Motorists endure considerably greater traffic delays at the work zone under congested traffic conditions than under uncongested conditions. The ability of dynamically predicting traffic flow rates with real-time data is essential for highway engineers to maintain smooth traffic flows at work zones. It would enable them to apply traffic control measures to prevent traffic congestion at work zones rather than to deal with traffic problems after traffic congestion already occurred. Methods of adaptive forecasting of traffic flow have been explored by many researchers. Ahmed and Cook (1982) applied the time series methods to provide short-term forecast of traffic occupancies for incident detection. Okutani and Stephanedes (1984) employed the Kalman filtering theory in dynamic prediction of traffic flow. Davis et al. (1990) used pattern recognition algorithms to forecast freeway traffic congestion. Lu (1990) developed a model of adaptive prediction of traffic flow based on the least-mean-square algorithm.

As part of the effort to improve traffic control at work zones, the time series theory and the Kalman filtering theory were applied in this study to adaptively predict traffic flow at the work zones on Indiana's freeways with real-time data. It was found that using the Kalman predictor in combination with the autoregressive process of time series could provide satisfactory dynamic predictions of work zone traffic flow. As traffic capacity values of Indiana's freeway work zones were determined in this study, a prediction of traffic flow also constitutes a prediction of traffic congestion. If the predicted traffic flow rate is equal to or greater than the traffic capacity, a traffic congestion is expected in the coming time period and appropriate traffic control actions can be taken to prevent the traffic congestion.

The model of dynamic traffic flow and congestion prediction is also presented in this report.

CHAPTER 2. TRAFFIC CAPACITIES, SPEEDS, AND QUEUE-DISCHARGE RATES AT WORK ZONES

2.1. Data Collection

Traffic data at select work zones on interstate highway sections was collected between October 1995 and April 1997. Traffic counters with road tubes were used for data collection. Traffic volume, speed, and vehicle classification were recorded at 5-minute intervals during high traffic volume hours and at 1-hour intervals during low traffic volume hours. Eight work zones on four-lane interstate highways and two work zones on six-lane interstate highways were selected for traffic data collection. At each of the work zones, traffic data was recorded for two to three days. The traffic data showed that four of the eight four-lane freeway work zones experienced traffic congestion during the data collection. Traffic data was recorded with the traffic counters on a six-lane freeway work zone on I-69 near Fort Wayne, however, the data did not catch any traffic congestion. It was planned the INDOT LaPorte District would use the installed Autoscope video cameras to record the traffic data on a six-lane freeway work zone on the Borman Expressway (I-80/I94 between the Illinois/Indiana board to I-65). However, because the installed devices were never operational as promised, the LaPorte District personnel did not collect or provide the work zone traffic data as planned. Therefore, traffic capacity values and other characteristics could be analyzed and determined only for work zones on four-lane freeways. Two types of work zones in Indiana on four-lane divided highways are shown in Figures 2.1 and 2.2. They are defined as follows (FHWA 1989):

1. Partial Closure (or single lane closure) - when one lane in one direction is closed, resulting in little or no disruption to traffic in the opposite direction.

2. Crossover (or two-lane two-way traffic operations) - when one roadway is closed and the traffic which normally uses that roadway is crossed over the median, and two-way traffic is maintained on the other roadway.

At each work zone, traffic counters were placed before the work zone transition area, within the transition area, and within the activity area. Thus, the recorded traffic data includes free flow traffic (uninterrupted by work zone), merging traffic, and work zone traffic. Traffic flow rate, vehicle speed and classification were recorded at 5-minute intervals during high traffic volume hours and at 1-hour intervals during low traffic volume hours. The vehicle counters were set up to classify the detected vehicles into three groups: 1). passenger cars, 2) heavy trucks and 3) buses. Figure 2.3 illustrates a typical layout of traffic counters at a work zone.

2.2 Work Zone Capacity

As can be seen in Figures 2.1 and 2.2, the partial closure work zone disrupts traffic in only one direction and the crossover work zone affects traffic in both directions (the median crossover direction and the opposite direction). However, the crossover work zone allows the construction crew to work on two lanes and also provides a safer work area because the work area is separated from traffic.

As defined in the 1994 Highway Capacity Manual, the capacity of a highway facility is "the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions." Previous studies applied different methods to identify capacities of freeway work zones. The Texas Transportation Institute (TTI) identified work zone capacity as the hourly traffic volume under congested traffic conditions (Dudek and Richards 1982). A Pennsylvania study defined the hourly traffic volume converted from the maximum recorded 5-minute flow rate as the work zone capacity. The North Carolina study (Dixon et al. 1995) defined work zone capacity as "the flow rate at which traffic behavior quickly changes from uncongested conditions to queue conditions" and used speed-flow curve to identify the capacity value. This author believes that among these different definitions the North Carolina's definition is most close to the general definition of capacity given by the 1994 Highway Capacity Manual. It is because it utilized the flow rate under 'prevailing conditions' (before traffic was congested) as the work zone capacity, while other methods used or included congested traffic flow rates (queue-discharge rates) in work zone capacity.

It was observed that traffic flows at Indiana freeway work zones changed from uncongested to congested conditions always with a sharp speed drop. Therefore, work zone capacity is defined in this study as "the traffic flow rate just before a sharp speed drop followed by a sustained period of low vehicle speed and fluctuated traffic flow rate." To express work zone capacity in passenger car per hour, the traffic flow rate was converted to hourly volume and the adjustment factors from the 1994 Highway Capacity Manual were used to convert trucks and buses to passenger car equivalents. To identify capacity values, traffic flow and speed data points were plotted in order of time into one graph. For example, Figure 2.4 shows such a graph that plotted a one-day traffic flow and speed data at a work zone on I-69, where traffic flow values were divided by 10 for easy comparison. As indicated in the figure, the capacity value is identified as the traffic flow rate (1590 passenger cars per hour) just before the sharp speed drop, from 54 mph (87 km/hour) to 29 mph (45 km/hour), and a long period of traffic congestion (low vehicle speed and fluctuated flow rate).

With this method, the traffic capacity values at the four freeway work zones were identified as presented in Table 2.1. As shown in the table, congestion in work zones could start within a work zone as well as in the transition areas. To compare the capacity values of different work zones, an analysis of variance (ANOVA) (Neter et al. 1985) was conducted on the work zone capacity data. The ANOVA is to test whether or not mean capacity values are the same in the four work zones:

H_0: $\mu_1 = \mu_2 = \mu_3 = \mu_4$

Ha: not all μ_i are equal

Where μ_i are the mean traffic capacity values at work zone i. Suppose a Type I error is controlled at $\alpha = 0.05$, then $F(0.95, 3, 8) = 4.07$ with 3 and 8 as the degrees of freedom associated with the factor level and the error term of the given data in Table 2.1. The decision rule is thus:

If $F^* \leq 4.07$, conclude H_0

If $F^* > 4.07$, conclude Ha

As part of the ANOVA test, the Bartlett test (Neter et al. 1985) of variance homogeneity was performed. The purpose of the Bartlett test was to determine if the work zone capacity values had statistically equal variances as assumed by the ANOVA model. If the test showed that the variances were statistically unequal, the data must be transformed in some way to improve the variance homogeneity before conducting ANOVA. Anderson and McLean (1974) proposed that if the homogeneity test is accepted at α=0.01 level, then the data does not need to be transformed for the ANOVA test. The Bartlett test on the capacity values resulted in a p-value of 0.0166, which is greater than α=0.01. Therefore, the homogeneity test was accepted at α=0.01 level and a data transformation was not needed for the ANOVA test.

Using the data from Table 2.1 and the Statistix (Analytical 1996) statistical program, the ANOVA test statistic was calculated:

$F^* = MSTR/MSE = 34085/41965 = 0.81$

Since $F^* = 0.81 < 4.07$, it is concluded that the mean capacity values of the four work zones are statistically equal. The mean capacity values and standard deviations for the four work zones are:

Work Zone	Mean Capacity (passenger cars/hour)	Standard Deviation
Zone #1	1537	242.21
Zone #2	1745	268.69
Zone #3	1612	28.54
Zone #4	1521	5.66

The data in Table 2.1 shows that the work intensities were different in these work zones. In the partial closure work zones (Zone #1 and Zone #4), construction work was performed in the lane adjacent to the traffic lane. However, in the crossover work zones (Zone #2 and Zone #3), construction work was performed in the areas separated from the traffic lanes. Three levels of work intensities were observed in the four work zones, i.e., medium intensity (Zone #1), work not adjacent to traffic (Zone #2 and Zone #3), and high intensity (Zone #4). The Bartlett test of variance homogeneity was also performed on the capacity data grouped according to the three levels of work intensities. The Bartlett test yielded a p-value of 0.0819, which is greater than α=0.01. According to Anderson and McLean (1996), the homogeneity test was accepted at α=0.01 level and a data transformation was not needed for the ANOVA test. An ANOVA was then conducted to test whether the mean work zone capacities were the same for different work intensities. The ANOVA yielded: $F^* = 0.88 < F(0.95, 2, 9) = 4.26$, which indicates that the mean work zone capacities are statistically equal for the three levels of work intensities. The mean capacity values and standard deviations are listed below.

Work Intensity	Mean Capacity (passenger cars/hour)	Standard Deviation
Medium	1537	242.21
Non-Adjacent	1688	203.50
High	1521	5.66

Although the ANOVA tests indicated that the mean capacity values are statistically equal for different work zone types and work intensities, there indeed exist some differences in the individual mean values and confidence intervals. The lower mean value of the partial closure work zones might be attributed to the influences of the work activities in the work area adjacent to the traffic lane. Because of the statistics equality, the mean capacity values could be combined into one single value based on the principles of statistics. However, to reflect the minor differences in the capacities of the four types of work zones, the individual values (capacity means and confidence intervals) of the four work zone types are presented in Table 2.2 as the typical capacities of the work zones on Indiana four-lane freeways. It should also be pointed out that these capacity values are one-directional capacities. Because a crossover work zone affects the traffic flows in both directions, it may cause greater traffic disruptions and delays (the total of two directions) than a partial closure work zone that affects the traffic flow in only one direction.

2.3. Queue-Discharge Rates and Vehicle Speeds

Of interest are the values of vehicle speed and flow rate at work zones before and during traffic congestion. Under uncongested traffic conditions, vehicle speed at a work zone remains relatively stable with minor fluctuations and vehicles pass through the work zone smoothly without formation of vehicle queues. Figure 2.4 shows a plot of a consecutive 48-hour uncongested traffic flow and speed at a work zone on I-65.

It is clearly shown in the figure that vehicle speed was consistently stable throughout the 48 hours as traffic flow changed cyclically from daytime high to nighttime low. The mean speed for the 48 hours is 56 mph (90 km/hour) with a standard deviation of 2.4. This small standard deviation indicates that vehicle speeds remained within a close range from the mean speed of 56 mph (90 km/hour) during the 48-hour time period.

When traffic is congested at a work zone, vehicle speed remains low and inconsistent and traffic flow rate changes irregularly. Figures 2.5 and 2.6 are two examples of work zone traffic flow and speed patterns during congestion. It can be seen in the two figures that during congestion the vehicle speeds at both work zones were considerably lower than the work zone speed limit of 55 mph (88.5 km/hour). However, the behaviors of traffic flows were quite different at the two work zones. Traffic flow at the I-65 work zone remained consistently below capacity with relatively small fluctuations, while that at the I-70 work zone had values both above and below capacity with significant fluctuations. Traffic flow rate at a work zone during congestion is actually the rate of the queued vehicles being discharged from the work zone. Therefore, it is called the queue-discharge rate of the work zone.

The average queue-discharge rate and the average vehicle speed during congestion are important input for estimating traffic delays and user costs. Based on the collected traffic data at the Indiana freeway work zones, several statistical values of traffic characteristics are calculated and presented in Table 2.3. As given in the table, under uncongested traffic conditions the means of vehicle speeds were about the same for the four types of work zones with mean speed values of 56.27 mph (90.6 km/hour), 56.93 mph (91.6 km/hour), 58.51 mph (94.2 km/hour) and 57.34 mph (92.3 km/hour). Under congested traffic conditions, the means of vehicle speeds were 25.45 mph (41 km/hour) for crossover (in the opposite direction), 25.24 mph (40.6 km/hour) for crossover (in the crossover direction), 31.46 mph (50.6 km/hour) for partial closure (with the right lane closed), and 38.58 mph (62.1 km/hour) for partial closure (with the left lane closed). Compared with those under uncongested conditions, the vehicle speeds under congested conditions had larger standard deviations, which indicates that vehicle speeds vary more under congested traffic conditions. Under congested traffic conditions, the mean traffic flow rates (queue-discharge rates) were respectively 1393, 1587, 1216, and 1374 passenger cars per hour for the four types of work zones. All of these queue-discharge rates are lower than their corresponding work zone capacity values. This indicates that although during congestion traffic flow rate could be occasionally higher than the work zone capacity, the average flow rate remained below the work zone capacity. For easy reference, the mean values of work zone capacity, queue-discharge flow rate and congested vehicle speed are listed into one table (Table 4) with the values being rounded to whole numbers.

As can be seen in Table 2.4, among the four types of work zones, the crossover (in the crossover direction) has the largest value of mean queue-discharge rate. In addition, as shown in Table 2.3, the queue-discharge rates (congested traffic flow) for the crossover (in the crossover direction) also has the smallest standard deviation, or the least traffic fluctuations. Therefore, the crossover (in the crossover direction) has relatively smoother traffic flow under congested traffic conditions than the other three types of work zones. Compared to the two partial closure work zones, the two crossover work zones have higher capacities and queue-discharge rates; however, they also have lower mean speeds during congestion. The differences between the values of mean capacity and mean queue-discharge rate are 352 for crossover (opposite direction), 25 for crossover (crossover direction), 321 for partial closure (right lane closed), and 147 for partial closure (left lane closed). The values of these differences correspond to drops of traffic flow rates of 20.2%, 1.6%, 20.9%, and 9.7%. These percentages indicate that traffic congestion at work zones could result in minor (1.6%) as well as considerable (20.9%) reduction of traffic flow rates. The drops of mean vehicle speeds caused by congestion are 31 mph (49.9 km/hour) (55.4%), 32 mph (51.5 km/hour) (56.1%), 28 mph (45.1 km/hour) (47.5%), and 18 mph (29 km/hour) (31.6%), respectively, for the four work zones. It is apparent that traffic congestion exerts more significant impact on vehicle speeds than on traffic flow rates.

The data shows that the crossover work zone in the opposite direction has a higher mean capacity but a lower mean queue-discharge rate than in the crossover direction. A similar result can also be seen in the partial closure work zone with the right lane closed and with the left lane closed. This phenomenon can be explained by the values of the standard deviations of the corresponding congested traffic flow rates shown in Table 2.3. The standard deviation of congested traffic for the crossover work zone in the opposite direction is 437.70, while the value in the crossover direction is 158.90. The standard deviation is 471.10 for the partial closure work zone with the right lane closed and is 159.00 with the left lane closed. Therefore, the larger fluctuations of traffic flow resulted in the lower queue-discharge rates.

2.4. Summary

It was found that traffic congestion at work zones was characterized by sustained low vehicle speeds and fluctuated traffic flow rates. Therefore, work zone capacity is defined in this study as the traffic flow rate just before a sharp speed drop followed by a sustained period of low vehicle speed and fluctuated traffic flow rate. Based on this definition, the capacity values can be identified on the graph of traffic flow and speed data in order of time series. In addition to traffic capacities, this chapter also discussed and provided the mean queue-discharge rates and vehicle speeds for both uncongested and congested traffic conditions. These values can be used as a basis of predicting traffic congestion, estimating traffic delays and analyzing user costs at work zones. The study results indicated that the mean queue-discharge rates at Indiana freeway work zones were lower than the work zone capacities, even though at times individual queue-discharge rates could be higher than capacities. Therefore, it is not justified to use work zone capacity values, instead of queue-discharge rates, in estimating traffic delays and user costs under congested conditions. Vehicle speeds at work zones under uncongested conditions remained stable and close to the given work zone speed limit of 55 mph (88.5 km/hour). The drops of traffic flow rates caused by traffic congestion ranged from 1.6% to 20.9%, while the drops of vehicle speeds ranged from 31.6% to 56.1%.

Table 2.1. Work Zone Capacity Data

Work Zone , Type, and Location	Capacity (Vehicles/Hour)	Heavy Vehicle Percent	Equivalent Capacity (Passenger Cars/Hour)	Construction Type and Work Intensity	Congestion Starting Location
Zone #1 - Partial Closure (Right Lane Closed) on I-65 N. of SR-32	1500	25	1689	Bridge Rehabilitation, Medium Intensity	Transition Area
Zone #1 - Partial Closure (Right Lane Closed) on I-65 N. of SR-32	1572	12	1665	Bridge Rehabilitation, Medium Intensity	Transition Area
Zone #1 - Partial Closure (Right Lane Closed) on I-65 N. of SR-32	1190	11	1258	Bridge Rehabilitation, Medium Intensity	Transition Area
Zone #2 - Crossover (In Opposite Direction) on I-70 E. of SR-9	1823	39	2142	Pavement Overlay, Not Adjacent to Traffic	Within Work Zone
Zone #2 - Crossover (In Opposite Direction) on I-70 E. of SR-9	1475	22	1598	Pavement Overlay, Not Adjacent to Traffic	Transition Area
Zone #2 - Crossover (In Opposite Direction) on I-70 E. of SR-9	1595	10	1672	Pavement Overlay, Not Adjacent to Traffic	Transition Area
Zone #2 - Crossover (In Opposite Direction) on I-70 E. of SR-9	1386	6	1566	Pavement Overlay, Not Adjacent to Traffic	Transition Area
Zone #3 - Crossover (In Median Crossover Direction) on I-69 S. of SR-332	1404	28	1601	Pavement Overlay, Not Adjacent to Traffic	Within Work Zone
Zone #3 - Crossover (In Median Crossover Direction) on I-69 S. of SR-332	1536	7	1590	Pavement Overlay, Not Adjacent to Traffic	Within Work Zone
Zone #3 - Crossover (In Median Crossover Direction) on I-69 S. of SR-332	1488	21	1644	Pavement Overlay, Not Adjacent to Traffic	Within Work Zone
Zone #4 – Partial Closure (Left Lane Closed) on I-69 at SR-14	1308	32	1517	Bridge Rehabilitation, High Intensity	Within Work Zone
Zone #4 – Partial Closure (Left Lane Closed) on I-69 at SR-14	1320	31	1525	Bridge Rehabilitation, High Intensity	Within Work Zone

Table 2.2. ANOVA Results of Work Zone Capacities (in Passenger Cars/Hour)

Work Zone Type	Mean	Standard Deviation	95% Confidence Interval
Crossover (Opposite Direction)	1745	268.7	(1500, 1970)
Crossover (Crossover Direction)	1612	28.5	(1350, 1860)
Partial Closure (Right Lane Closed)	1537	242.3	(1275, 1800)
Partial Closure (Left Lane Closed)	1521	5.7	(1175, 1850)

Source	***Degree of Freedom***	***SS***	***MS***	***F****	***P-Value***
Zone-Type	3	102254	34085	0.81	0.522
Error	8	335716	41965		
Total	11	437971			

