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# QUEUE AND USER'S COSTS IN HIGHWAY WORK ZONES

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16. Abstract

The IDOT Bureau of Design and Environment (BDE) Manual requires that traffic control plans for freeway reconstruction projects include a queuing analysis to determine the anticipated traffic backups in work zones. Queue length and delay calculations rely on the estimation of capacity and operating speed. In this study, field data were collected from five work zones in Illinois. Thirteen data sets were extracted from the field data. Each data set represents a particular traffic condition at a given site. A work zone capacity value was suggested for each traffic condition based on the field data. The suggested capacity for the sites with speed limit of 45 mph ranges from 1200 pcphpl (passenger cars per hour per lane) to 1550 pcphpl. The 1200 value was suggested for a traffic condition with flagger and queue, and 1550 value was for a traffic condition with no work activity, no speed management treatment and no queue. A capacity of 1600 pcphpl was suggested for a site with speed limit of 55 mph, dynamic speed feedback sign, no work activity, and no queue; and a capacity value of 1750 pcphplwas recommended for a short-distance work zone with speed limit of 55 mph, no work activity, and no queue. Using the field data, speed-flow curves were proposed for: work zones with speed limit of 45 mph and a flagger, work zones with speed limit of 45 mph and without a flagger, and work zones with speed limit of 55 mph. Each of these models can be adjusted to non-ideal conditions. Methods to estimate the length of moving queue, delay and users' cost were developed to handle the cases where a demand higher than capacity causes queue. The queue length and delay were estimated for all the data sets using the proposed method. The results also were compared with the QuickZone 2 outputs. When the arrival volume in an interval was less than the capacity of the interval, the QuickZone2 did not yield any delay or queue length even though there was congestion and delay in a part of the interval.

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## EXECUTIVE SUMMARY

The IDOT Bureau of Design and Environment (BDE) Manual requires that traffic control plans for freeway reconstruction projects include a queuing analysis to determine the anticipated traffic backups in work zones. The results of the queuing analysis are used when deciding the hours of work zone operation (i.e. peak, off peak, day time, nighttime), selecting detours, making temporary capacity improvements, or providing real-time information to motorists. To reduce delay and inconvenience to motorists, contractual procedures such as lane rental and incentive/disincentive are used to shorten the duration of construction time. The monetary gains/losses in the contractual procedures depend, to a large degree, on the results of the queuing analysis. Thus, accurate prediction of queue length and average motorist delay are critical issues. To determine the motorist delay and users' costs in work zones, one needs to know travel delay and queue delay. The travel delay depends on the operating speed of traffic whereas the queue delay depends on queue length and its duration. Both the operating speed and the queue length depend on work zone capacity. Thus, accurate determination of work zone capacity is a critical step in estimating road users' costs.

In order to estimate capacity, field data were collected from five work zone sites in Illinois. Thirteen data sets were extracted from these sites (five with queuing condition and eight with non-queuing condition). The sites were two-to-one lane work zones (i.e. one of the two lanes was closed) with different prevailing conditions. Based on field data, capacity values were suggested for different work zone conditions. Also, speed-flow curves were developed for three types of work zones: work zones with speed limit of 45 mph and a flagger, work zones with speed limit of 45 mph and without a flagger, and work zones with speed limit of 55 mph.

Each of these models can be adjusted to account for non-ideal conditions. A work zone with ideal conditions is defined as a work zone with ideal geometric conditions, no speed management treatment and no work activity. Work zone operating speed and capacity are estimated using the appropriate speed-flow curve and are then used to compute delay and queue length. A methodology was also proposed to estimate moving queue length and the corresponding delay. It was also discussed how the proposed methodology should be applied when a combination of stopped and moving queue exists in the field. Queue length and delay were estimated for all the data sets using the proposed method. The results were then compared with the QuickZone 2 outputs. A step-by-step algorithm was developed to find the queue length, delay, and users' cost in work zones. The algorithm is based on the speed-flow curves established by the field data and the proposed methodology to estimate moving queue length and delay.