Table 2.3. Summarized Traffic Flow and Speed Data at Work Zones

Work Zone Type	Traffic	Minimum Observed Value	Maximum Observed Value	Mean	Standard Deviation
Crossover (Opposite Direction)	Uncongested Speed	39.31 mph (63.3 km/h)	69.66 mph (112 km/h)	56.27 mph (90.6 km/h)	3.78
	Congested Speed	5.39 mph (8.7 km/h)	44.86 mph (72.2 km/h)	25.45 mph (41 km/h)	7.54
	Congested Traffic Flow (cars/hour)	240	2127	1393	437.70
Crossover(Crossover Direction)	Uncongested Speed	36.09 mph (58.1 km/h)	69.82 mph (112 km/h)	56.93 mph (91.6 km/h)	3.52
	Congested Speed	4.68 mph (7.53 km/h)	42.08 mph (67.7 km/h)	25.24 mph (40.6 km/h)	8.17
	Congested Traffic Flow (cars/hour)	1184	1877	1587	158.90
Partial Closure (Right Lane Closed)	Uncongested Speed	39.60 mph (63.7 km/h)	79.70 mph (128 km/h)	58.51 mph (94.2 km/h)	4.03
	Congested Speed	1.90 mph (3.1 km/h)	51.80 mph (83.4 km/h)	31.46 mph (50.6 km/h)	14.00
	Congested Traffic Flow (cars/hour)	5	1909	1216	471.10
Partial Closure (Left Lane Closed)	Uncongested Speed	42.50 mph (68.4 km/h)	63.30 mph (102 km/h)	57.34 mph (92.3 km/h)	3.08
	Congested Speed	21.60 mph (34.8 km/h)	50.50 mph (81.3 km/h)	38.58 mph (62.1 km/h)	7.25
	Congested Traffic Flow (cars/hour)	973	1715	1374	159.00

Table 2.4. Mean Values of Work Zone Capacities, Queue-Discharge Rates and Vehicle Speeds

Work Zone Type	Mean Capacity (passenger cars/hour)	Mean Queue-Discharge Rate (passenger cars/hour)	Mean Speed During Uncongestion	Mean Speed During Congestion
Crossover (Opposite Direction)	1745	1393	56 mph (90 km/hour)	25 mph (40 km/hour)
Crossover (Crossover Direction)	1612	1587	57 mph (92 km/hour)	25 mph (40 km/hour)
Partial Closure (Right Lane Closed)	1537	1216	59 mph (95 km/hour)	31 mph (50 km/hour)
Partial Closure (Left Lane Closed)	1521	1374	57 mph (92 km/hour)	39 mph (63 km/hour)

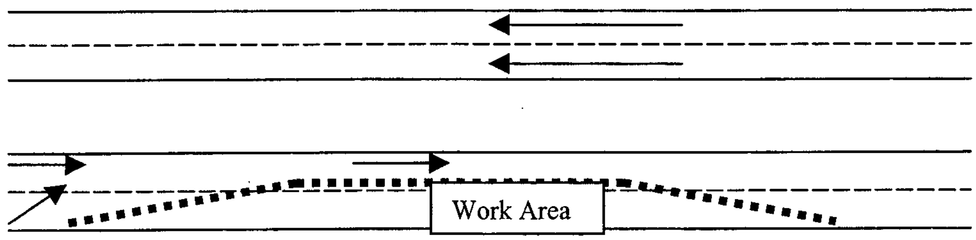

Figure 2.1. Partial Closure Work Zone

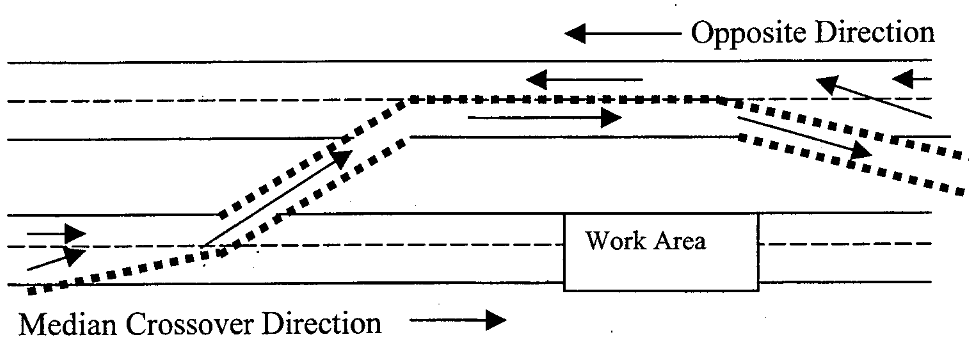

Figure 2.2. Crossover Work Zone

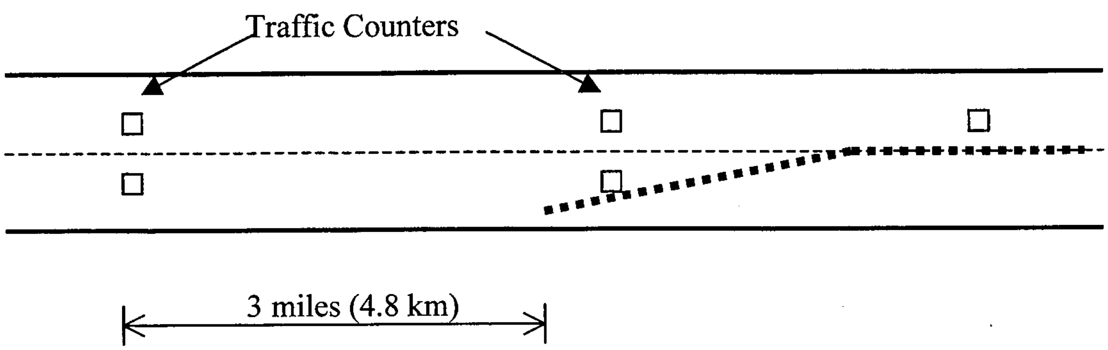

Figure 2.3. Layout of Traffic Counters for Data Collection

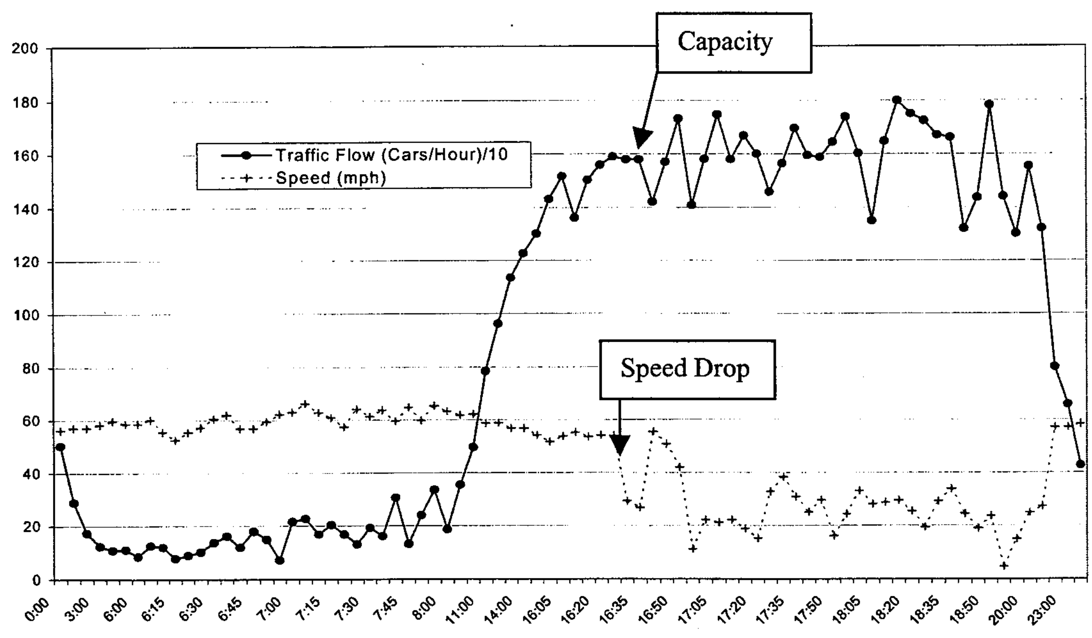

Figure 2.4. Traffic Flow and Speed Curves for Capacity Identification

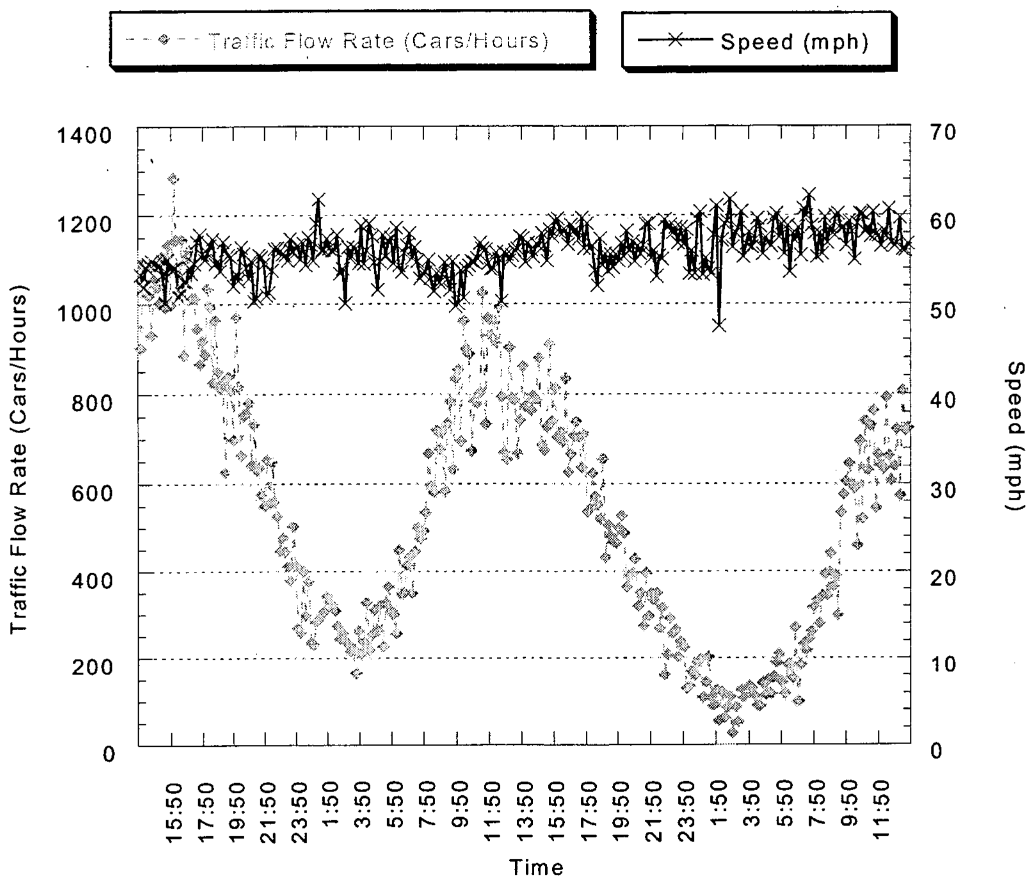

Figure 2.5. Uncongested Traffic Flow and Speed (I-65 over SR-46, Crossover Work Zone)

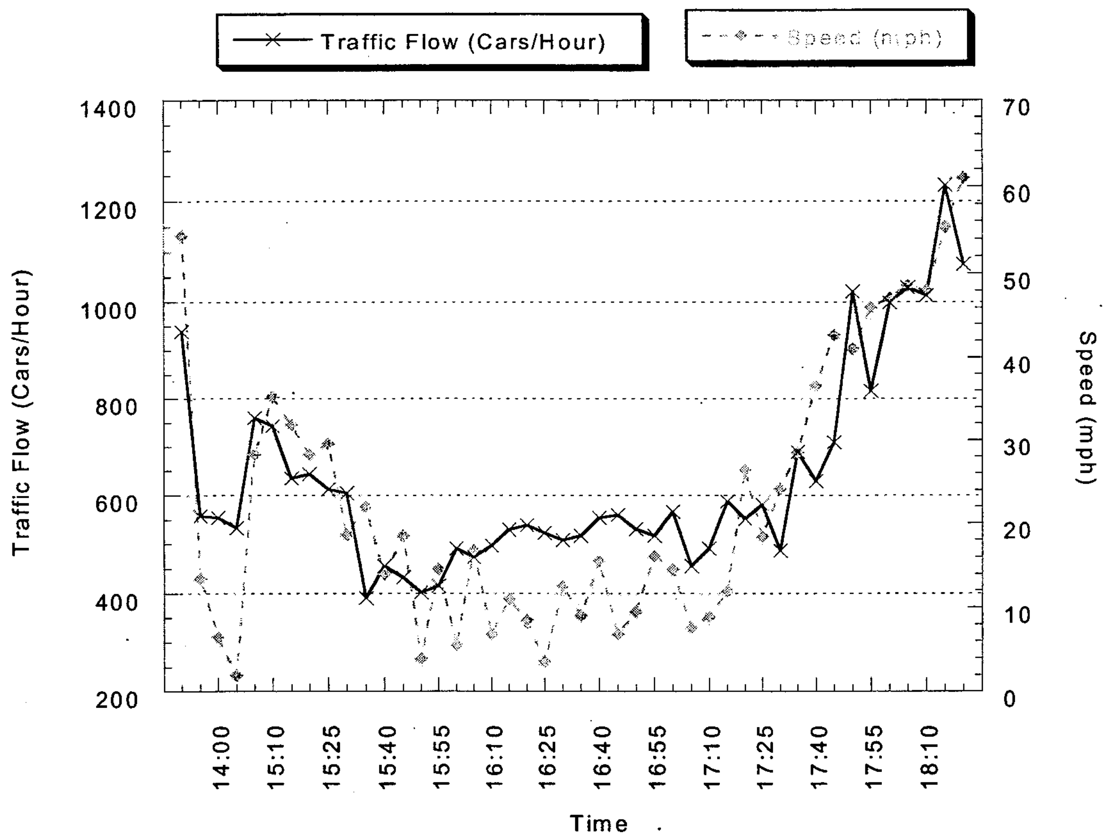

Figure 2.6. Work Zone Traffic Flow and Speed During Congestion (I-65 N. of SR-32)

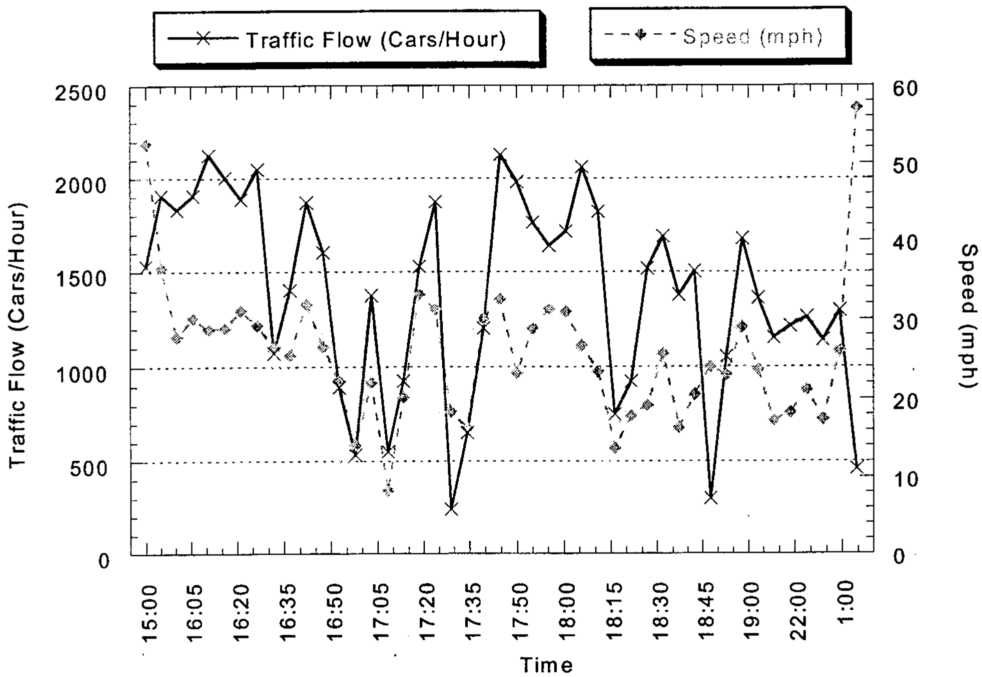

Figure 2.7. Work Zone Traffic Flow and Speed During Congestion (I-70 E. of SR-9)

CHAPTER 3. TRAFFIC DELAY AND VEHICLE QUEUE ESTIMATION

Traffic delays at work zones are caused by reduced number of lanes for traffic and lower speed limit. Traffic delays consist of those under uncongested traffic condition and those under congested traffic condition. When the arrival traffic flow rate exceeds the work zone capacity, traffic congestion may occur and therefore result in vehicle queues and traffic delays. On the other hand, when the arrival traffic flow rate is below the work zone capacity, vehicles may pass a work zone smoothly but at a lower speed than normal driving. Vehicles at this reduced travel speed through the work zone need a longer time to pass the work zone than the time needed to pass the same length of the roadway without a work zone. This additional time spent at the work zone is also a traffic delay caused by the work zone. Furthermore, because of the stochastic feature of traffic flow, vehicle queues may also form at a work zone even when the arrival traffic flow rate is below the work zone capacity. All these types of traffic delays at work zones should be accounted and estimated to examine the impact of work zones on highway traffic and the resulted user costs.

3.1. Delay Due To Vehicle Deceleration Before Entering Work Zone

Assuming a uniform deceleration, the delay for each vehicle before entering a work zone can be calculated using the basic equations of dynamics. It was observed during data collection in Indiana that as vehicles approach a work zone they normally decelerate to the work zone speed from the freeway speed over a distance of about 2 miles (3.2 kilometers). Without a work zone, the travel time (t_f) of a vehicle over a section of length s at the freeway speed (v_f) is:

$$t_f = \frac{s}{v_f} \qquad (1)$$

With a work zone, the approach travel time (t_a) of the vehicle with a uniform deceleration over the same section to reduce its speed from the freeway speed (v_f) to the work zone speed (v_z) is:

$$t_a = \frac{2s}{v_f + v_z} \tag{2}$$

Then the delay (d_d) due to a vehicle deceleration (from v_f to v_z) when approaching a work zone is:

$$d_d = t_a - t_f = \frac{2s}{v_f + v_z} - \frac{s}{v_f} \tag{3}$$

This delay is called deceleration delay because it occurs when vehicles decelerate before entering a work zone.

3.2. Delay Due To Reduced Speed Through Work Zone

The traffic delay when vehicles travel through a work zone is the difference between the travel time needed to pass the work zone at the reduced speed and the travel time to pass the same length of the roadway without a work zone at the normal freeway speed. If the length of a work zone is L, then the delay (d_z) of a vehicle travelling within the work zone can be calculated as:

$$d_z = L\left(\frac{1}{v_z} - \frac{1}{v_f}\right) \tag{4}$$

3.3. Delay For Resuming Freeway Speed After Exiting Work Zone

Vehicles travel at reduced speed through a work zone and accelerate to their original freeway speed after exiting the work zone. The extra time for this speed resuming is a delay compared to freeway traffic without a work zone interruption. If the average acceleration is denoted as a, then the distance (S) traveled to change speed from v_z to v_f is:

$$S = \frac{v_f^2 - v_z^2}{2a} \tag{5}$$

The time needed for a vehicle to accelerate from v_z to v_f is

$$t_1 = \frac{v_f - v_z}{a} \tag{6}$$

If there is no work zone, the time needed for a vehicle to travel the same distance is

$$t_2 = \frac{S}{v_f} = \frac{v_f^2 - v_z^2}{2av_f} \tag{7}$$

Therefore, the delay for a vehicle to accelerate to its original speed is the difference between t_1 and t_2:

$$d_a = t_1 - t_2 = \frac{v_f - v_z}{a} - \frac{v_f^2 - v_z^2}{2av_f} = \frac{(v_f - v_z)^2}{2av_f} \tag{8}$$

During the traffic data collection at work zones on Indian freeways, it was observed that the average acceleration of vehicles was about 2 miles (3.2 kilometers) per hour per second after exiting a work zone. This average value of acceleration can be used in Equation 8 with the appropriate speed values to estimate the delay caused by speed resuming.

3.4. Delay Due To Vehicle Queues

Vehicle queues at work zone can occasionally form even when traffic volume is less than the work zone capacity. This type of delay can be attributed to the stochastic nature or the randomness of traffic flow. It can be analyzed and estimated using queuing theory (Bhat 1984; Gerlough and Huber 1975).

Queuing theory is used to mathematically predict the characteristics of a queuing system. A queuing system consists of a servicing facility, a process of arrival of customers to be served by the facility, and the process of service. For a queuing system, it is necessary to specify the following system characteristics and parameters:

1. Input process -- average rate of arrival and statistical distribution of time between arrivals;
2. Service mechanism -- service time average rates and distribution and number of customers that can be served simultaneously;
3. Queue discipline -- to the rules followed by the server in taking the customers into service, such as "first-come, first-service", or "random selection for service".

A notational representation is often used to describe the input distribution, service time distribution, and the number of servers of a queuing system. The notational representation can be written as: Input distribution/Service time distribution/Number of servers. Some standard notations used in queuing theory include G for an arbitrary distribution, M for Poisson (if arrivals) or exponential distribution (for interarrival or service times), D for a constant length of time (for interarrival or service times). For example, M/M/1 represents a queuing system with Poisson arrivals, exponentially distributed service times, and one server.

To estimate traffic delays with queuing theory, a work zone can be modeled as a server for vehicles to enter the work zone in order of vehicle arrivals. A work zone with one lane open is thus a one server queuing system and the queue discipline is apparently first-come first-service. The average arrival rate of the vehicles is the traffic flow rate and the service rate of the system is the traffic capacity of the work zone. Because of the randomness of highway traffic, the queuing system can be represented as a system with Poisson arrivals, exponentially distributed service times, and one server. That is, a freeway work zone with one lane open can be modeled as a M/M/1 queuing system.

If the average arrival rate of vehicles is denoted as F_a, then the average interval between arrivals is $1/F_a$. If the service rate of the system is the work zone capacity F_c, the average service time is $1/F_c$. The ratio $\rho = F_a/F_c$ is called the traffic intensity. If $\rho < 1$ (that is, $F_a < F_c$, or the traffic flow rate is below the work zone capacity), the vehicle queues can be mathematically estimated with queuing theory. On the other hand, if $\rho \geq 1$ (traffic flow rate exceeds the work zone capacity), queuing theory can not be used to analyze queues.

In this queuing system, the vehicles in the queuing system are defined as those vehicles that have already merged from the closed lane into the open lane leading to the work zone. Based on queuing theory (Bhat 1984; Gerlough and Huber 1975), the average number of vehicles in the system is

$$E(n) = \frac{\rho}{1-\rho} = \frac{F_a}{F_c - F_a} \qquad \text{for } \rho < 1 \qquad (9)$$

The average waiting time that an arrival vehicle spent before entering the work zone is

$$d_w = E(w) = \frac{F_a}{F_c(F_c - F_a)} \qquad (10)$$

The average queue length (or the average number of vehicles in the waiting line) is

$$E(m) = \frac{F_a^2}{F_c(F_c - F_a)} \tag{11}$$

Equation 11 is the average queue length over all time, including the period when there is no queue (i.e., queue length is 0). In practice, it is more helpful to know the average vehicle queue length if there is indeed a vehicle waiting line before the work zone. This is defined as the average queue length, given that the queue length is greater than 0. The equation for estimating this queue length is

$$\overline{Q} = E(m \mid m > 0) = \frac{F_c}{F_c - F_a} \tag{12}$$

In analyzing traffic delays at work zones, Equations 10 and 12 can be utilized to estimate the average vehicle delay time and the average queue length under uncongested traffic conditions.

Traffic congestion occurs when traffic flow rate exceeds the work zone capacity. As given in Table 2.4, under congested traffic conditions the average speeds were lower than under uncongested traffic conditions and the average flow rates were below the capacity values. Apparently, these average values of speeds and flow rates should be used in estimating work zone traffic delays under congested traffic conditions.

Once the flow rate of arrival vehicles exceeded the work zone capacity, for a given time period the number of vehicles arrived would be larger than the number of vehicles departed at the work zone. The difference between the number of vehicles arrived and the number of vehicles departed is the vehicle queue formed at the work zone. This can be written as

$$Q = (F_a - F_d)t \tag{13}$$

where t = time;

Q = vehicle queue formed during time t;

F_a = traffic flow rate of arrival vehicles;

F_d = vehicle queue-discharge rate (traffic flow rate of departure vehicles during congestion).

If there was an original queue (Q_0) at the beginning of the time (t=0), then the total queue length at time t is

$$Q_t = Q_0 + (F_a - F_d)t \tag{14}$$

If vehicles arrive at a constant rate and depart the work zone at the vehicle queue-discharge rate within a given hour, then the total vehicle queue at the end of hour i can be calculated as follows:

$$Q_i = Q_{i-1} + F_{ai} - F_d \tag{15}$$

where Q_i = total vehicle queue at the end of hour i;

F_{ai} = hourly volume of arrival vehicles at hour i;

F_d = vehicle queue-discharge rate.

This equation is equivalent to

$$Q_m = Q_0 + \sum_{i=1}^{m}(F_{ai} - F_d) = Q_0 + \sum_{i=1}^{m} F_{ai} - mF_d \tag{16}$$

It should be pointed out that in Equation 15 or 16 F_{ai} and F_d are hourly traffic volumes under congested traffic conditions and time t is not explicitly expressed because it equals 1.0 hour. If time t is less than 1.0, i.e., t is somewhere between hour i-1 and hour i, the equation should be written as

$$Q_i(t) = Q_{i-1} + (F_{ai} - F_d)\,t \tag{17}$$

$Q_i(t)$ represents the vehicle queue length at time t within hour i, where t is measured starting at the beginning of hour i.

In Equation 15, only if $F_{ai} > F_d$, the queue length increases during hour i, or $Q_i > Q_{i-1}$. On the other hand, if $F_{ai} < F_d$, the queue length decreases during hour i, or $Q_i < Q_{i-1}$. If the calculated Q_i from Equation 15 is less than 0, it implies that F_{ai} was less than F_d and that the vehicle queue has dissipated at some point in time within hour i. If $Q_i = 0$ from Equation 15, then the queue dissipated exactly at the end of hour i. If $Q_i < 0$, then the queue was cleared at a time point t before the end of hour i. Setting Equation 17 equal to 0, i.e., $Q_t(t) = Q_{i-1} + (F_{ai} - F_d)\,t = 0$, the time t at which the last vehicle in the queue was cleared can be obtained as

$$t = \frac{Q_{i-1}}{F_d - F_{ai}} \tag{18}$$

Here t is less than 1.0 hour because the queue was cleared before the end of hour i.

The traffic delay associated with the queued vehicles can be calculated based on the vehicle queue lengths. As given in Equation 17, the vehicle queue length at time t within hour i is $Q_i(t) = Q_{i-1} + (F_{ai} - F_d)\,t$. The delay (in vehicle-hours) of these $Q_i(t)$ vehicles during an infinitesimal time interval $(t, t + \Delta t]$ within hour i can be expressed as

$$\Delta D_i = Q_i(t)\,\Delta t \tag{19}$$

The total traffic delay during a time period from $t = 0$ to $t = T$ is

$$D_i\Big|_{t=0}^{t=T} = \int_0^T \Delta D = \int_0^T Q_i(t)\,dt = \int_0^T [Q_{i-1} + (F_{ai} - F_d)t]\,dt = Q_{i-1}T + \frac{1}{2}(F_{ai} - F_d)T^2 \tag{20}$$

For $T = 1$, then Equation 20 results in the total delay (in vehicle-hours) in hour i, that is

$$D_i = Q_{i-1} + \frac{1}{2}(F_{ai} - F_d) \tag{21}$$

If the traffic congestion started at hour 1 and ended during hour I, then D_1, $D_2 \ldots D_{I-1}$ can be calculated with Equation 21. Because the traffic congestion ended during hour I, the time t, at which the last vehicle in the queue was cleared, should be first calculated using Equation 18.

$$t_I = \frac{Q_{I-1}}{F_d - F_{aI}} \tag{22}$$

With t_I, the delay can be estimated using Equation 20.

$$D_{i=I} = D_I\Big|_{t=Q_{i-1}/(F_d - F_{aI})} = Q_{I-1}\left(\frac{Q_{I-1}}{F_d - F_{aI}}\right) + \frac{1}{2}(F_{aI} - F_d)\left(\frac{Q_{I-1}}{F_d - F_{aI}}\right)^2$$

or

$$D_I = \frac{Q_{I-1}^2}{2(F_d - F_{aI})} \tag{23}$$

In the Texas study, Memmott and Dudek (1982) obtained results similar to Equations 21 and 23 using a graphic method. However, a significant difference is that their study used work zone capacity, instead of queue-discharge rate, as the flow rate of departure vehicles. As can be seen in Table 2.4, the values of work zone capacities are higher than those of queue-discharge rates. It was observed at the Indiana freeway work zones that traffic flow rates could not sustain the capacity level during traffic congestion. Therefore, using the capacity values as the departure traffic flow rates would result in under estimations of the work zone traffic delays.

3.5. Total Traffic Delay At Work Zone

The total traffic delay at a work zone is then the sum of the individual delays discussed above. Under uncongested traffic conditions, the total traffic delay at a work zone in hour i is

$$DELAY_i = F_{ai}\,(d_d + d_z + d_a + d_w) \qquad (24)$$

Under congested traffic conditions, the total delay at a work zone in hour i is

$$DELAY_i = F_{ai}\,(d_d + d_z + d_a) + D_i \qquad (25)$$

As Equation 22 shows, traffic congestion exists only during a portion (t_I) of the last hour (hour I). Therefore, the total delay in hour I should include the discharged queued vehicles during the first portion of the hour (t_I) and the expected vehicle queues due to the randomness of traffic flow during the second portion of the hour (1-t_I).

$$DELAY_I = F_{aI}\,[d_d + d_z + d_a + (1 - t_I)d_w] + D_I \qquad (26)$$

where t_I and D_I are defined in Equations 22 and 23, respectively.

3.6. Equations of Vehicle Queue Characteristics

In addition to traffic delay estimations, also derived are the equations of other characteristics of vehicle queues caused by traffic congestion. These equations can be utilized to calculate such values as maximum and average queue lengths, time needed to clear a given vehicle queue, and waiting time of vehicles in queue.

According to Equation 15, Q_i increases as long as F_{ai} is greater than F_d. Therefore, the maximum queue length occurs just before F_{ai} drops below F_d. For example, if $F_{ai} > F_d$ during hour 0 through hour I-1 and $F_{ai} < F_d$ at hour I, then the maximum of the vehicle queue up to hour I is $\max(Q) = Q_{I-1}$.

At the beginning of hour i the queue length (in number of vehicles) is Q_{i-1}, and at the end of hour i the queue length is Q_i. According to Equation 17, the queue length changes linearly with time within each hour. Therefore, the average queue length of hour i is the mean of Q_{i-1} and Q_i:

$$\overline{Q}_i = \frac{1}{2}(Q_{i-1} + Q_i) = Q_{i-1} + \frac{1}{2}(F_{ai} - F_d) \qquad (27)$$

It is interesting to note that Equation 27 is the same as Equation 21, however, the difference is that $\overline{Q}_i$ is queue length in number of vehicles and D_i is traffic delay in vehicle-hours.

Queue length at any time t between hour i-1 and hour i is given by Equation 17 as $Q_i(t)$, which is the number of vehicles in the waiting line (or queue). Therefore, when a vehicle arrived at time t, this vehicle became the $Q_i(t)$ th vehicle in the queue. That is, the queue length at time t is $Q_i(t) = Q_{i-1} + (F_{ai} - F_d)t$. Since the queue-dissipating rate is F_d and the number of vehicles in the queue at time t is $Q_i(t)$, the time needed to clear all $Q_i(t)$ vehicles from the queue is

$$W_t = \frac{Q_i(t)}{F_d} = \frac{Q_{i-1} + (F_{ai} - F_d)t}{F_d} \tag{28}$$

W_t is also the waiting time for the $Q_i(t)$th vehicle to be cleared from the queue. This waiting time is nothing but the delay incurred to the vehicle that arrived at time t. Therefore, Equation 28 can be used to estimate the delay for any vehicle after it joined the queue. The values of W_t are not only important to traffic engineers, but also important to motorists. For example, the values of W_t can be used in changeable traffic message boards along freeway as the "expected delay time" at the work zone. Because delay for the n*th* vehicle is given as $\frac{n}{F_d}$ by Equation 28, the total delay of all $Q_i(t)$ vehicles in the queue is

$$W_{total} = \frac{1}{F_d} + \frac{2}{F_d} + \cdots + \frac{Q_i(t)-1}{F_d} + \frac{Q_i(t)}{F_d} = \frac{Q_i(t)[1+Q_i(t)]}{2F_d} \tag{29}$$

or

$$W_{total} = \frac{[Q_{i-1} + (F_{ai} - F_d)t] \times [1 + Q_{i-1} + (F_{ai} - F_d)t]}{2F_d} \tag{30}$$

It should be emphasized that W_{total} obtained from Equation 29 or Equation 30 is the total delay counted from time t, because the vehicles that joined the queue before time t had already sustained delays between the time they arrived and time t. The average delay time per vehicle in the queue (counted from time t) is then equal to the total queue delay, W_{total}, divided by the total number of vehicles in the queue, $Q_i(t)$.

$$W_{avg} = \frac{W_{total}}{Q_i(t)} = \frac{1+Q_i(t)}{2F_d} \tag{31}$$

3.7. An Application Example of the Traffic Delay Equations

To demonstrate the applications of the derived traffic delay equations, these equations were applied to calculate the traffic delays at an Indiana freeway work zone during a 24-hour period. The work zone was a crossover work zone of 7.3 miles (11.7 kilometers) long on Interstate 70 (I-70) between State Road 9 and State Road 29. As a crossover work zone affects traffic in both directions, the traffic delays at the work zone were calculated for both the median crossover and the opposite directions. In calculating the traffic delays, the values of work zone capacities and queue-discharge rates listed in Table 2.4 and the observed average vehicle speeds were used. It should be pointed out that if the actual vehicle speeds were not available, the values of the mean vehicle speeds in Table 2.4 could be used in the traffic delay calculations.

The hourly arrival traffic data and the calculated traffic delays are presented in Tables 3.1 and 3.2. The adjustment factors from the 1994 Highway Capacity Manual were used to convert trucks and buses to passenger car equivalents. Therefore, the traffic flow rates are expressed in passenger cars per hour. The traffic delays listed in the two tables are the hourly delays of the individual and total delays. It was calculated that the total 24-hour delay was 2922 car-hours in the crossover direction and 2380 car-hours in the opposite direction. The average hourly delay was 117 car-hours and 99 car-hours in the two directions, respectively. Thus, the whole work zone (including both directions) caused a total traffic delay of 5302 car-hours over the 24 hours. The average hourly traffic delay at the work zone was 221 car-hours.

To illustrate the changes of traffic delays with time, the traffic delays over the 24-hour period are plotted in Figures 3.1 and 3.2. Figure 3.1 indicates that in the crossover direction the traffic delays caused by reduced speed and vehicle queues were the two major sources of the total traffic delay. These two traffic delays increased considerably during the traffic congestion hours when vehicle queue formed and vehicle speed decreased. In the opposite direction, as shown in Figure 3.2, the reduced speed delay was significantly greater than other delays. Same as in the crossover direction, Figure 3.2 also shows that the reduced speed delay increased during the hours when the traffic flow rate was high and the vehicle speed was low. The queue delay was not significant in the opposite direction because the traffic flow rates did not exceed the work zone capacity during the 24 hours.

The equations of vehicle queue characteristics were also utilized to calculate additional queue attributes during traffic congestion in the crossover direction. The maximum queue length occurred at 17:00 with a length of 304 equivalent passenger cars. Since the queue length at 16:00 was 293 equivalent passenger cars, the average queue length during 16:00-17:00 was calculated with Equation 27 as

$$\overline{Q_i} = \frac{1}{2}(Q_{i-1} + Q_i) = \frac{1}{2}(293 + 304) = 299 \text{ equivalent passenger cars.}$$

Using Equation 28, the time needed to clear all 304 queued vehicles was calculated as

$$W_t = \frac{Q_i(t)}{F_d} = \frac{304}{1587} = 0.19 \text{ hours} = 11.5 \text{ minutes.}$$

Using Equation 29, the total delay of these queued vehicles (counted from time 17:00) was calculated as

$$W_{total} = \frac{Q_i(t)[1 + Q_i(t)]}{2F_d} = \frac{304 \times (1 + 304)}{2 \times 1587} = 29 \text{ car-hours.}$$

The average delay time per vehicle in the queue (counted from time 17:00) can be calculated by Equation 31:

$$W_{avg} = \frac{W_{total}}{Q_i(t)} = \frac{1+Q_i(t)}{2F_d} = 0.096 \text{ hours} = 5.8 \text{ minutes.}$$

3.8. Summary

This chapter presented the derivations and applications of a series of equations for estimating work zone traffic delays and vehicle queues. It has been emphasized that the vehicle queue-discharge rates, instead of the work zone capacity, should be used in calculating traffic delays because the queue-discharge rates are lower than the work zone capacity. The application of the derived equations in the I-70 work zone indicated that the reduced speed delay was a major contributor to the total traffic delay for both directions of the work zone. It should be noted that the I-70 work zone was a relatively long one (7.3 miles, or 11.7 kilometers). Because the reduced speed delay is directly proportional to the work zone length, a shorter work zone would result in a less considerable reduced speed delay. The application example also showed that the vehicle queue delay played a important role during traffic congestion. However, vehicle queues under uncongested traffic condition, or the stochastic queues, caused much less traffic delays than vehicle queues during traffic congestion.