The findings of the study are:

- The suggested capacity values for the sites with speed limit of 45 mph ranges from 1200 pcphpl (passenger cars per hour per lane) to 1550 pcphpl. The 1200 capacity value was for a traffic condition with flagger and queue, and the 1550 capacity value was for a traffic condition with no work activity, no speed management treatment, and no queue.
- The suggested capacity values for the sites with speed limit of 55 mph ranged from 1600 pcphpl to 1750 pcphpl. The 1600 capacity value was for a site with dynamic speed-feedback sign, no work activity, and no queue, and the 1750 capacity value was for a short-distance work zone with speed limit of 55 mph, no work activity, and no queue. A short-distance work zone is a work zone where drivers are able to see

the end of the work zone when they enter the transition area. Consequently, it requires less-than-a-minute travel time at the posted speed limit to exit the work zone.

- When the arrival volume in an interval was less than the capacity of the interval, the QuickZone2 did not yield any delay or queue length even though there was congestion and delay in a part of the interval.

## RECOMMENDATION FOR FUTURE RESEARCH

The following issues need to be studied:

- The proposed speed-flow curves are based on the data collected from two-to-one-lane work zones. A similar study is recommended for work zones with other types of lane configurations.
- Although the proposed method suggested an interim solution for the effect of some of ITS on work zone capacity, a more specific study on this issue is needed.
- Effect of flow breakdown on work zone capacity and operating speed needs to be investigated.
- Under what circumstances stop-and-go condition occurs in work zones and how it affects departure rate and operating speed needs to be determined.
- A computer program (software) for the proposed approach needs to be written to make it easier for the users and to avoid the errors that may occur with manual calculations.
- The accuracy of delay and queue length estimation is related to the accuracy of capacity estimation. Passenger car equivalent (PCE) for heavy vehicles is one of the elements used in capacity calculation. Currently, the method relies on the PCE values suggested by the HCM 2000 for basic freeway sections. A specific study for work zones is recommended.

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# CHAPTER 1 INTRODUCTION

The Illinois Department of Transportation (IDOT) Bureau of Design and Environment (BDE) Manual requires that traffic control plans for freeway reconstruction projects include a queuing analysis to determine the anticipated traffic backups in work zones. The results of the queuing analysis are used in deciding hours of work zone operation (peak, off peak, day time, nighttime), in selecting detours, making temporary capacity improvements, or providing real-time information to motorists. To reduce delay and inconvenience to motorists, contractual procedures such as lane rental and incentive/disincentive are used to shorten the duration of construction time. The monetary gains/losses in the contractual procedures depend, to a large degree, on the results of the queuing analysis. Thus, accurately predicting the length of queues and average motorist delay become critical issues. To determine user's costs in work zones, one needs to know the travel delay and queue delay. Travel delay depends on the operating speed of traffic, and queue delay depends on queue length and its duration. The values computed for speed and queue length depend on capacity of the work zone. Thus, accurately determining work zone capacity is a critical step in road user's costs calculations.

Work zone configurations are normally different than basic highway sections. The number of available lanes might be reduced in a particular section of highway and workers might be working close to the travel lanes. Geometric conditions may be more restricted compared to the basic highway sections. Therefore, the capacity of highway work zones is normally less than the capacity of basic freeway sections, and that may cause queuing. Research is needed to investigate and quantify possible causes of queuing. All the factors which result in capacity reduction are potential causes of queuing. Examples of these factors are: less-than-ideal lane width and lateral clearance, work intensity, and lane closures.

The objectives of this study are to develop procedures for estimating speed, capacity, delay, queuing, and user's costs for interstate highway work zones where queuing and congestion occurs (The work zones are oversaturated at least for some time period). The study developed speed-flow relationship for work zones under different prevailing conditions. It also developed delay and queue length estimation methods for moving queue. It compared filed data to the results from QuickZone2. The study also developed a method to estimate user's costs in work zones.

This report includes 11 chapters. Chapter 1 introduces and presents the objectives of the study. In chapter 2 causes and effects of queuing are discussed. The queuing conditions and factors which cause a v/c ratio greater than 1 are presented. Chapter 3 reviews the recent studies on work zones capacity estimation. Different definitions of capacity in literature and the models proposed to estimate work zone capacity are so presented. Then the parameters affecting work zone capacity are discussed. In chapter 4, detailed information about data collection and reduction is given. Data were collected from six sites. Speed, vehicle type, and headway were the basic information extracted from field data. Three sites had queues; two sites had volumes near capacity but no queues; and one site had traffic volume less than capacity.