The equations of the characteristics of individual vehicle queues can be used to estimate the maximum and average queue lengths of a vehicle queue for a given time period, the time needed to clear individual vehicles in a vehicle queue, and the total and average traffic delays of a vehicle queue. The quantities of these vehicle queue attributes have some useful applications. For example, one of such applications is to display real-time traffic information on changeable message boards for motorists, such as the expected vehicle delay time at the work zone. The values of expected queue lengths and delay time can also be applied for adaptive traffic controls at work zones.

Table 3.1. Estimated Traffic Delays at the I-70 Work Zone (in the Crossover Direction)

Time	Speed (km/h)	Traffic Flow (Passenger Cars/Hour)	Deceleration Delay (Car-Hours)	Reduced Speed Delay (Car-Hours)	Acceleration Delay (Car-Hours)	Queue Delay (Car-Hours)	Total Delay (Car-Hours)
0:00-1:00	91	513	1.54	12.12	0.09	0.15	13.9
1:00-2:00	94	437	1.14	8.87	0.06	0.10	10.2
2:00-3:00	90	407	1.29	10.20	0.08	0.09	11.6
3:00-4:00	92	444	1.29	10.12	0.07	0.10	11.6
4:00-5:00	91	463	1.42	11.25	0.09	0.12	12.9
5:00-6:00	94	523	1.35	10.43	0.07	0.16	12.0
6:00-7:00	92	690	1.97	15.39	0.11	0.32	17.8
7:00-8:00	91	894	2.69	21.17	0.16	0.69	24.7
8:00-9:00	94	861	2.23	17.27	0.12	0.61	20.2
9:00-10:00	94	1081	2.82	21.89	0.15	1.36	26.2
10:00-11:00	87	1209	4.40	35.58	0.30	2.25	42.5
11:00-12:00	89	1202	4.08	32.66	0.26	2.19	39.2
12:00-13:00	89	1266	4.15	33.03	0.26	2.87	40.3
13:00-14:00	88	1394	4.97	40.06	0.33	5.53	50.9
14:00-15:00	84	1530	6.24	51.35	0.46	17.64	75.7
15:00-16:00	59	1905	17.15	176.48	2.13	146.44	342.2
16:00-17:00	45	1598	19.69	244.16	2.82	298.38	565.1
17:00-18:00	48	1321	15.04	176.32	2.09	171.12	364.6
18:00-19:00	46	1365	16.30	197.50	2.31	4.68	220.8
19:00-20:00	46	1360	16.17	195.19	2.28	4.56	218.2
20:00-21:00	50	1154	12.70	145.66	1.74	1.81	161.9
21:00-22:00	47	1214	14.23	170.15	2.00	2.30	188.7
22:00-23:00	42	1262	16.31	209.85	2.38	2.83	231.4
23:00-0:00	41	1138	15.16	200.31	2.24	1.70	219.4

Table 3.2. Estimated Traffic Delays at the I-70 Work Zone (in the Opposite Direction)

Time	Speed (km/h)	Traffic Flow (Passenger Cars/Hour)	Deceleration Delay (Car-Hours)	Reduced Speed Delay (Car-Hours)	Acceleration Delay (Car-Hours)	Queue Delay (Car-Hours)	Total Delay (Car-Hours)
0:00-1:00	93	428	1.16	9.07	0.06	0.08	10.4
1:00-2:00	94	308	0.79	6.11	0.05	0.04	7.0
2:00-3:00	93	269	0.74	5.78	0.03	0.03	6.6
3:00-4:00	94	212	0.54	4.17	0.04	0.02	4.8
4:00-5:00	91	231	0.69	5.41	0.03	0.02	6.1
5:00-6:00	95	253	0.61	4.67	0.04	0.02	5.3
6:00-7:00	93	315	0.85	6.60	0.04	0.04	7.5
7:00-8:00	94	451	1.18	9.15	0.05	0.09	10.5
8:00-9:00	95	671	1.58	12.18	0.10	0.24	14.1
9:00-10:00	92	872	2.45	19.12	0.16	0.50	22.2
10:00-11:00	91	1189	3.68	29.12	0.29	1.46	34.6
11:00-12:00	87	1374	5.00	40.39	0.40	2.92	48.7
12:00-13:00	85	1472	5.89	48.32	1.16	4.54	59.9
13:00-14:00	67	1486	10.70	100.86	2.44	4.89	118.9
14:00-15:00	47	1306	15.30	182.77	2.23	2.23	202.5
15:00-16:00	46	1259	15.16	184.67	1.96	1.87	203.7
16:00-17:00	49	1458	16.43	191.26	2.34	4.24	214.3
17:00-18:00	48	1355	15.62	184.68	2.50	2.69	205.5
18:00-19:00	43	1182	15.01	190.58	1.96	1.42	209.0
19:00-20:00	47	1212	14.30	171.82	3.16	1.58	190.9
20:00-21:00	30	1097	18.11	302.55	1.80	1.06	323.5
21:00-22:00	47	1060	12.38	147.68	1.90	0.94	162.9
22:00-23:00	44	1279	15.95	199.54	2.16	2.01	219.7
23:00-0:00	46	582	6.95	84.22	0.00	0.17	91.3

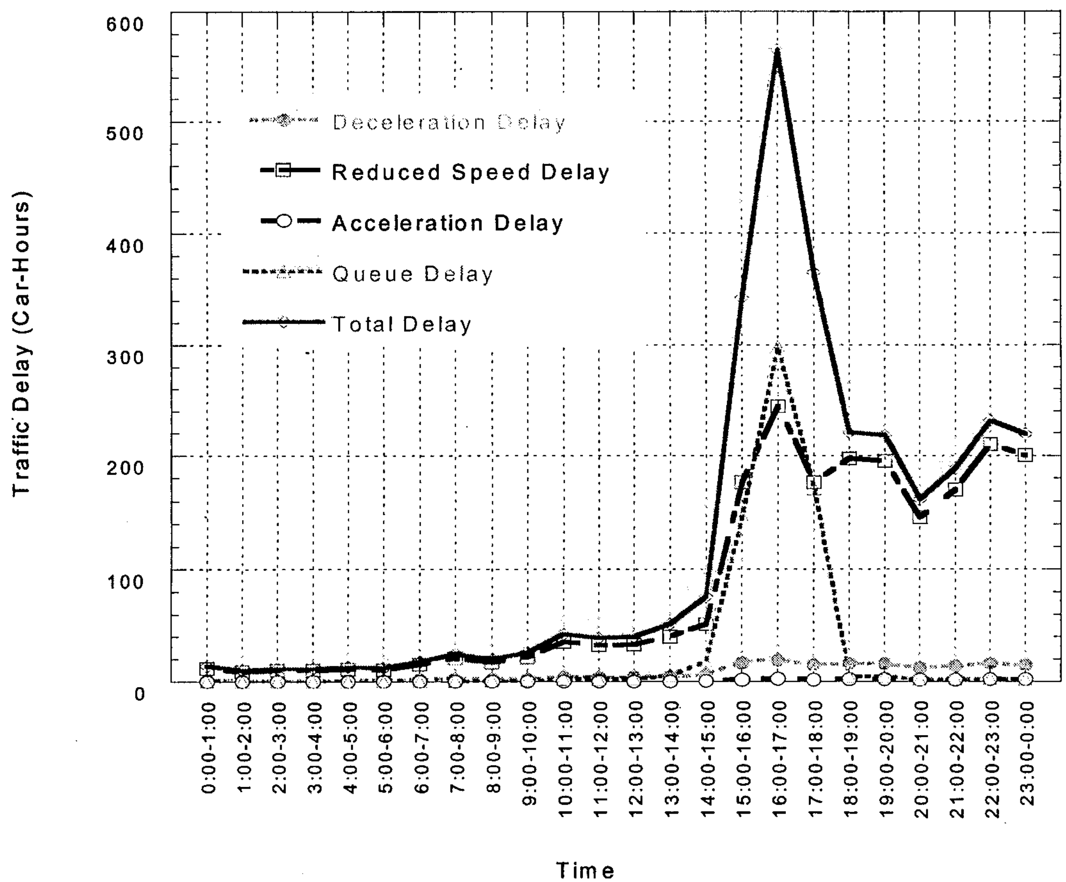

Figure 3.1.Traffic Delays at the I-70 Work Zone (Crossover Direction)

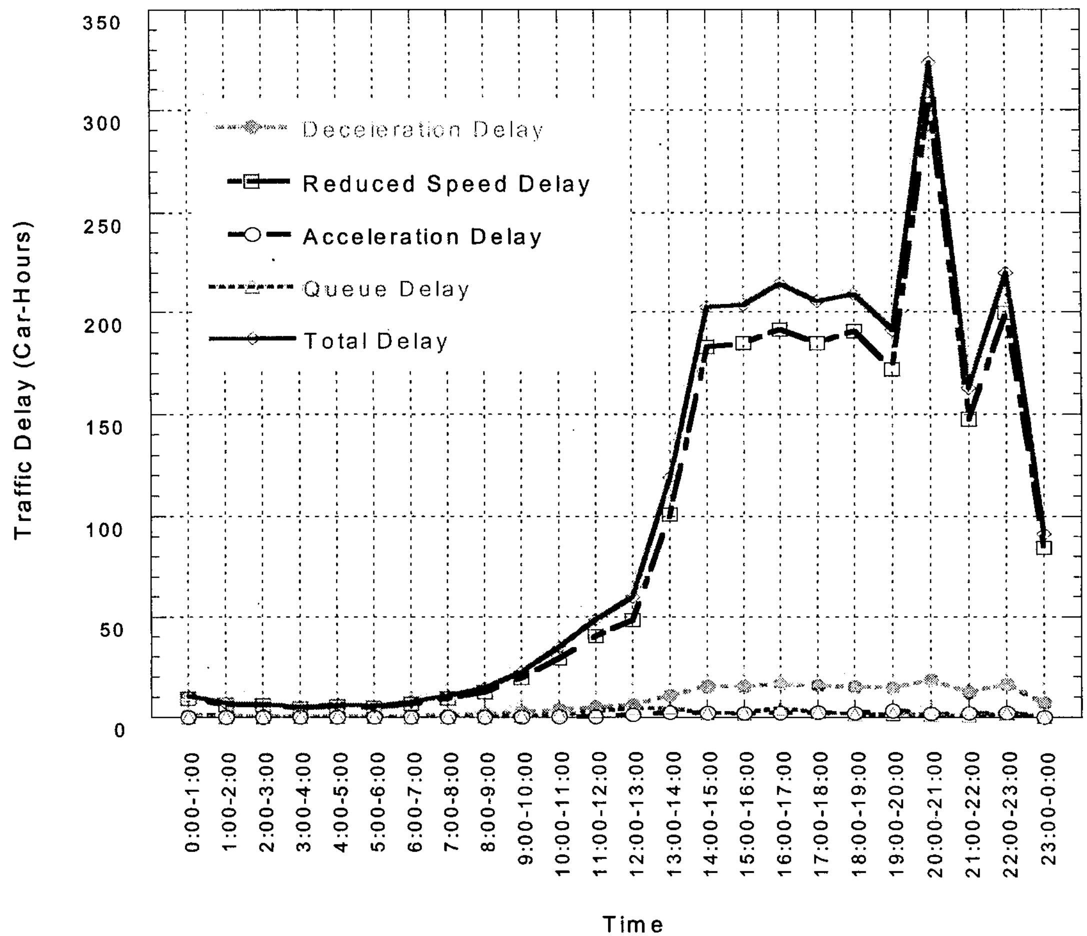

Figure 3.2.Traffic Delays at the I-70 Work Zone (Opposite Direction)

CHAPTER 4. EXCESS USER COSTS AT WORK ZONES

The excess user costs of traffic delays caused by the presence of a work zone are essential for assessment of the impact of the work zone on traffic. They are basically the costs incurred to the motorists because of reduced travel speed and capacity at work zones. The excess user costs include the traffic delay costs and the additional vehicle operating costs resulted from the speed changes at work zones. The traffic delay costs are estimated on the basis of the equations for traffic delay estimation, which were described in the last chapter.

4.1. Deceleration Delay Cost

When approaching a work zone on a freeway, a vehicle gradually reduces its speed from the freeway speed (v_f) to the work zone speed (v_z) over a deceleration distance (s). The deceleration delay (d_d) is expressed in Equation 3 as:

$$d_d = \frac{2s}{v_f + v_z} - \frac{s}{v_f}$$

The deceleration delay cost of hour i can then be calculated by multiplying d_d with the related traffic flow rates and unit costs of time for the given types of vehicles.

$$C_{di} = d_d \cdot F_{ai}(P_c \cdot U_c + P_t \cdot U_t) \quad (32)$$

where C_{di} = deceleration cost of hour i ($)
d_d = deceleration delay per vehicle (hour)
F_{ai} = approach traffic flow rate of hour i (vph)
P_c = percentage of cars
U_c = unit cost of time for cars ($/hour)

P_t = percentage of trucks
U_t = unit cost of time for trucks ($/hour)

This equation can be used to estimate the deceleration delay cost for either congested or uncongested traffic using the appropriate work zone speed value v_z in calculation of d_d. As shown in Table 2.4, v_z is lower under congested conditions than under uncongested conditions.

The values of unit time for vehicles in 1975 dollar values can be found in the AASHTO Red Book (AASHTO 1977) -- *A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements* (1977). In order to assess the current impact of work zones, the values of unit time from the ASSHTO Red Book should be updated to the current dollar values. The Red Book introduced the procedures for updating the time values using the Consumer Price Indexes (CPI) or the Wholesale Price Indexes (WPI). The updating procedures use CPI to update time values for cars and WPI for trucks. In 1978, the Wholesale Price Indexes (WPI) were renamed as Producer Price Indexes (PPI). Therefore, PPI is now used in place of WPI for updating the values of costs and prices. The indexes are published by the U.S. Bureau of Labor Statistics and are available on the Internet.

A travel time value of $3.0 per car per hour was used in the AASHTO Red Book in 1975 dollar value. The CPI was 161.2 for 1975 and was 486.8 for April 1998, then the time value for cars in 1998 should be

$$\frac{486.8}{161.2} \times \$3.0 = \$9.1\,/\text{car-hour}$$

The travel time value for combination trucks was \$7.28 per vehicle per hour in 1975. Analogously, it can be updated to 1998 value using PPI of 1975 (54.9) and PPI of April 1998 (125.0):

$$\frac{125.0}{54.9} \times \$7.28 = \$16.6/\text{truck-hour}$$

The travel time values for cars and trucks in any future year can be updated in the same manner using the corresponding CPI or PPI for the given year.

4.2. Reduced Speed Delay Cost

The traffic delay due to reduced speed at a work zone of length L is given by Equation 4:

$$d_z = L\left(\frac{1}{v_z} - \frac{1}{v_f}\right)$$

Substituting d_z for d_a in Equation 32, obtained is the delay cost of hour i due to reduced speed at a work zone:

$$C_{zi} = d_z \cdot F_{ai}(P_c \cdot U_c + P_t \cdot U_t) \qquad (33)$$

Using the appropriate work zone speed v_z in calculation of d_z, this equation can be used to estimate the delay cost for either congested or uncongested traffic.

4.3. Acceleration Delay Cost

After exiting a work zone, a vehicle accelerates from the work zone speed to the freeway speed. Assuming a constant acceleration rate a , the delay for the vehicle to accelerate to the freeway speed is shown in Equation 8:

$$d_a = \frac{(v_f - v_z)^2}{2 a v_f}$$

The delay cost of hour i for accelerating to the freeway speed is

$$C_{ai} = d_a \cdot F_{ai}(P_c \cdot U_c + P_t \cdot U_t) \qquad (34)$$

This equation is also applicable for both congested and uncongested traffic conditions.

4.4. Vehicle Queue Delay Cost

The calculations of vehicle queues are different for traffic flow rate below the capacity and for traffic flow rate above the capacity. Therefore, the calculations of the corresponding delay costs are also different. When traffic flow rate is less than the work zone capacity, vehicle queues may form because of the stochastic nature of traffic flows. Using the hourly flow rate, F_{ai}, as the arrival traffic flow rate of hour i and the traffic flow at work zone capacity, F_c, as the departure traffic flow rate, the average waiting time per vehicle can be written as in Equation 10:

$$d_w = \frac{F_{ai}}{F_c(F_c - F_{ai})}$$

Then when traffic flow rate is below the capacity, the cost of vehicle waiting time of hour i at the work zone is

$$C_{wi} = d_w \cdot F_{ai}(P_c \cdot U_c + P_t \cdot U_t) \tag{35}$$

Traffic congestion occurs with the formation of vehicle queues when the traffic flow rate exceeds the work zone capacity. If traffic congestion started at hour 1 and ended during hour I, then the traffic delay for hour i=1, 2, 3… I-1 is calculated with Equation 21:

$$D_i = Q_{i-1} + \frac{1}{2}(F_{ai} - F_d)$$

The traffic delay for hour i=I is given by Equation 23:

$$D_I = \frac{Q_{I-1}^2}{2(F_d - F_{aI})}$$

Then when traffic flow rate exceeds the capacity, the cost of traffic delay of hour i due to vehicle queues at the work zone is

$$C_{qi} = D_i(P_c \cdot U_c + P_t \cdot U_t) \tag{36}$$

4.5. Excess Cost of Speed Change Cycles

Speed changes at work zones result in additional operating costs of vehicles as a result of excess consumption of fuel, engine oil, tires, and vehicle parts. The AASHTO Red Book tabulated the excess costs of speed change cycles above costs of continuing at initial speed for vehicles in 1975 dollar value. The Red Book also presented the formulas of multipliers for updating the cost values to the dollar values of future years. For example, the multiplier formula for updating costs of speed change cycles of passenger cars is

$$M_{car} = 0.0022\,CPI_F + 0.0001\,CPI_O + 0.0033\,CPI_T + 0.0001\,CPI_M + 0.0017\,CPI_D$$

The multiplier formula for updating costs of speed change cycles of combination trucks is

$$M_{truck} = 0.0008\,PPI_F + 0.0047\,PPI_T + 0.0001\,CPI_M + 0.0003\,PPI_D$$

where

CPI_F = Consumer Price Index – Private Transportation, Gasoline Regular and Premium
CPI_O = Consumer Price Index – Private Transportation, Motor Oil, premium
CPI_T = Consumer Price Index – Private Transportation, Tires, new, tubeless
CPI_M = Consumer Price Index – Private Transportation, Auto Repairs and Maintenance
CPI_D = Consumer Price Index – Private Transportation, Automobile, new
PPI_F = Producer Price Index – Diesel Fuel to Commercial Consumers
PPI_O = Producer Price Index – Motor Oil, Premium Grade
PPI_T = Producer Price Index – Truck Tires
PPI_D = Producer Price Index – Motor Truck