Chapter 5 estimates the capacity of work zones based on field data. Several methods of capacity estimation were investigated and a method was proposed. In Chapter 6, speed-flow relationships for three different types of work zones were developed. They are: work zones with speed limit of 45 mph and flagger, work zones with speed limit of 45 and without flagger, work zones with speed limit of 55 mph. Application of the speed-flow relationships are discussed in Chapter 7. Operating speed and capacity are determined using the speed-flow relationships. Chapter 8 proposes a model to estimate delay, queue length, and user's cost for moving queues. In Chapter 9, QuickZone2 outputs are compared with delay and queue length based on the field data. In chapter 10, a step-by-step procedure is proposed to estimate capacity, delay, queue length and user's costs in work zones. An example problem is provided to illustrate the proposed method. Chapter 11 contains the conclusions and recommendations of the study.

# CHAPTER 2 CAUSES AND EFFECTS OF QUEUING IN WORK ZONES

It is estimated that 15% of the total congestion on U.S. highways stems from work zones (American Highway Users Alliance, 1999-2004). More specifically, around 24% of non-recurring delay results from work zones on freeways (U.S. Department of Energy, 2002). The congestion in U.S. work zones was observed to lead to around 60 million vehicles per hour per day of capacity loss over a two-week period when the summer roadwork season was in its peak in 2001 (U.S. Department of Transportation, 2002). Moreover, in 2003, around 480 million vehicle-hours of delay were brought about by roughly 7,200 U.S. work zones (U.S. Department of Transportation, 2009). Traffic congestion and queuing in work zones occur when traffic demand exceeds the vehicle processing capability. Congestion and queue in work zones happen when arriving volume exceeds the capacity. In general, three conditions would yield a v/c (volume-to-capacity) ratio greater than 1: Condition A: Demand increases, but capacity remains constant or decreases; Condition B: Demand is constant, but capacity decreases; or Condition C: Both demand and capacity decrease, but the rate of decrease in capacity is higher.

## 2.1 CONDITION A

When demand increases beyond the capacity of a work zone, congestion and queue will develop. Examples of the increases in demand are:

- Traffic during peak hour
- High volume weekend traffic
- Special event traffic (e.g. football game, concert, etc.)
- Holiday traffic
- Increase in the percentage of heavy vehicles

In some cases, the increase in demand may also cause a decrease in capacity. When demand exceeds capacity, a small disturbance in traffic stream may propagate through the line of vehicles and make them slow down. Once traffic slows down, that may cause capacity reductions as well. This can happen without any change to work zone layout and geometry.

## 2.2 CONDITION B

Demand remains constant but capacity decreases due to changes in geometric, traffic, weather, or operating conditions. Various factors can lead to reduction in work zone capacity, and they can be broadly categorized as: traffic-related factors, work zone activity-related factors, geometric factors, and weather-related factors.

### 2.2.1 Traffic-Related Factors

Traffic-related factors can be caused by slower moving vehicles. Examples are:

- Change in driver population: Even though the traffic demand does not change, an increase in the percentage of unfamiliar/ recreational drivers can reduce the speed and thus capacity drop can occur.
- Slow moving vehicles/ over-weight and over-dimension vehicles: Some motorists travel slower than the speed the work zone conditions allow. Likewise, over-weight and over-dimension vehicles may travel slower than the general traffic stream.

Consequently, capacity drops and congestion and queue will develop, especially if there is no opportunity for passing.

### 2.2.2 Work Zone-Related Factors

Examples of work zone related factors are:

- Work activity: In general, motorists tend to slow down when they see workers in the work zone. The higher the work intensity, the slower motorists tend to travel. As motorists reduce their speed, work zone operating capacity may also decrease and that may cause queuing in the work zone.
- Construction equipment entering/ exiting the closed lanes: Construction equipment entering or leaving the closed lanes in work zone may cause speed reduction for the general motorist on the open lanes and that may temporarily reduce capacity.
- Presence of flagger/ police: When a flagger is present, vehicles usually slow down in response to the flagger. The speed reduction may also cause a reduction in work zone operating capacity. The reductions are more significant when the flagger acts aggressively. Likewise, police presence may reduce work zone speed that can lead to some capacity drop.

### 2.2.3 Geometric Factors

Examples of geometric related factors are:

- Less-than-ideal lane width/ lateral clearance: A less-than-ideal lane width (i.e. less than 12 ft) may reduce speed and/or increase headway which may cause a reduction in operating capacity. A less-than-ideal lateral clearance can cause similar effects.
- Lane availability: Most of the times, work zones involve lane closure that significantly reduces the total available capacity. Besides, if only one lane per direction is open in the work zone, the passing opportunity within the work zone is eliminated. Thus, queues may back-up in the work zone depending on the traffic demand.
- Edge drop and uneven pavement: Speed reduction can also result from edge drop and uneven pavement, and consequently, the capacity of the work zone can be lowered.

### 2.2.4 Weather-Related Factors

Examples of weather-related factors are:

- Precipitation/ icy pavement: Rainfall, snowfall, and frost action lead to more slippery pavement and as a result, motorists tend to drive cautiously at lower speeds. The lower speed may result in lower capacity.
- Fog: Fog causes poor visibility conditions which can slow down motorists. Similar to the case of precipitation, fog can also reduce speeds and introduce some drop in work zone capacity.

## 2.3 CONDITION C

Both demand and capacity decrease, but the rate of decrease in capacity is higher than the rate of decrease in demand. Some of the cases in which reduction in traffic demand is observed are as follows:

- Some of the incoming traffic is diverted to alternate routes.
- The traffic transitions from peak period to off-peak period.

- The total traffic volume does not change significantly, but the percentage of heavy vehicles decreases.

For instance, consider the case when some of the vehicles approaching the work zone are diverted to alternate routes, but the work zone capacity is still not enough to handle the reduced demand. Then queuing can occur in the work zone as a result of unmet traffic demand. Another example for condition C would be heavy precipitation occurring after the peak period of traffic flow as the traffic transitions from peak period to off-peak period, traffic demand decreases. However, the reduction in work zone capacity due to the heavy precipitation may be higher than the decrease in traffic demand. Hence, queues may form in the work zone.

# CHAPTER 3 METHODS FOR DETERMINING CAPACITY

In this chapter, articles related to work zone capacity will be reviewed in five sections. First, different definitions used to determine the capacity are presented. In the second section, effects of several factors on work zone capacity are presented. In the third section, models that were proposed in recent years for work zone capacity estimation are reviewed. Following that, those studies are analyzed, and finally, work zone capacity values captured in different studies are reported.

## 3.1 CAPACITY DEFINITIONS

Researchers did not use a unique definition to estimate work zone capacity. The following definitions have been used in previous literature:

- “The discharge flow when there is a continuous flow of traffic.” (Benekohal et al., 2004)
- “The traffic flow rate just before a sharp speed drop followed by a sustained period of low vehicle speed and fluctuating traffic flow rate.” (Jiang, 1999)
- “The mean queue discharge flow rate from the bottleneck that was located at the end of the transition area.” (Al-Kaisy et al., 2000)
- “95th percentile value of all 5-min within-a-queue” flow rate (Dixon et al., 1996).
- “The average volume of the ten highest volumes immediately before and after queuing conditions” (Maze et al., 2000)

It is evident that some of these definitions are based on the mean traffic flow rate whereas the others are based on the maximum observed values. Some definitions give queue discharge rate while the others estimate maximum flow that can be processed before and after flow breakdown.

## 3.2 FACTORS AFFECTING CAPACITY OF WORK ZONES

Traffic, geometric, construction and environmental characteristics of a work zone may affect its capacity. Researchers tried to quantify the effects of these parameters. This section recaps the main points of recent studies.