As indicated in the AASHTO Red Book, the values of these indexes were relative values to the base year of 1967 for which the value was assigned a value of 100. The current available CPI and PPI values from the U.S. Bureau of Labor Statistics use the year of 1982 as the base year. Therefore, they should be converted to the values relative to the base year of 1967. Using the appropriate CPI and PPI values for April 1998 with the year of 1967 as the base year, the multipliers are computed as follows:

$$M_{car} = 0.0022 \times 273.6 + 0.0001 \times 275.4 + 0.0033 \times 301.8 + 0.0001 \times 497.6 + 0.0017 \times 433.3 = 2.4$$
$$M_{truck} = 0.0008 \times 155.9 + 0.0047 \times 293.8 + 0.0001 \times 497.6 + 0.0003 \times 441.9 = 1.7$$

If S_{car} and S_{truck} denote the excess cost values of speed change cycles for passenger cars and trucks (in dollars per 1000 cycles), then the excess cost of speed change cycles of hour i is calculated in dollars as

$$C_{ci} = F_{ai}(P_c \cdot S_{car} + P_t \cdot S_{truck})/1000 \quad (37)$$

If the cost values of speed change cycles from the AASHTO Red Book are used directly, then the updating multipliers should be applied to the appropriate costs in the above equation. Consequently, the equation should be written as

$$C_{ci} = F_{ai}(P_c \cdot M_{car} \cdot S_{car} + P_t \cdot M_{truck} \cdot S_{truck})/1000 \quad (38)$$

4.6. Excess Running Cost of Vehicles at Reduced Speed through Work Zone

Vehicles travel through work zones at lower than normal freeway speeds. The differences in travel speeds would result in different vehicle running costs. The AASHTO Red Book tabulated the running costs of passenger cars and trucks for different speeds in 1975 dollar value. Similar to the excess cost of speed change cycles, the running cost values listed in the Red Book should also be updated to a future year using the formulas of multipliers. The updating multiplier of passenger cars running on general and level tangents is given in the Red Book as

$$M_{car} = 0.0017\,CPI_F + 0.0001\,CPI_O + 0.0004\,CPI_T + 0.0016\,CPI_M + 0.0032\,CPI_D$$

The updating multiplier of combination trucks running on general and level tangents is given as

$$M_{truck} = 0.0013\,PPI_F + 0.0001\,PPI_O + 0.0007\,PPI_T + 0.0022\,CPI_M + 0.0013\,PPI_D$$

Using the appropriate CPI and PPI values for April 1998 with the year of 1967 as the base year, the multipliers are computed as follows:

$$M_{car} = 0.0017 \times 273.6 + 0.0001 \times 275.4 + 0.0004 \times 301.8 + 0.0016 \times 497.6 + 0.0032 \times 433.3 = 2.8$$
$$M_{truck} = 0.0013 \times 155.9 + 0.0001 \times 348.8 + 0.0007 \times 293.8 + 0.0022 \times 497.6 + 0.0013 \times 441.9 = 2.1$$

To convert the running costs from the dollar values of 1975 to the dollar values of 1998, the cost values from the Red Book should be multiplied by the appropriate multipliers. Table 4.1 shows the conversions of running costs on level grade for passenger cars and trucks. The original values for 1975 were given in dollars per 1000 vehicle-miles, the updated values for 1998 were converted to a metric system unit, dollars per 1000 vehicle-kilometers.

If R_{f-car}, $R_{f-truck}$, R_{w-car}, and $R_{w-truck}$ denote running costs in 1998 dollar value for cars on freeway, trucks on freeway, cars at work zone, and trucks at work zone, respectively, then the excess running cost of hour i caused by a work zone of L miles long is

$$C_{ri} = L \cdot F_{ai}[P_t(R_{w-truck} - R_{f-truck}) + P_c(R_{w-car} - R_{f-car})]/1000 \qquad (39)$$

If the cost values from the AASHTO Red Book are used directly, the updating multipliers should be multiplied by the corresponding running costs to convert the costs from 1975 dollars to 1998 dollars.

As shown in Table 4.1, the cost values are relatively low when running speeds are in the middle range and they are relatively high when running speeds are at the low and high ends. This means that the value of C_{ri} could be either positive or negative depending on the given running speeds. A negative C_{ri} would indicate that the vehicles incurred lower running cost at the reduced speed through a work zone as compared with traveling at the normal freeway speed.

4.7. Total Hourly Excess User Cost

The above individual user costs are the hourly excess user costs at a work zone in one direction. Therefore, the total hourly excess user cost at the work zone in that direction is the sum of these individual user costs. As presented above, the calculations of delay costs due to vehicle queues are different under congested and uncongested traffic conditions. Consequently, the delay cost due to vehicle queues, C_{wi}, should be used to calculate the total hourly excess user cost when traffic flow rate is less than the work zone capacity. If the cost update factor based on the dollar value in 1998 is *J*, then the corresponding equation for total hourly excess user cost under uncongested traffic conditions is

$$C_{total} = J\,(C_{di} + C_{zi} + C_{ai} + C_{wi} + C_{ci} + C_{ri}) \qquad (40)$$

When traffic flow rate is greater than the work zone capacity, the delay cost C_{qi} should be used in place of C_{wi}. Then the equation for total hourly excess user cost under congested traffic conditions should be

$$C_{total} = J\,(C_{di} + C_{zi} + C_{ai} + C_{qi} + C_{ci} + C_{ri}) \qquad (41)$$

The cost update factor at any time can be obtained with the Consumer Price Index (CPI) at the given time (with 1982-1984 CPI=100) as

$$J = \frac{CPI}{162.05}$$

The denominator, 162.05, in this formula is the CPI for 1998.

4.8. Applications of the User Cost Estimation Model

This excess user cost estimation model was applied to calculate the user costs at two work zones. One was a crossover work zone on Interstate 70 (I-70) between State Road 9 and State Road 29 and the other was a partial closure work zone on Interstate 65 (I-65) northbound lanes located about 12 miles (19.3 kilometers) north of Lebanon, Indiana. The length of the I-70 work zone was 7.3 miles (11.7 kilometers) and the I-65 one was 0.3 miles (0.5 kilometers). For each of the two work zones, a 24-hour traffic data was used for the excess user cost calculations. As a crossover work zone affects traffic in both directions, the excess user costs at the I-70 work zone were calculated for both the median crossover and the opposite directions. In calculating the excess user costs, the values of work zone capacities listed in Table 2.4 and the observed average vehicle speeds and traffic flow rates were used. It should be pointed out that if the actual vehicle speeds and traffic flow rates are not available, the values of the vehicle speeds and queue-discharge rates listed in Table 2.4 can be used in this excess user cost estimation model.

Presented in Tables 4.2, 4.3 and 4.4 are the estimated individual hourly excess user costs along with the recorded hourly traffic flow rates, the calculated total hourly excess user costs, and the estimated average hourly vehicle queue lengths. The tables show that the estimated total excess user cost of the 24-hour period is $28,186 for the I-70 work zone in the crossover direction, and $19,551 in the opposite direction. The total 24-hour excess user cost at the I-70 work zone is thus the sum of the two directional costs, i.e., $47,737. Although the partial closure work zone on I-65 affected traffic in only one direction, the estimated total 24-hour excess user cost is $45,985, which is close to the total amount ($47,737) of the two directional excess user costs at the I-70 work zone. The large user cost at the I-65 work zone can be attribute to the long period of traffic congestion. The total 24-hour traffic volumes were 25233, 21359 and 22977 equivalent passenger cars for the I-70 work zone in the crossover direction, the I-70 work zone in the opposite direction, and the I-65 work zone, respectively. That is, the total traffic volume of the I-65 work zone was in the middle of the three traffic volumes. However, the I-65 work zone had consecutive more than eight-hour traffic congestion (traffic flow exceeded work zone capacity) between 11:00 and 20:00, while the I-70 work zone had only two-hour traffic congestion in the crossover direction. The longer traffic congestion period at the I-65 work zone contributed significantly to the total excess user cost. Tables 4.2, 4.3 and 4.4 also provide the values of unit user costs as $1.20 per vehicle at the I-70 work zone in the crossover direction, $0.69 per vehicle at the I-70 work zone in the opposite direction, and $2.00 per vehicle at the I-65 work zone. These unit user costs are the average additional costs incurred to each vehicle passing through the particular work zones. They are the result of traffic delays and speed changes caused by the present of the work zones.

Figures 4.1, 4.2 and 4.3 depicts the change patterns of traffic flow, total user cost, and unit user cost in order of time. The unit costs in these figures are shown in dollars per 1,000 vehicles, instead of in dollars per vehicle, for the purpose of legibility. These figures indicate that the excess user costs generally follow the change patterns of traffic flow rates. The excess user costs increased slowly with the traffic flow rates when the traffic flows were below the work zone traffic capacities. However, the excess user costs increased considerably when traffic flow rates were close to or higher than the capacities. The sharp increases of the excess user costs were caused primarily by the vehicle queues as well as the reduction of vehicle speeds. Therefore, traffic flow and vehicle speed at a work zone are the two main factors affecting the excess user costs.

As shown in Tables 4.2, 4.3 and 4.4, the values of one of the six individual user costs, C_{ri} (the excess running cost due to speed changes), are all negative. C_{ri} is the additional vehicle operating cost through a work zone, which is related to the wearing of vehicle tires and other parts and consumption of fuel and motor oil. The negative values of C_{ri} indicate that the change of vehicle speed from high (freeway speed) to low (work zone speed) is beneficial in terms of vehicle running costs. To show the magnitudes of the individual user cost items, the values of the six individual user costs are plotted in Figures 4.4, 4.5 and 4.6. In the crossover direction of the I-70 work zone, as Figure 4.4 shows, C_{zi} (reduced speed delay cost) and C_{ri} (excess running cost due to speed changes) have relatively large magnitudes of values. In addition, the delay cost of vehicle queues (C_{wi} or C_{qi}) has large values during traffic congestion period. In the opposite direction of the I-70 work zone, as Figure 4.5 shows, C_{zi} and C_{ri} also have relatively large values. However, the values of C_{wi} or C_{qi} remain consistently small because there was no traffic congestion during the 24-hour period. Figure 4.6 shows that in the I-65 work zone the only noticeable user cost is the delay cost of vehicle queues (C_{wi} or C_{qi}) during the long period of traffic congestion. The reason that the I-70 work zone has relatively greater values of C_{zi} and C_{ri} than the I-65 work zone is the difference in work zone lengths. As Equations 3, 4 and 14 imply, both C_{zi} and C_{ri} are directly proportional to the work zone length. The length of the I-70 work zone (7.3 miles or 11.7 kilometers) is much longer than that of the I-65 work zone (0.3 miles or 0.5 kilometers). Therefore, the lengths of the work zones greatly affected the values of C_{zi} and C_{ri}.

4.9. Summary

A model for estimating the excess user costs at freeway work zones was presented in this chapter. Through the application examples, it was demonstrated that the work zone user costs were mainly affected by traffic flow rates, vehicle speeds, and work zone lengths. During traffic congestion at work zones, the delay costs of vehicle queues contributed greatly to the total excess user costs. In both of the I-70 and I-65 work zones, the excess running costs due to speed changes had negative values, indicating the reduced speeds at the work zones actually reduced the vehicle running costs. It was also shown that C_{zi} (reduced speed delay cost) and C_{ri} (excess running cost due to speed changes) could contribute considerably to the total excess user costs in long work zones.

Table 4.1. Running Cost Updates on Level Grade Roadways

Speed	Running Cost for Passenger Cars		Running Cost for Combination Trucks	
	A (in 1975 Dollar) (per 1000 vehicle-mile)	B (in 1998 Dollar) (per 1000 vehicle-km) $B=A*M_{car}$	C (in 1975 Dollar) (per 1000 vehicle-mile)	D (in 1998 Dollar) (per 1000 vehicle-km) $D=C*M_{truck}$
5 mph 8 km/h	$108.95	$305.06	$270.42	$567.88
10 mph 16 km/h	81.28	227.58	182.69	383.65
15 mph 24 km/h	74.43	208.40	156.02	327.64
20 mph 32 km/h	70.72	198.02	145.75	306.08
25 mph 40 km/h	70.00	196.00	143.22	300.76
30 mph 48 km/h	70.06	196.17	145.66	305.89
35 mph 56 km/h	70.81	198.27	151.33	317.79
40 mph 64 km/h	72.03	201.68	160.00	336.00
45 mph 72 km/h	73.2	204.96	171.85	360.89
50 mph 80 km/h	74.5	208.60	189.91	398.81
55 mph 88 km/h	76.23	213.44	204.19	428.80
60 mph 97 km/h	78.49	219.77	216.48	454.61
65 mph 105 km/h	81.37	227.84	-	-
70 mph 113 km/h	84.57	236.80	-	-
75 mph 121 km/h	88.81	248.67	-	-
80 mph 129 km/h	93.87	262.84	-	-

Table 4.2. Estimated User Costs at the I-70 Work Zone (in the Crossover Direction)

Time	Flow Rate (Cars/Hour)	Avg. Queue (Vehicles)	C_{di} ($)	C_{zi} ($)	C_{ai} ($)	C_{wi} or C_{qi} ($)	C_{ci} ($)	C_{ri} ($)	Total Cost ($)	Unit Cost ($/Vehicle)
0:00-1:00	513	0.47	15.73	123.89	0.92	1.52	9.36	-70.80	80.62	0.19
1:00-2:00	437	0.37	11.96	92.83	0.63	1.05	8.45	-58.26	56.67	0.17
2:00-3:00	407	0.34	13.57	107.54	0.83	0.90	8.03	-53.56	77.30	0.25
3:00-4:00	444	0.38	13.70	107.52	0.78	1.11	8.93	-57.75	74.30	0.23
4:00-5:00	463	0.40	15.00	118.49	0.90	1.22	9.11	-61.16	83.55	0.24
5:00-6:00	523	0.48	14.01	108.56	0.72	1.62	9.99	-70.31	64.60	0.16
6:00-7:00	690	0.75	20.20	158.18	1.13	3.29	12.79	-94.53	101.07	0.18
7:00-8:00	894	1.25	27.31	215.13	1.60	7.03	16.10	-124.59	142.57	0.19
8:00-9:00	861	1.15	22.28	172.68	1.15	6.13	14.88	-122.83	94.29	0.13
9:00-10:00	1081	2.03	27.72	215.04	1.45	13.40	17.81	-157.89	117.53	0.12
10:00-11:00	1209	3.00	43.16	348.71	2.95	22.03	19.80	-177.14	259.50	0.24
11:00-12:00	1202	2.93	39.43	315.51	2.55	21.14	18.93	-179.57	217.99	0.20
12:00-13:00	1266	3.66	39.97	318.24	2.51	27.70	19.80	-189.73	218.49	0.19
13:00-14:00	1394	6.39	47.80	385.05	3.21	53.13	21.64	-209.56	301.26	0.24
14:00-15:00	1530	18.59	60.01	493.61	4.47	169.57	23.75	-229.98	521.44	0.37
15:00-16:00	1905	146.4	174.61	1797.25	21.65	1826.81	34.47	-903.64	2951.16	1.90
16:00-17:00	1598	151.95	239.81	2973.11	34.36	4281.75	36.80	-1110.15	6455.69	4.76
17:00-18:00	1321	19.18	232.16	2720.95	32.19	2506.94	38.88	-1182.29	4348.85	3.12
18:00-19:00	1365	5.53	274.03	3319.48	38.77	78.72	44.34	-1365.32	2390.02	1.58
19:00-20:00	1360	5.41	251.83	3040.06	35.56	71.05	39.81	-1193.56	2244.76	1.50
20:00-21:00	1154	2.52	222.81	2554.95	30.44	31.67	38.75	-1182.87	1695.75	1.24
21:00-22:00	1214	3.05	255.03	3048.90	35.80	41.18	41.05	-1236.01	2185.96	1.43
22:00-23:00	1262	3.61	150.40	1935.32	21.95	26.08	22.62	-700.04	1456.32	1.93
23:00-0:00	1138	2.40	202.86	2679.62	29.94	22.70	29.64	-918.70	2046.05	2.08
Daily Total	25233	-	$2415.4	$27350.6	$306.5	$9217.8	$545.7	-$11650.2	$28185.7	-
Hourly Avg.	1051	15.9	$100.6	$1139.6	$12.8	$384.1	$22.7	-$485.4	$1174.4	$1.12/Veh.

Note:

C_{di} --- Deceleration Delay Cost
C_{ai} --- Acceleration Delay Cost
C_{qi} --- Vehicle Queue Delay Cost (Congested Traffic)
C_{ri} --- Excess Running Cost due to Speed Changes
C_{zi} --- Reduced Speed Delay Cost
C_{wi} --- Vehicle Waiting Time Cost (Uncongested Traffic)
C_{ci} --- Excess Cost of Speed Change Cycles

Table 4.3. Estimated User Costs at the I-70 Work Zone (in the Opposite Direction)

Time	Flow Rate (Cars/Hour)	Avg. Queue (Vehicles)	C_{di} (\$)	C_{zi} (\$)	C_{ai} (\$)	C_{wi} or C_{ai} (\$)	C_{ci} (\$)	C_{ri} (\$)	Total Cost (\$)	Unit Cost (\$/Vehicle)
0:00-1:00	428	0.36	11.57	90.13	0.63	0.79	7.26	-61.52	48.85	0.11
1:00-2:00	308	0.24	7.92	61.34	0.41	0.38	5.38	-43.69	31.74	0.10
2:00-3:00	269	0.20	7.49	58.41	0.41	0.28	4.77	-37.76	33.59	0.12
3:00-4:00	212	0.15	5.49	42.51	0.28	0.17	3.85	-29.31	23.00	0.11
4:00-5:00	231	0.17	6.91	54.42	0.40	0.20	4.05	-32.60	33.39	0.14
5:00-6:00	253	0.19	6.05	46.54	0.29	0.24	4.32	-36.25	21.19	0.08
6:00-7:00	315	0.24	8.33	64.80	0.45	0.39	5.19	-45.97	33.18	0.11
7:00-8:00	451	0.39	11.48	89.06	0.60	0.88	7.25	-66.77	42.49	0.09
8:00-9:00	671	0.71	15.27	117.38	0.73	2.32	10.49	-100.55	45.64	0.07
9:00-10:00	872	1.18	23.51	183.79	1.30	4.79	13.53	-131.04	95.88	0.11
10:00-11:00	1189	2.81	35.31	279.16	2.12	13.98	18.33	-179.37	169.52	0.14
11:00-12:00	1374	5.79	47.93	387.20	3.27	28.01	21.19	-207.32	280.28	0.20
12:00-13:00	1472	10.49	56.65	464.45	4.16	43.65	22.85	-221.25	370.50	0.25
13:00-14:00	1486	11.82	104.15	981.62	11.40	47.61	23.88	-219.87	948.78	0.64
14:00-15:00	1306	4.27	150.99	1803.82	21.19	22.01	21.80	-581.82	1437.98	1.10
15:00-16:00	1259	3.57	153.15	1865.86	21.74	18.90	22.33	-623.30	1458.69	1.16
16:00-17:00	1458	9.46	134.42	1564.84	18.56	34.70	20.24	-542.70	1230.05	0.84
17:00-18:00	1355	5.26	176.10	2081.83	24.55	30.35	27.13	-767.59	1572.39	1.16
18:00-19:00	1182	2.75	202.91	2575.81	29.42	19.20	27.95	-797.25	2058.04	1.74
19:00-20:00	1212	3.03	175.68	2110.47	24.74	19.38	25.06	-665.94	1689.37	1.39
20:00-21:00	1097	2.13	151.74	2534.97	23.97	8.91	17.55	-534.77	2202.37	2.01
21:00-22:00	1060	1.92	191.44	2283.21	26.84	14.53	28.40	-780.82	1763.60	1.66
22:00-23:00	1279	3.84	200.38	2506.88	28.85	25.27	26.63	-693.72	2094.29	1.64
23:00-0:00	1130	1.99	190.32	2305.48	26.93	4.57	26.42	-688.03	1865.68	1.65
Daily Total	21319	-	\$2075.2	\$24554.0	\$273.2	\$341.5	\$395.8	-\$8089.2	\$19550.5	-
Hourly Avg.	888	2.98	\$86.5	\$1023.1	\$11.4	\$14.2	\$16.5	-\$337.1	\$814.6	\$0.69/Veh.