Dixon et al. (1996) investigated 24 short-term work zones located in urban and rural freeways. Data were collected from different types of work zones including 2-to-1 lane, 3-to-1 lane, 3-to-2 lane, and crossover for a freeway with two lanes in each direction. Physical conditions were recorded by using two methods. In the first method, road conditions and odometer readings were recorded by filming at “critical locations, including sign placement, transition location, and active work location”. In the second method, other physical conditions such as the proximity of a ramp to the transition and work activity area and whether it was a left-merge or right-merge work zone were recorded manually. Five-minute traffic data, including number of vehicles, distribution of vehicles across lanes, percentage of heavy vehicles, and average speed of vehicles were recorded by “Vehicle Magnetic Imaging traffic counters and classifiers”. Classifiers were located at the end of the transition area, and adjacent to the work activity area. Counters were located at the beginning of the transition area and an advance warning location. Authors defined capacity for the end transition area as “95th percentile value of all 5-min within-a-queue observations” and reported it in pcphpl by using PCE factors proposed in HCM 94 (PCE for trucks on level terrain was 1.5) . However, they observed that some trucks tended to move parallel to each other, and this behavior created an unusable area in the front of the trucks before the transition area. Such behaviors affect the normal operation of heavy vehicles and passenger cars. Analyses showed that behavior of drivers in queue conditions were the same during day and night. Variations in capacity for the work activity area were larger than the end-transition area. In rural work zones, mean capacity for the work activity area was smaller than the end-transition area. There was no significant difference between these values for urban work zones. Mean capacity of the transition area for rural work zones (1454 pcphpl) was less than urban work zones (1743pcphpl).

In the research conducted by Jiang (1999), traffic volume, speed and vehicle type were collected from four work zones that experienced the queue conditions. Using traffic counters with road tubes, traffic data were recorded at five-minute intervals for "high-traffic-volume" and at one-hour intervals for "low-traffic-volume". These devices were located before the transition area, in the transition area, and close to the work activity area. Free-flow traffic, merging traffic, and work zone traffic were extracted from the raw data. The work zone capacity was defined as "the traffic flow rate just before a sharp speed drop followed by a sustained period of low vehicle speed and fluctuating traffic flow rate." To detect the capacity, time series diagrams for speed and flow rate were plotted in one graph. Based on the capacity definition, Jiang determined the capacities of these sites as well as the queue discharge rates. The capacities of these sites were not statistically different from each other. The mean queue-discharge rate was statistically lower than the capacity. The results were reported in pcphpl, using PCE factors suggested by HCM 1994. (PCE for trucks on level terrain was 1.5).

In order to analyze a work zone located on an interstate highway, Maze et al. (2000) used 15-minute traffic data including volume, speed, and density. For the purpose of data collection, two video cameras were installed before the transition area and at the end-transition area. Traffic encountered queuing condition during only four out of 19 days of data collection. In order to compute the queue discharge flow rate, the queue length was recorded in each minute using delineators that were placed at the side of the road. The queue length was reported to the nearest 0.05 mile. Time series diagram for the speed and flow rate data indicated that there was no significant difference in the amount of flow rate just before and after queuing conditions. So "The average volume of the ten highest volumes immediately before and after queuing conditions" was considered as the maximum capacity of lane closure. Data showed that capacity of a work zone located in a rural highway varied from 1400 pcphpl to 1600 pcphpl.

Al-Kaisy and Zahou (2000) studied two long-term work zones located on either side of a six-lane freeway. One lane was closed due to construction in each work zone, and both work zones had a lane drop of 3-to-2. Lane closures were located at the median and shoulder lane in the downgrade and upgrade direction, respectively. Al-Kaisy et al. defined work zone capacity as "the mean queue discharge flow rate from the bottleneck that was located at the end of the transition area". Detectors collected space-mean-speed and occupancy within 20-sec intervals. By aggregation of these data, queue discharge flow rate during each 15-min interval was obtained. The capacity was found to vary between 1750 and 2150 vphpl and the mean capacity was equal to 1943 vphpl. Due to the lack of information about the vehicle type and the percentage of heavy vehicles, the capacity values were not been reported in pcphpl. The work zone capacity during peak hours (2117 vphpl) was statistically greater than those during off-peak hours (1955 vphpl). In addition, variation of work zone capacity during peak-hours was less than during the remaining hours. Variation of weekend capacity was small. The mean work zone capacity for weekend was less than that for weekdays. However, it was mentioned that adverse weather conditions existed during the weekend.