Note:

C_{di} --- Deceleration Delay Cost
C_{ai} --- Acceleration Delay Cost
C_{qi} --- Vehicle Queue Delay Cost (Congested Traffic)
C_{ri} --- Excess Running Cost due to Speed Changes
C_{zi} --- Reduced Speed Delay Cost
C_{wi} --- Vehicle Waiting Time Cost (Uncongested Traffic)
C_{ci} --- Excess Cost of Speed Change Cycles

Table 4.4. Estimated User Costs at the I-65 Work Zone (Partial Closure with the Right Lane Closed)

Time	Flow Rate (Cars/Hour)	Avg. Queue (Vehicles)	C_{di} ($)	C_{zi} ($)	C_{ai} ($)	C_{wi} or C_{ai} ($)	C_{ci} ($)	C_{ri} ($)	Total Cost ($)	Unit Cost ($/Vehicle)
0:00-1:00	243	0.19	4.05	1.29	0.15	0.26	3.80	-0.87	8.68	0.04
1:00-2:00	162	0.12	3.57	1.16	0.17	0.11	2.63	-0.52	7.12	0.04
2:00-3:00	158	0.11	3.51	1.14	0.16	0.10	2.57	-0.51	6.98	0.04
3:00-4:00	119	0.08	3.80	1.28	0.23	0.06	2.02	-0.73	6.66	0.06
4:00-5:00	123	0.09	3.01	0.99	0.15	0.06	2.16	-0.31	6.07	0.05
5:00-6:00	173	0.13	3.45	1.12	0.15	0.13	2.83	-0.55	7.12	0.04
6:00-7:00	319	0.26	6.11	1.97	0.25	0.47	4.88	-1.20	12.48	0.04
7:00-8:00	531	0.53	12.66	4.16	0.63	1.56	8.31	-1.91	25.41	0.05
8:00-9:00	786	1.05	34.13	12.15	2.73	4.48	12.37	-7.15	58.71	0.07
9:00-10:00	1223	3.90	53.68	19.17	4.34	22.94	18.99	-11.04	108.07	0.09
10:00-11:00	1494	17.20	88.69	33.95	8.82	112.32	23.19	-17.65	249.33	0.17
11:00-12:00	1659	61.00	108.79	42.95	11.56	632.69	25.39	-19.21	802.17	0.48
12:00-13:00	1652	118.71	117.59	47.70	13.12	1604.93	25.29	-22.41	1786.22	1.08
13:00-14:00	1617	158.71	111.41	44.94	12.31	2506.83	23.62	-20.20	2678.92	1.66
14:00-15:00	1591	185.75	97.65	38.00	10.08	3148.16	23.05	-17.04	3299.89	2.07
15:00-16:00	1647	240.70	109.35	44.04	12.05	4014.48	23.41	-20.15	4183.19	2.54
16:00-17:00	1720	332.34	124.92	52.75	14.80	5562.66	23.82	-20.75	5758.19	3.35
17:00-18:00	1541	334.39	132.06	56.57	15.94	6706.00	24.80	-22.10	6913.28	4.49
18:00-19:00	1689	410.22	134.28	56.11	15.68	7627.76	27.04	-24.34	7836.53	4.64
19:00-20:00	1345	314.08	138.11	60.28	17.06	7569.87	25.41	-23.20	7787.52	5.79
20:00-21:00	1058	74.68	116.56	47.92	13.29	3719.39	23.92	-20.96	3900.11	3.69
21:00-22:00	817	1.14	125.91	51.33	14.16	10.65	26.57	-23.42	205.20	0.25
22:00-23:00	739	0.93	102.39	40.00	10.65	8.12	24.61	-18.56	167.21	0.23
23:00-0:00	570	0.59	106.95	42.97	11.74	5.01	22.82	-19.38	170.11	0.30
Daily Total	22977	-	$1742.6	$703.9	$190.2	$43259.0	$403.4	-$314.2	$45985.2	-
Hourly Avg.	957	94.1	$72.6	$29.3	$7.9	$1802.4	$16.8	-$13.1	$1916.1	$2.00/Veh.

Note:

C_{di} --- Deceleration Delay Cost
C_{ai} --- Acceleration Delay Cost
C_{qi} --- Vehicle Queue Delay Cost (Congested Traffic)
C_{ri} --- Excess Running Cost due to Speed Changes
C_{zi} --- Reduced Speed Delay Cost
C_{wi} --- Vehicle Waiting Time Cost (Uncongested Traffic)
C_{ci} --- Excess Cost of Speed Change Cycles

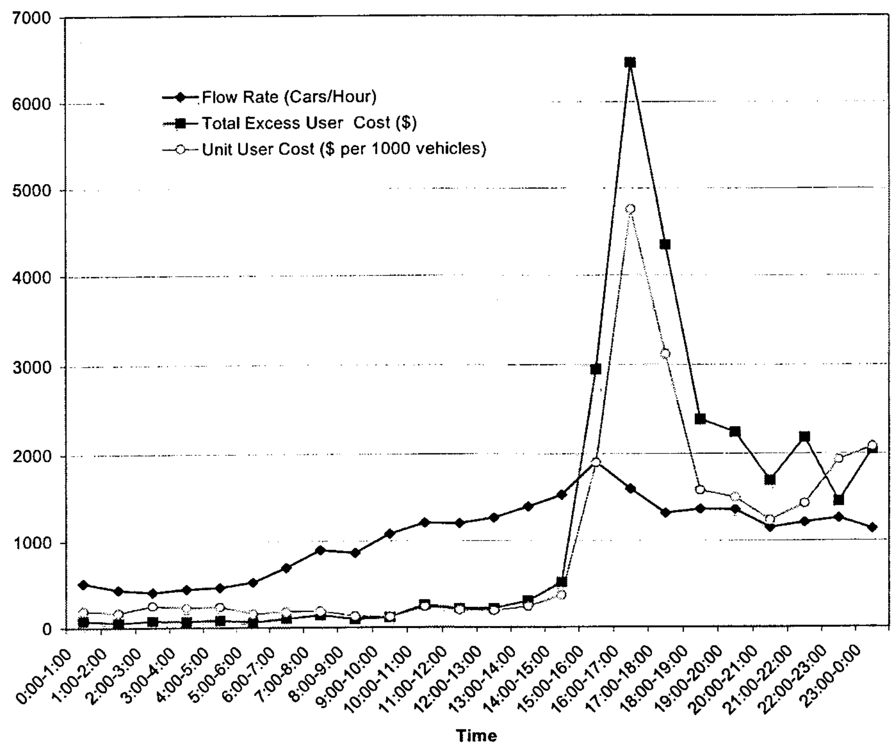

Figure 4.1.Traffic Flow Rates and Excess User Costs of the I-70 Work Zone (in the Crossover Direction)

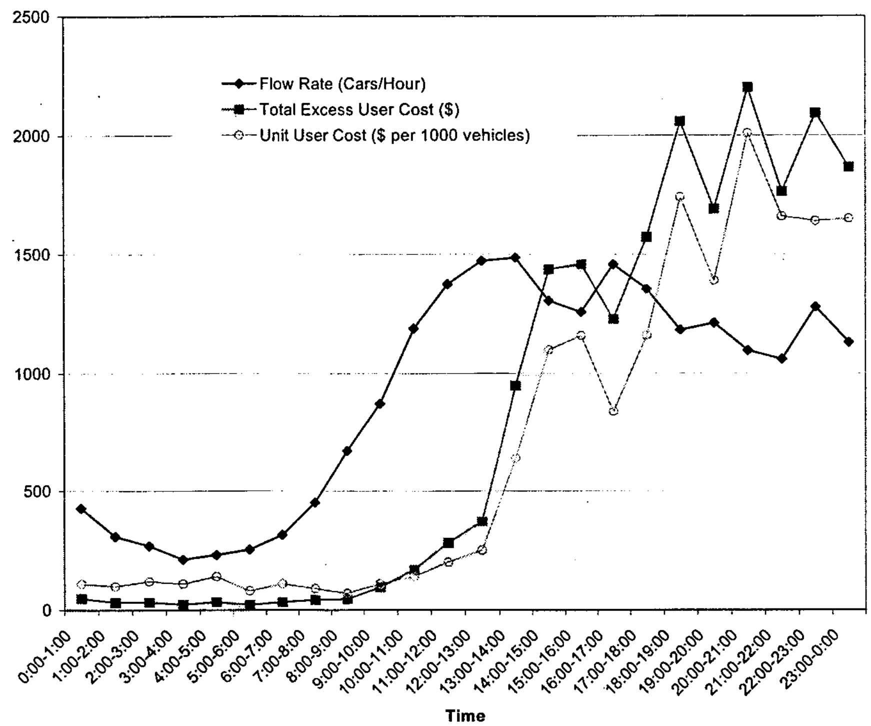

Figure 4.2. Traffic Flow Rates and Excess User Costs of the I-70 Work Zone (in the Opposite Direction)

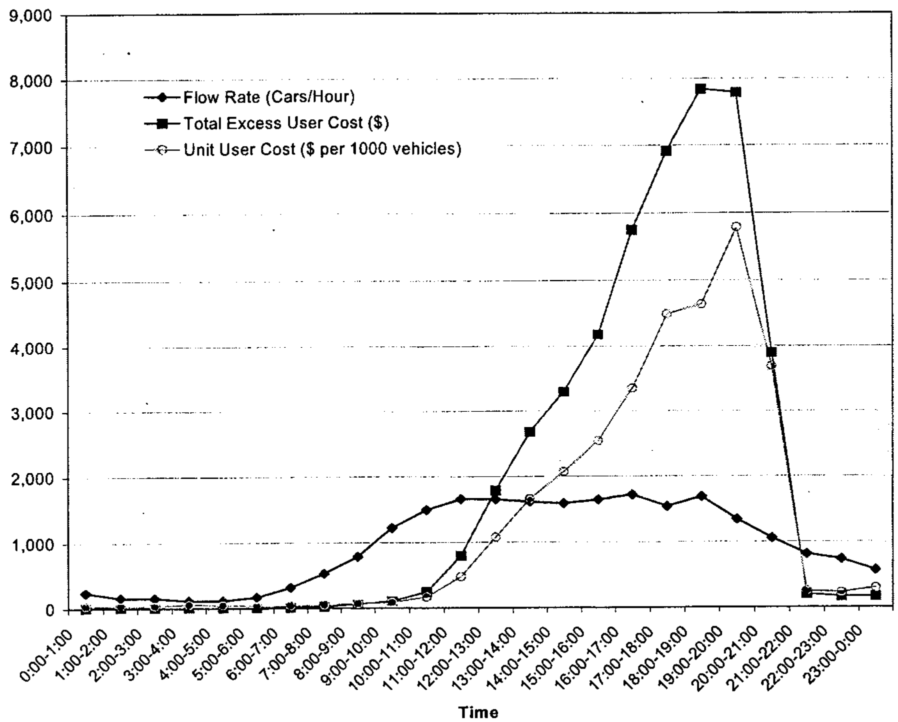

Figure 4.3. Traffic Flow Rates and Excess User Costs of the I-65 Work Zone

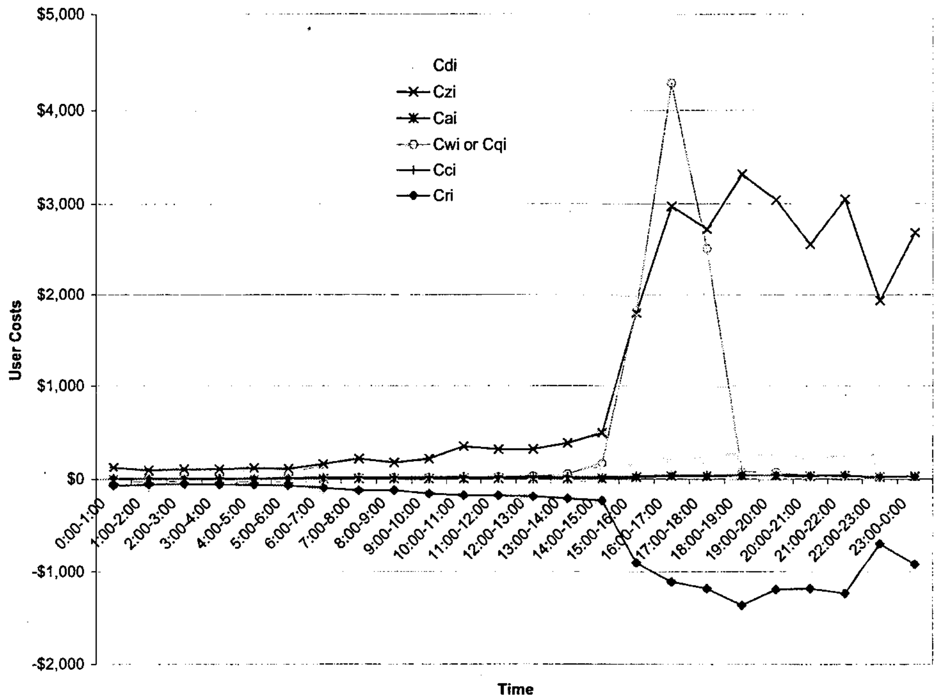

Figure 4.4. Individual Components of User Costs of the I-70 Work Zone (in the Crossover Direction)

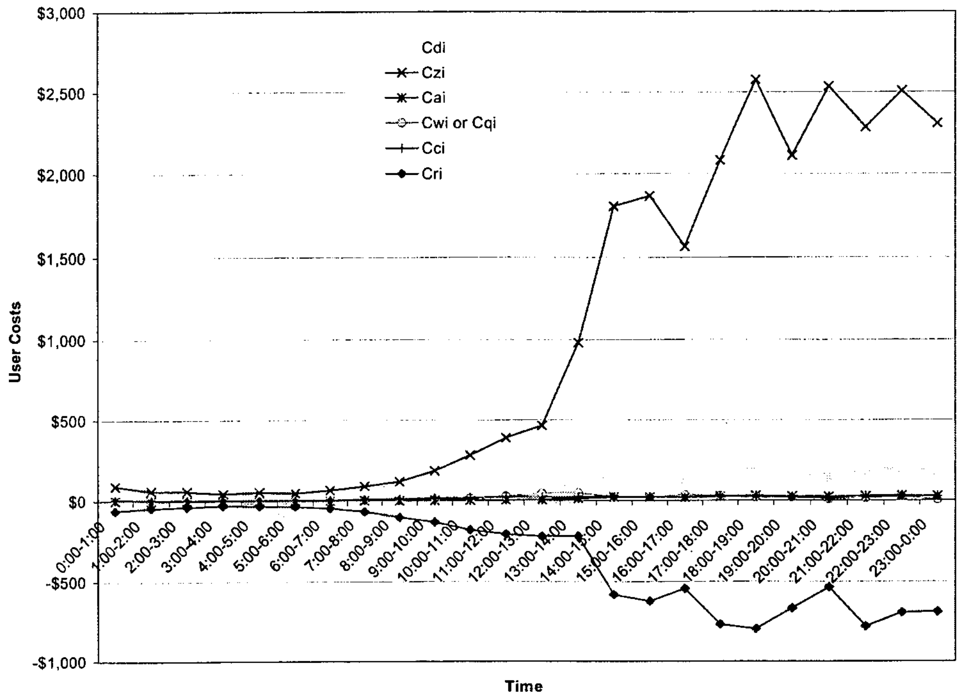

Figure 4.5. Individual Components of User Costs of the I-70 Work Zone (in the Opposite Direction)

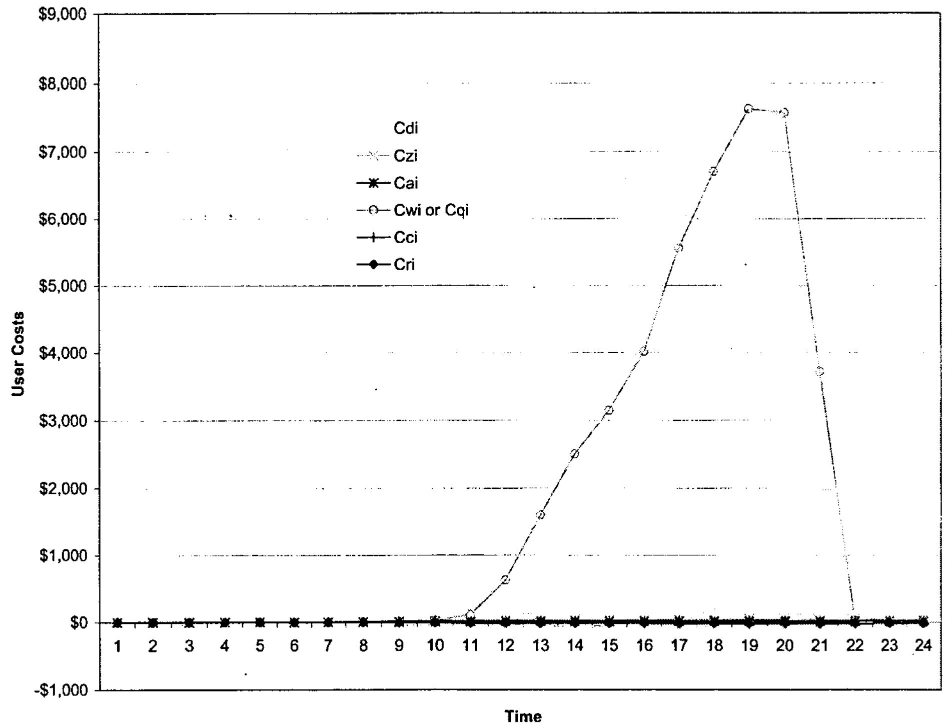

Figure 4.6. Individual Components of User Costs of the I-65 Work Zone

CHAPTER 5. DYNAMIC PREDICTION OF WORK ZONE TRAFFIC FLOW AND TRAFFIC CONGESTION

5.1. Prediction of Traffic Flow Using Time Series

Based on the collected traffic data, the traffic capacity values were determined for four types of work zone layouts on Indian four-lane freeways, i.e., crossover work zone in the opposite direction, crossover work zone in the crossover direction, partial closure with the right lane closed, and partial closure work zone with the left lane closed. Table 5.1 presents the four work zone capacity values obtained with traffic data at work zones on Indiana four-lane freeways.