Al-Kaisy and Hall (2000) used 5-min traffic volume data recorded by video from two sites to investigate the effect of darkness on work zone capacity. One of these work zones was 800m in length with right-side lane closure, one lane open during construction, and insignificant grade. The other site was 500m long and lane-closures were placed at both the right and the left side of the roadway where the work zone was located. The middle lane was open for traffic during construction. There was also an upgrade and a down grade section. Ideal lane width, sufficient light during night, presence of an off-ramp at the downstream of the work zones and presence of queue at PM peak hours were the conditions in both sites. Data were recorded during the PM peak hours in weekdays. Capacity reduction due to the darkness for the first and second site was equal to 7.5% and 3.25%, respectively. It should be noted that the capacities were converted from vphpl to pcphpl by using PCE factors proposed by HCM 1997 (PCE for trucks on level terrain was 1.5).

In order to investigate the effect of driver familiarity, Al-Kaisy et al. (2001) selected two long-term work zones located in each direction of an eight-lane freeway. Two of these lanes were closed in each direction due to construction activity. It should be noted that these sites were located over a bridge. The grades of this bridge in upgrade and downgrade sections were equal to 3% and the length of each section was equal to 1 km. Volume, vehicle type and queue presence were recorded at 15-min intervals by detectors in spring season. Other information such as incidence occurrence, weather conditions and work activity was also recorded. Data were classified into three groups: a.m. peak and p.m. peak for weekdays and weekend data. Capacity values were statistically different for these groups. A population factor of one was assigned to a.m. peak hour traffic. Based on this assumption, population factors of 0.93 and 0.84 were suggested for p.m. peak and weekend, respectively.

### 3.3 ESTIMATION OF CAPACITY

Benekohal et al. (2004) analyzed 11 work zones located in interstate highways with two lanes in each direction, but one lane was closed. Three sites were short-term work zones and eight were long-term work zones. Four types of data called as general, geometric, traffic and construction data were collected. Location of work zone, type of traffic (inbound or outbound), weather conditions, police presence and flagger presence belong to the general data. Geometric data consisted of lane width, total number of lanes in each direction, number of open lanes, presence of ramps close to work zone, length of lane closure, position of closed lane, and length of work activity area. The following work activity information was collected: type of work activity, number of workers present, number and size of construction equipment, proximity of work activity to the travel lane in use, and traffic control devices used were recorded. Traffic data included the headway and speed of each vehicle in the work zone, volume of traffic, and queue length. Data related to general conditions, geometry, work activity and how they varied during the data collection period were recorded by field observation. In most of the sites, in order to collect the traffic data, two markers apart from each other with an approximate distance of around 250 ft were used. A camera recorded the traffic stream between these markers. In three sites due to heavy traffic stream, markers could not be installed, so two observers for speed data and one observer for queue data (i.e. presence of queue and length of the queue) were used. Data collection period varied from 2 to 4 hours. Vehicle type, time at each marker, and whether a vehicle was in platoon or not was obtained from time coded videotapes. The accuracy of the reading of travel time was within 1/30 second. In order to measure headway of a particular vehicle, the time at which the front bumper of the vehicle passed the more visible marker was recorded. The corresponding time difference between two successive vehicles gives the headway of the following vehicle. The distance between two markers was measured and the actual distance between the two markers that vehicles traveled was computed considering the angle between camcorder and the markers.

Benekohal et al. defined capacity as “the discharge flow when there is a continuous flow of traffic.” In order to have continuous flow of traffic, platooning vehicles which had a spacing less than or equal to 250 ft or did not have headway greater than 4 seconds were considered. Queuing condition was observed in three sites. For these sites, the top 15-min intervals were selected to measure the headways whereas for the others, the top 5-min intervals were used. For each site, service capacity was computed as the reciprocal of the average headway. It should be noted that the average speed of platooning vehicles in these periods was very close to the average speed of all vehicles including non-platooning and platooning.

A speed-flow relation was developed based on the field data. Knowing the operating speed of traffic, one can use the speed-flow relation to estimate the capacity under the prevailing conditions. Benekohal et al. suggested the following model to compute the operating speed:

$$U_o = FFS-R_{WI}-R_{LW}-R_{LC}-R_o \tag{3.1}$$

Where
$U_o$ = Operating speed (mph)
FFS = Free-flow speed (FFS=speed limit+5 mph when there are no field data)
$R_{LW}$ = Reduction in speed due to lane width (mph), based on HCM 2000,
$R_{LC}$ = Reduction in speed due to lateral clearance (mph), based on HCM 2000,
$R_{WI}$ = Reduction in speed due to work intensity (mph), and
$R_o$ = Reduction in speed due to all other factors that may reduce speed (mph) (if there is no relevant information, it is equal to zero).