Given the work zone capacity values, it was desired to develop methods for predicting traffic flow and congestion at work zones so that appropriate traffic control strategies could be applied to avoid traffic congestion and to reduce traffic delay. Traffic flow rate constantly changes with time on any given highway sections. To predict traffic conditions, the relationship between traffic flow and time must be studied. The time series theory (Cryer 1986, Bowerman and O'Connell 1979) is a frequently used tool to study the traffic and time relationship. One of the time series models is the *autoregressive process* $\{Z(t)\}$. A *p*th-order autoregressive process, AR(p), satisfies the following equation (Bowerman and O'Connell 1979):

$$Z(t) = \phi_1 Z(t-1) + \phi_2 Z(t-2) + \cdots + \phi_p Z(t-p) + \varepsilon_t \qquad (42)$$

where:

$Z(t)$ = value of the process Z at time t;
ϕ_i = unknown parameters; i = 1, 2, 3, …, p
ε_t = a random variable with zero mean and variance σ_w^2.

This equation requires that the mean of the series has been subtracted out so that *Z(t)* has a zero mean. This time series implies that the current value of the series *Z(t)* is a linear combination of the *p* most recent past values of itself plus an error term ε_t.

To show the use of the time series method in traffic flow prediction, the recorded traffic flow data at a work zone on Interstate 65 (I-65) over Indiana's State Road 46 was selected for fitting the first-order autoregressive process model. It was a crossover work zone for bridge rehabilitation. The traffic flow data was collected inside the work zone in the crossover direction at 10-minute intervals from 4:00 a.m. to noon on November 2, 1996. Figure 5.1 shows the observed traffic flow rates in order of time. With the traffic flow data at this work zone, an AR(1) model was fitted using the MINITAB (Minitab 1996) computer software. The AR(1) equation for the traffic flow rate is expressed as follows:

$$f(t) = \phi_1 f(t-1) + \varepsilon_t \tag{43}$$

In Equation 43, *f(t)* denotes the traffic flow rate at time t. As expressed by the equation, the traffic flow rate at time t, *f(t)*, can be predicted from the traffic flow rate observed at the most recent past time point t-1, *f(t-*1*)*. It should be noted that the mean of the series of traffic flow rates must be subtracted from *f(t)* as required by the autoregressive model of Equation 1. The actual prediction is then the calculated *f(t)* plus the mean. If *f(t-1)* is given, then *f(t)* can be predicted as:

$$\hat{f}(t \mid t-1) = \overline{\phi}_1 f(t-1) \tag{44}$$

In this equation, $\overline{\phi}_1$ is the estimate of ϕ_1, and $\hat{f}(t \mid t-1)$ is the predicted value of *f*(*t*) based on the most recent observed traffic flow rate, *f*(*t*-1). Through this equation, predictions of traffic flow rates at the given work zone were calculated from 4:00 a.m. to noon at 10-minute intervals. For comparison, plotted in Figure 5.2 are the predicted and observed values of the traffic flow rates.

The curves in Figure 5.2 indicate that the predicted values followed the patterns of the observed traffic flows. The accuracy of the time series predictions is reflected by the values of residuals. In this case, a residual is the difference between the observed traffic flow rate and the traffic flow rate predicted by the time series model, that is, residual = $f(t) - \hat{f}(t \mid t-1)$. The residuals of the time series predictions are listed in Table 5.2 for all data points during the eight-hour period. To examine the magnitudes of the residuals, the absolute values of the residuals were used to calculate the statistics. As shown in Table 5.2, the absolute values of residuals have a mean of 83.9, a standard deviation of 72.9, and a minimum of 1.7, and a maximum of 276.1. Although these values are not extremely unacceptable, they certainly suggest the needs for improvement in the accuracy of the time series predictions.

5.2. Prediction of Traffic Flow Using Kalman Predictor in Combination with Time Series

One of the applications of control theory is to use the Kalman predictor (Bozic 1979) in recursive predictions of random signal processes. For example, the signal model can be a first-order autoregressive process:

$$x(t+1) = a\,x(t) + w_t \tag{45}$$

The observation (or measurement) is affected by additive random error v_t:

$$y(t) = c\,x(t) + v_t \tag{46}$$

where v_t is a random variable with zero mean and variance σ_v^2.

The Kalman predictor for the above signal model can be expressed as follows:

Predictor equation:

$$\hat{x}(t+1 \mid t) = a\,\hat{x}(t \mid t-1) + k(t)[y(t) - c\hat{x}(t \mid t-1)] \tag{47}$$

Predictor gain:

$$k(t) = \frac{acp(t \mid t-1)}{c^2 p(t \mid t-1) + \sigma_v^2} \tag{48}$$

Prediction mean-square error:

$$p(t+1 \mid t) = \frac{a}{c} k(t) \sigma_v^2 + \sigma_w^2 \tag{49}$$

Equations 47, 48 and 49 are called one-step Kalman predictor of the signal process expressed by Equations 45 and 46. The Kalman method yields the estimate of $x(t+1)$, i.e. the signal at time t+1, given the measured data $x(t)$ and the previous estimate $\hat{x}(t \mid t-1)$ at time t. It can be proved (Bozic 1979) that this one-step prediction estimate, denoted as $\hat{x}(t+1 \mid t)$, is an optimum estimate because the Kalman recursive prediction process minimizes the mean-square prediction error $E[x(t+1) - \hat{x}(t+1 \mid t)]^2$.

Some features of the Kalman predictor, such as recursive, continuously incorporating the most recent real-time data, and optimum prediction, are exactly the desirable functions for an efficient traffic flow prediction model. To use the Kalman predictor in traffic flow prediction, the AR(1) time series model as in Equation 43 can be used as the traffic flow model, that is:

$$f(t+1) = \phi f(t) + \varepsilon_t \tag{50}$$

Equation 50 is the first-order autoregressive process for the traffic flow. In addition, the observation (or measurement) of the traffic flow, m(t), is affected by additive random error v_t:

$$m(t) = \beta f(t) + v_t \tag{51}$$

Equation 10 is related to the accuracy of the traffic data measurement devices used in data collection. The one step Kalman recursive prediction equations can then be readily obtained from Equations 47 through 49:

Predictor equation:

$$\hat{f}(t+1|t) = \phi\hat{f}(t|t-1) + k(t)[m(t) - \beta\hat{f}(t|t-1)] \tag{52}$$

Predictor gain:

$$k(t) = \frac{\phi\beta p(t|t-1)}{\beta^2 p(t|t-1) + \sigma_v^2} \tag{53}$$

Prediction mean-square error:

$$p(t+1|t) = \frac{\phi}{\beta}k(t)\sigma_v^2 + \sigma_\varepsilon^2 \tag{54}$$

With Equations 50 through 54, traffic flow rate at t+1, $f(t+1)$, can be predicted as $\hat{f}(t+1|t)$ for each observed data at time t, $f(t)$. Since Equation 50 is a time series model of the first order autoregressive process, this Kalman predictor model is a combination of the time series and the Kalman predictor. It was expected that this prediction model would improve the prediction accuracy over the time series model as defined in Equation 43. To verify this, the Kalman predictor model was also applied to the work zone traffic flow data described in Figure 5.1. The predicted traffic flow rates from the Kalman predictor along with the corresponding observed values and the values from the time series method are plotted in Figure 5.3.

As shown in the figure, most of the predicted values from the Kalman model are closer to the observed values than the predicted values from the time series model. This indicates that the Kalman method indeed improved the prediction accuracy over the time series method. The differences in the prediction accuracy of the two methods can be more clearly described by plotting their corresponding residual values into the same graph, as shown in Figure 5.4. The residual graph distinctly shows that the most residuals of the Kalman predictions are considerably smaller than those of the time series predictions. Therefore, the improvement of the Kalman predictor over the time series method in traffic flow prediction is apparent and significant.

For a quantitative comparison, the values of the observed and predicted traffic flow rates are presented in Table 5.3 with the corresponding residual values. In addition, the differences between the absolute values of the time series and the Kalman residuals are also included in the table. Because there are positive and negative residuals, the use of the absolute values of the residuals is to compare the magnitudes of the residuals from the two prediction methods. The magnitude of a residual is the difference between the observed value and the predicted value. Therefore, a more accurate prediction yields a smaller magnitude of residual. If the absolute value of time series residual (TR) minus the absolute of the Kalman residual (KR) is positive, i.e., abs(TR)-abs(KR) > 0, then the magnitude of time series residual is greater than the Kalman residual, indicating the time series prediction is less accurate than the Kalman prediction.

As shown in the last column of Table 5.3, there are 40 positive values and 9 negative values of abs(TR)-abs(KR). This indicates that 40 out of the 49 Kalman predictions are more accurate than the time series predictions. The statistics of the absolute values of the residuals were also calculated for the predictions from the two methods. Table 5.3 indicates that the Kalman predictions have smaller values of mean, standard deviation, minimum and maximum of the absolute residuals than the time series predictions. Compared to the time series predictions, the Kalman predictions reduced the mean of the absolute residual values by (83.9-37.1)/83.9=55.8% and the standard deviation by (72.9-29.0)/72.9=60.2%. These large reductions in the values of the mean and standard deviation represent a significant improvement in the traffic flow predictions.

To statistically compare the predictions of the two methods, a paired t-test was performed. Since a t-test requires the data follow a normal distribution, the Anderson-Darling normality test (Minitab 1996) was used to check if the absolute values of the residuals follow a normal distribution. The normality test resulted in a p-value of 0.000 for the absolute values of the time series residuals and a p-value of 0.015 for the absolute values of the Kalman residuals, indicating neither of the data sets follows a normal distribution at a level of $\alpha = 0.05$. Then the data sets were transformed by square root of the absolute values of the residuals, i.e., $r'_{1i} = \sqrt{abs(TR)}$ and $r'_{2i} = \sqrt{abs(KR)}$. The Anderson-Darling normality test on the transferred data yielded a p-value of 0.135 for r'_{1i} and a p-value of 0.175 for r'_{2i}. Therefore, both of the transformed data sets are normally distributed at a level of $\alpha = 0.05$ and the paired t-test can be applied to compare them. The paired t-test was used to test if the difference between the mean of r'_{1i} (μ_1) and the mean of r'_{2i} (μ_2) is zero or greater than zero. The hypotheses to be tested are as follows:

H_0: $\mu_1 - \mu_2 = 0$

H_a: $\mu_1 - \mu_2 > 0$

If the Type I error is controlled at $\alpha = 0.05$, then the p-value of the paired t-test can be compared to the α value according to the decision rule:

If p-value $\geq \alpha$, conclude H_0.

If p-value $< \alpha$, conclude H_a.

The p-value of the paired t-test is 0.000, which is less than $\alpha = 0.05$. Therefore, H_a is concluded, i.e., the mean difference is greater than zero or μ_1 is significantly greater than μ_2. This implies that the Kalman predictor in combination with the time series method provides much better predictions of traffic flow rates than the time series method.

5.3. Prediction of Traffic Congestion at Work Zones

Once the traffic capacity of a work zone is known, the dynamic prediction of traffic flow rates discussed above constitutes a dynamic prediction of traffic congestion at the work zone. As previously indicated, the traffic data used in the above example was collected at a crossover work zone in the crossover direction. From Table 5.1, it can be found that the traffic capacity of this type of work zone in Indiana is 1612 passenger cars per hour. Thus, the traffic congestion at this work zone can be predicted with the Kalman predictor method at each step of the prediction according to the following criteria:

If $\hat{f}(t+1|t) < 1612$ passenger cars per hour, then no congestion at time t+1 is predicted;

If $\hat{f}(t+1|t) \geq 1612$ passenger cars per hour, then congestion at time t+1 is predicted.

5.4. Summary

This study showed that using the Kalman predictor in combination with the first-order autoregressive process of time series provided significantly improved traffic flow predictions over using only the time series method. This Kalman predictor model predicts the traffic flow at a work zone dynamically with each newly available traffic data. Therefore, it can be used as an efficient tool for real-time work zone traffic control and can be applied in such areas as the Intelligent Transportation Systems. A dynamic prediction of traffic flow rate at a work zone with the Kalman predictor constitutes a dynamic prediction of traffic congestion at the work zone as long as the traffic capacity is given.

Table 5.1. Traffic Capacities of Work Zones on Indiana's Four-Lane Freeways

Work Zone Type	Traffic Capacity
Crossover (Opposite Direction)	1745 Passenger Cars Per Hour
Crossover (Crossover Direction)	1612 Passenger Cars Per Hour
Partial Closure (Right Lane Closed)	1537 Passenger Cars Per Hour
Partial Closure (Left Lane Closed)	1521 Passenger Cars Per Hour

Table 5.2. Comparison of Observed and Time Series Predicted Traffic Flow Rates

Time	Observed = $f(t)$	Time Series = $\hat{f}(t \mid t-1)$	Residual = $f(t)$- $\hat{f}(t \mid t-1)$
4:00	210	258.8	-48.8
4:10	237	235.3	1.7
4:20	328	260.4	67.4
4:30	218	344.6	-126.2
4:40	256	243.1	12.7
4:50	311	277.8	33.0
5:00	264	328.7	-65.0
5:10	321	285.1	35.4
5:20	226	337.8	-112.2
5:30	328	249.8	78.2
5:40	364	344.7	19.7
5:50	310	378.5	-69.0
6:00	300	327.6	-27.4
6:10	257	319.0	-61.8
6:20	449	279.0	169.9
6:30	348	456.7	-108.6
6:40	352	363.4	-11.2
6:50	413	367.1	46.1
7:00	434	423.7	10.4
7:10	351	443.0	-92.2
7:20	446	365.9	80.2
7:30	501	454.1	46.5
7:40	475	504.6	-29.3
7:50	494	481.3	13.1

Table 5.2. (continued)

Time	Observed = $f(t)$	Time Series = $\hat{f}(t \mid t-1)$	Residual = $f(t)$- $\hat{f}(t \mid t-1)$
8:00	535	498.9	36.3
8:10	668	536.7	131.7
8:20	595	660.2	-65.0
8:30	581	592.4	-11.9
8:40	719	578.8	140.3
8:50	678	707.2	-29.0
9:00	716	669.3	46.9
9:10	585	704.5	-119.9
9:20	736	582.6	153.3
9:30	784	722.8	61.6
9:40	633	767.6	-134.4
9:50	834	627.6	206.8
10:00	853	814.1	39.1
10:10	696	831.4	-135.0
10:20	962	686.2	276.1
10:30	900	932.6	-32.6
10:40	889	874.9	13.8
10:50	675	864.3	-189.5
11:00	784	666.1	117.6
11:10	780	767.1	13.1
11:20	804	763.8	40.1
11:30	1026	785.8	239.9
11:40	735	991.4	-256.9
11:50	967	721.5	245.6
12:00	929	937.0	-8.4

Statistics of absolute values of residuals:

Mean=83.9 Standard Deviation = 72.9

Minimum = 1.7 Maximum = 276.1

Table 5.3. Results of Time Series and Kalman Predictions

Time	Observed	Time Series	Kalman	Time-Series Residual (TR)	Kalman Residual (KR)	Abs(TR)-Abs(KR)
4:00	210	258.8	235.4	-48.8	-25.4	23.5
4:10	237	235.3	259.6	1.7	-22.6	-20.8
4:20	328	260.4	318.6	67.4	9.3	58.1
4:30	218	344.6	280.2	-126.2	-61.8	64.4
4:40	256	243.1	286.8	12.7	-31.0	-18.3
4:50	311	277.8	319.9	33.0	-9.1	23.8
5:00	264	328.7	305.8	-65.0	-42.1	22.9
5:10	321	285.1	332.4	35.4	-11.8	23.6
5:20	226	337.8	289.1	-112.2	-63.5	48.7
5:30	328	249.8	330.4	78.2	-2.4	75.8
5:40	364	344.7	366.0	19.7	-1.5	18.2
5:50	310	378.5	348.4	-69.0	-38.9	30.1
6:00	300	327.6	336.7	-27.4	-36.5	-9.1
6:10	257	319.0	308.3	-61.8	-51.2	10.6
6:20	449	279.0	405.0	169.9	43.9	126.0
6:30	348	456.7	384.3	-108.6	-36.2	72.4
6:40	352	363.4	379.0	-11.2	-26.8	-15.6
6:50	413	367.1	411.1	46.1	2.2	44.0
7:00	434	423.7	434.6	10.4	-0.5	9.9
7:10	351	443.0	396.7	-92.2	-45.9	46.3
7:20	446	365.9	436.0	80.2	10.1	70.1
7:30	501	454.1	480.9	46.5	19.7	26.7
7:40	475	504.6	483.3	-29.3	-7.9	21.3
7:50	494	481.3	494.8	13.1	-0.5	12.6
8:00	535	498.9	521.9	36.3	13.3	23.0
8:10	668	536.7	606.3	131.7	62.2	69.6
8:20	595	660.2	596.5	-65.0	-1.2	63.8

Table 5.3. (continued)

8:30	581	592.4	584.6	-11.9	-4.1	7.8
8:40	719	578.8	657.7	140.3	61.4	78.9
8:50	678	707.2	661.7	-29.0	16.5	12.5
9:00	716	669.3	684.5	46.9	31.8	15.2
9:10	585	704.5	619.3	-119.9	-34.7	85.2
9:20	736	582.6	679.8	153.3	56.1	97.3
9:30	784	722.8	729.2	61.6	55.2	6.4
9:40	633	767.6	662.9	-134.4	-29.7	104.7
9:50	834	627.6	750.9	206.8	83.5	123.3
10:00	853	814.1	793.8	39.1	59.4	-20.3
10:10	696	831.4	722.0	-135.0	-25.6	109.4
10:20	962	686.2	844.2	276.1	118.2	157.9
10:30	900	932.6	854.3	-32.6	45.7	-13.1
10:40	889	874.9	851.7	13.8	37.0	-23.2
10:50	675	864.3	731.2	-189.5	-56.4	133.1
11:00	784	666.1	747.8	117.6	36.0	81.6
11:10	780	767.1	751.9	13.1	28.3	-15.2
11:20	804	763.8	766.7	40.1	37.3	2.8
11:30	1026	785.8	896.0	239.9	129.7	110.2
11:40	735	991.4	780.9	-256.9	-46.4	210.5
11:50	967	721.5	868.5	245.6	98.6	147.0
12:00	929	937.0	879.2	-8.4	49.4	-41.0

Statistics of absolute values of residuals:

	Time Series	Kalman
Mean	83.9	37.1
Standard Deviation	72.9	29.0
Minimum	1.7	0.47
Maximum	276.1	129.7

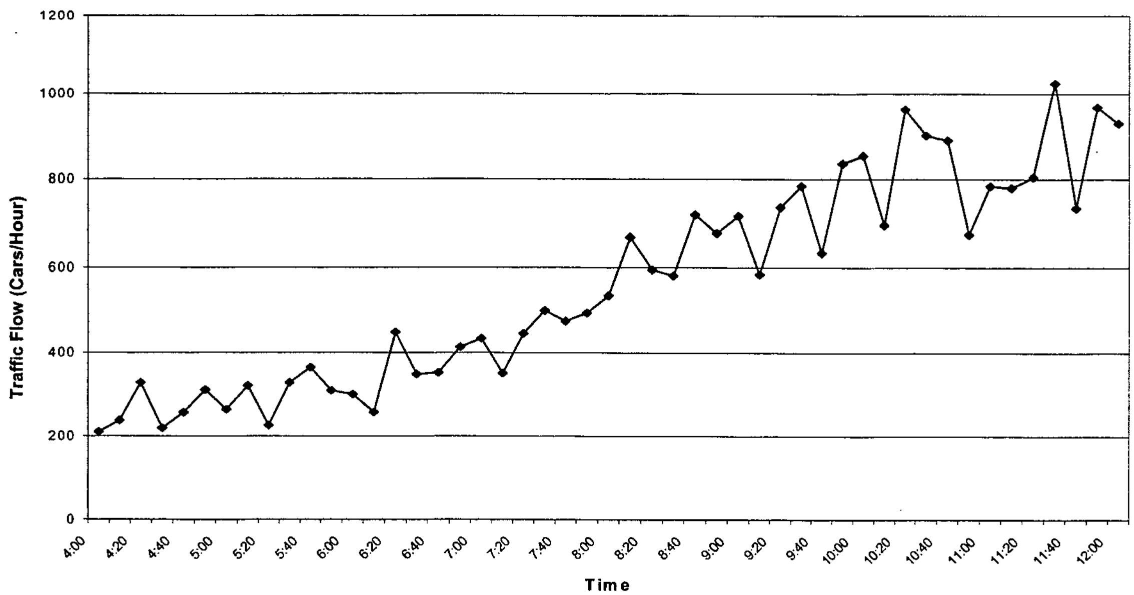

Figure 5.1. Observed Work Zone Traffic Flow

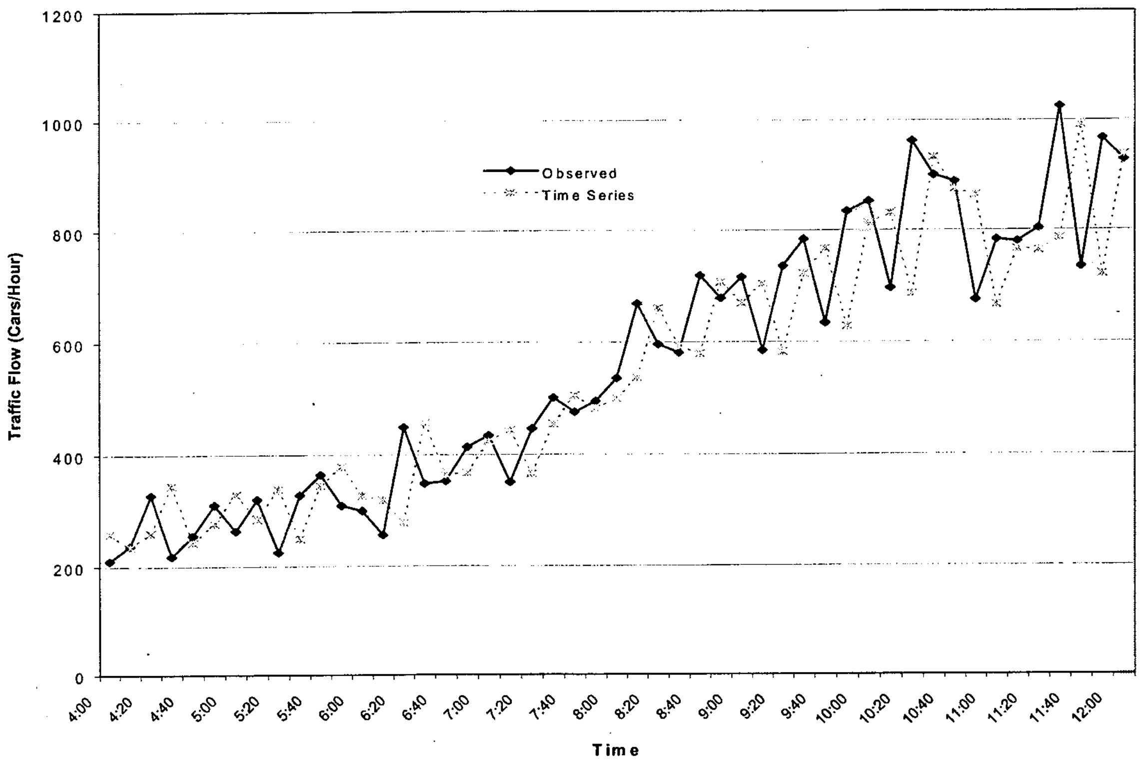

Figure 5.2. Observed and Time Series Predicted Traffic Flow Rates

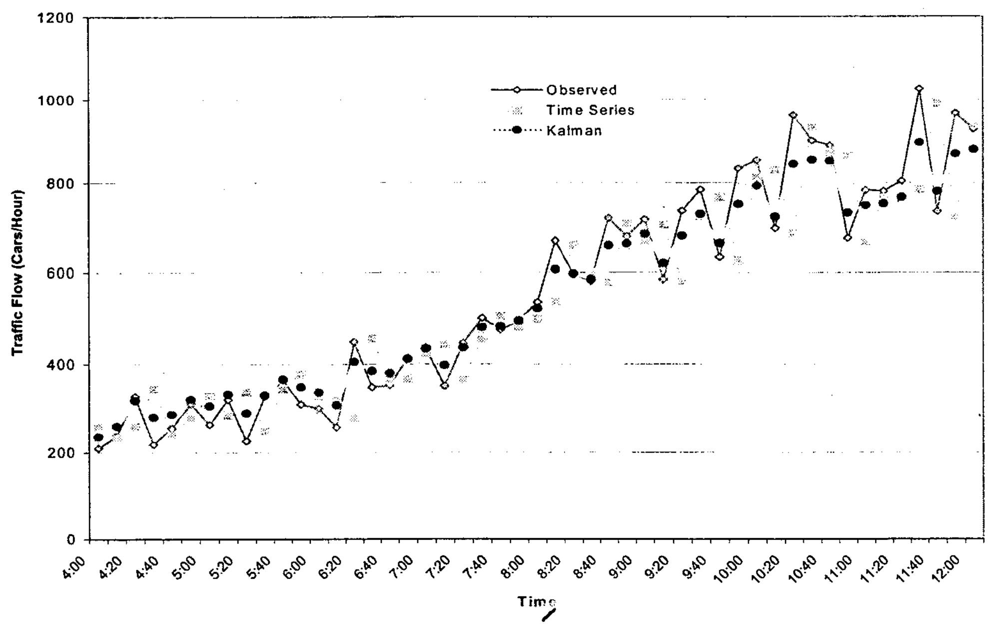

Figure 5.3. Observed and Kalman and Time Series Predicted Traffic Flow Rates

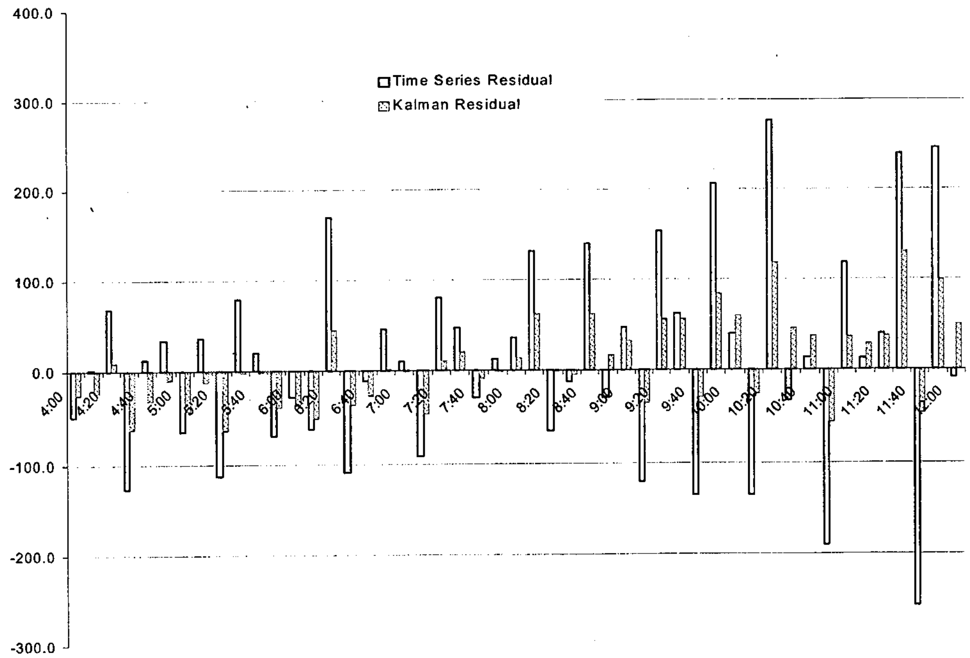

Figure 5.4. Residuals of Kalman Predictor and Time Series Predictions

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

The analyses of the traffic data at the Indiana freeway work zones indicated that traffic congestion at a work zone would cause sustained low vehicle speeds and fluctuated traffic flow rates. The collected traffic data showed that traffic congestion at the Indiana freeway work zones resulted in significant reductions in vehicle speeds (31.6% to 55.4% reduction). The work zone capacity values of the work zones on Indiana's four-lane freeways were obtained through the collected traffic data. The work zone capacities can be used as bases for work zone traffic controls and traffic congestion predictions. The mean queue-discharge rates were lower than the work zone capacities. It is therefore appropriate to use the queue-discharge rates rather than the capacity values in estimating traffic delays and associated user costs at work zones.

A series of equations were developed to estimate traffic delays and vehicle queues at work zones. Traffic delays at work zones are caused by reduced number of lanes for traffic and lower travel speed. Traffic delays at a work zone are the additional time for vehicles to decelerate before the work zone, traverse the work zone at reduced speed, and resume the original speed after exiting the work zone. Traffic congestion occurs when the arrival traffic flow exceeds the work zone capacity and vehicle queues form at the work zone during traffic congestion. Vehicle queues may also form when the arrival traffic flow is below the work zone capacity because of the randomness or the stochastic nature of traffic flow. The traffic delay equations derived in this study include four individual traffic delays: delay of vehicle deceleration before entering work zone, delay of reduced speed through work zone, delay of resuming the original speed after exiting work zone, and delay vehicle queues. Because vehicle queues can be triggered by either the over capacity traffic flow or by the randomness of arrival vehicles, the traffic delays of vehicle queues are calculated with different equations for over capacity traffic flow and under capacity traffic flow. In addition to the traffic delay equations, equations of the characteristics of individual vehicle queues were also developed. These equations can be used to calculate the maximum and average queue lengths, the time needed to clear individual vehicles from a vehicle queue, and the total and average traffic delays of a vehicle queue.

The presence of work zones on roadways result in excess costs for the motorists to travel on the roadways. The excess user costs are the results of lower travel speed and reduced traffic capacity at work zones. Besides traffic delay costs, the motorists also sustain additional vehicle operating costs from the additional vehicle wearing caused by the changes in driving maneuvers at work zones. It was demonstrated in this study that the main factors affecting the excess user costs were traffic flow rate, vehicle speed, and work zone length. The equations for calculating excess user costs at work zones were developed for both the traffic delay costs and the vehicle operating costs caused by freeway wok zones. Based on these work zone user cost equations, a computer program was developed for easy application. The computer program incorporated the obtained values of the work zone capacities, vehicle queue-discharge rates, and vehicle speeds at the work zones on Indiana four-lane freeways into the derived user cost equations. Because the values of these traffic measures at work zones on Indiana freeways with more than four lanes were not obtained in this study, the corresponding values from the 1994 Highway Capacity Manual were used in the computer program as default values. It is recommended that a study be conducted in the future to obtain a complete set of values for the Indiana freeway work zones.

Given the work zone capacity values, it was desired to develop methods for predicting traffic flow and congestion at work zones so that appropriate traffic control strategies could be applied to avoid traffic congestion and to reduce traffic delay. Such a method was developed in this study using the Kalman predictor in combination with the first-order autoregressive process of time series. The method provides significantly improved traffic flow predictions over using only the time series method. It predicts the traffic flow at a work zone dynamically with each newly available traffic data. Therefore, the prediction model can be used as an efficient tool for real-time work zone traffic control. This study showed that a dynamic prediction of traffic flow rate at a work zone with this prediction model would also constitute a dynamic prediction of traffic congestion at the work zone as long as the traffic capacity was given.

Through this study, the traffic characteristics at freeway work zones were analyzed, the equations for estimating traffic delays and user costs were derived, and a model of dynamically predicting work zone traffic flow and congestion was developed. The values of traffic capacities, queue-discharge rates, and vehicle speeds at work zones were determined based on the collected traffic data at work zones on Indiana's four-lane freeways. However, the values of traffic capacities, queue-discharge rates and vehicle speeds at work zones on freeways with more than four lanes were not provided in this study because the needed traffic data could not be collected. Traffic data at a work zone on a six-lane freeway near Fort Wayne was collected. The data from the work zone showed that traffic congestion did not occur during the data collection. Attempts were also made to obtain traffic data at the work zones on the Borman Expressway (I94/I80) through the installed data collection devices (video cameras, etc.), however, the data could not be collected because the data collection devices have not been operational as promised. Therefore, the values of traffic capacities and other measures could not be determined for the work zones on freeways with more than four lanes. In order to obtain a complete set of traffic measures for Indiana's freeway work zone traffic, it is recommended that data collections and further analyses be conducted for the work zones on Indiana freeways with more than four lanes. The additional traffic measures will certainly enhance the accuracy of the predictions of traffic delays, user costs, and traffic congestion occurrences at the Indiana freeway work zones. The traffic data should also be collected from the adjacent roads before and after the installation of the work zones to study the effects of the work zones on the approach traffic volumes.

CHAPTER 7. IMPLEMENTATION SUGGESTIONS

This study provides a set of equations and methods for INDOT to estimate traffic delays, vehicle queues, user costs, and traffic congestion occurrences at freeway work zones. For easy application, a computer software was developed to calculate work zone traffic delays and user costs. The research results and the computer software should be implemented by INDOT in making decisions on design and evaluation of freeway work zones and on work zone traffic controls. The following suggestions are made for implementation of the research results.

1. The computer software should be used to evaluate the work zone impact on the traffic flows and user costs at work zones. The estimations of the traffic delays and user costs can be used to choose appropriate work zone layouts or construction schedule based on the traffic conditions at the design and planning stage of highway construction projects. They can also be used to evaluate the traffic delays and user costs at existing work zones during construction.

2. The equations for calculating the attributes of work zone vehicle queues can be used to calculate the vehicle queue lengths and traffic delays of any given vehicle queues and to estimate the delay time of individual vehicles within the given vehicle queue. These equations are particularly useful for project engineers and traffic control engineers to evaluate the work zone impact on motorists. The estimated traffic delays of the vehicles and vehicle queues at a work zone are crucial information for the motorists to make such decisions as whether to pass the work zone or to use a detour. The information can be provided to the motorists by real-time displaying on the changeable message boards.

3. The model of dynamic predictions of work zone traffic flows and congestion provides a tool for adaptive traffic control at work zones. Adaptive traffic control is an essential component of an Intelligent Transportation System (ITS). Therefore, the dynamic prediction model can

be incorporated into the future projects of the Indiana's ITS program to enhance the capability and efficiency of traffic control at Indiana freeway work zones.

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APPENDIX: INSTRUCTIONS ON USE OF COMPUTER PROGRAM *DelayCost*

The ***DelayCost*** program can be used to estimate the traffic delays and user costs at freeway work zones.

To Install the ***DelayCost*** Program:

1. If you have previously installed ***DelayCost*** program. Make sure you uninstall it first by deleting the direction c:\program files\DelayCost.
2. Place the installation CD in the CD-ROM drive.
3. Click the **Start** button, and then click **Run**.
4. In the Run dialog, type < CD-ROM drive letter>:**SETUP**
5. Click the **OK** button and follow the instructions on the screen to complete the installation.
6. In some cases (depending on the configuration of the computer), additional system files need to be installed. When this happens, the setup program will alert you about it. You will then be prompted to restart the computer after the installation of these system files.

To Run the ***DelayCost*** Program:

Click the **Start** button, then the Programs button, and finally the ***DelayCost*** button.

How the ***DelayCost*** Program Works:

1. When the program starts, you will be asked to select the units of speed and distance. The default units are mile/hour and mile, respectively. You will then be prompted to choose from two actions: 1)."*To start a new calculation*" or 2)."*To open a previously saved file*".

2. If you choose to start a new calculation, you will be prompted to enter the following information: *the road name, the work zone type, number of lanes in the traffic direction, number of lane(s) in the work zone, work zone length (in miles or kilometers, depending on the unit you have chosen earlier), work zone grade, the starting hour, the total number of hours*. In this program, the maximum number of hours is limited to 24. After all the information is entered, a table with grid will be popped up for you to fill. You need to enter the hourly traffic information including speed, traffic volume, percentage of trucks in the available lanes. If you are not sure about the speed, leave the cell blank. Default value will be assigned by the program, and you will have a chance to review and/or change the default value before calculation starts. Truck percentage should be entered as decimal, e.g. 0.24 for 24%. If you accidentally enter 24 instead of 0.24, the program will automatically change it to 0.24. You have an option to enter a single value of truck percentage for all hours and lanes. To do this, click <**Option**> from the menu bar, then click <**Enter a single truck percentage for all**>. The program will also check the validity of the speed you entered. If the speed you entered is too high in congested traffic, the program will suggest a lower value. You can opt to accept or deny the change. To fill a cell in the table, double click the cell, enter a value, and press the **Enter** key. You can also use the arrow keys to move around the table. After you finish entering the table, proceed to step 4.

3. If you choose to open a previously saved file in step 1. All procedures described in step 2 are skipped. You will have immediate access to the main program interface. Click the <**File**> in the menu bar and then click <**Open Hourly Data**>. You will be prompted to select the file you need to open. Only files that are saved during previous calculation in the program can be retrieved. After you open the file, all the necessary information (including the data table) is filled automatically.

4. You have the option to change the default unit cost of time for cars (Uc) and trucks (Ut). To do this, just click <**Option**> and then <**Change unit cost of time**>. Enter new values and hit the OK button. You can always change the values back to the default by clicking the reset button. You also have the option to adjust the default capacity values. To do this, click <**Option**> and then <**Change default work zone capacities**>. To start the calculation, click <**Run**> and then <**Calculation**>. The program will first check the validity of the data you entered. If no problems are found, the program will ask if you would like to save the data in a file. If you choose yes, then you will be prompted to enter a file name and directory where you want the data to be saved. Once saved, the next time you run the program, you can load the data from the file and the corresponding table will be filled automatically.

5. After the calculation is completed, the result is displayed in a pop-up window. The window can be closed by clicking the **OK** button. The window can be opened again by clicking <**Result**> and then <**Show**> in the menu bar. To save the result in a file, click <**File**> then <**Save Results**>. You can also save the input data by clicking <**File**> and <**Save hourly data**>. You can print the current view of the program by clicking <**File**> and <**Print**>.

6. To exit the program after you completed your calculation and saved the result, click <**File**> and then <**Exit**>.