For speed reduction due to work intensity, authors proposed a model based on 90 accurate and consistent observations for short-term work zones. This model is:

$$SR_s = 11.918 + 2.676 \ln (WI_r) \tag{3.2}$$

Where $SR_s$ (mph) is the speed reduction in short-term work zones and $WI_r$ is the work intensity ratio that is computed as follows:

$$WI_r = \frac{w + e}{p} \tag{3.3}$$

Where
w = Number of workers in the active work area (varies from 0 to a maximum of 10),
e = Number of large equipment in the active work area (varies from 0 to a maximum of 5), and
p = Distance between the active work area and open lane (ft) (varies from 1 to a maximum of 9 ft).

A similar model was developed for long-term work zones as follows:

$$SR_L = 2.6625 + 1.2056 \ln (WI_r) \tag{3.4}$$

Where $SR_L$ is the speed reduction (mph) in long-term work zones and $WI_r$ is the work intensity ratio.

After determining the operating speed, capacity at this speed is determined from the speed-flow model. In the proposed model, the speed is equal to FFS at flow rates below 1300 pcphpl. At flow rates that fall between 1300 and the capacity, the following equation was developed:

(3.5)

In the above-mentioned formula, $U_C$ is the operating speed at capacity. In the case of congestion, hourly flow rates and average speeds under continuous discharge flow conditions were obtained at 5-min intervals. These data gave the following relation:

(3.6)

Where q is the flow in pcphpl and U is the speed in mph. For this relation, $R^2$ was equal to 0.6891. Based on the equations gotten from the field data, the following speed-flow curves were developed:

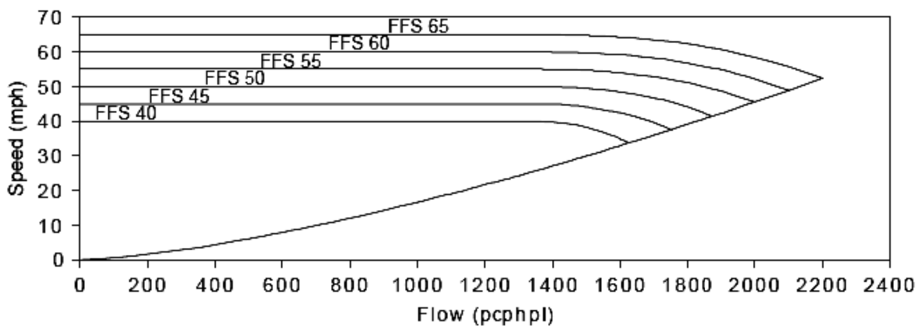


Figure 3-1. Speed-flow curves for work zones proposed by Benekohal et al. (2004).

Finally, work zone capacity is estimated by using the following relation:

$$C_{adj} = C \times f_{HV} \times PF \qquad (3.7)$$

where

$C_{adj}$ = adjusted capacity (vphpl) ,

$C_{U_o}$ = capacity at operating speed $U_o$ (pcphpl) ,

$f_{HV}$ = Heavy vehicle adjustment factor computed from HCM 2000,

$PF = 1$ when there is sufficient demand and all vehicles are in platoon.

The authors suggest that when there is sufficient demand but not all vehicles are in a platoon, a lower PF, not less than 0.75, should be chosen.

Analysis of data from seven sites without flow breakdown showed that the speeds estimated from the speed-flow relation were close to the operating speed computed from the field data. The maximum and minimum difference between the observed and estimated speeds was 5.71 and -5.19 mph, respectively while the average difference was -0.37 mph. In addition, a paired t-test showed that these values were statistically the same at 5% significance level. For the remaining sites, observed speed data were used as the input variable to estimate the capacity. The difference between the service capacity and estimated capacity ranged from -363 to +169 pcphpl with an average equal to -98 pcphpl. These differences were not statistically significant at 5% significance level.

Sarasua et al. (2004) conducted a study on 22 short-term work zones in South Carolina’s interstate highways. All sites had 12-foot lane width and similar taper length. In this study, two video cameras were used to record the traffic volume and vehicle type.

These cameras covered “taper and lane closure transition”. Speed was measured by radar gun. When the vehicle speeds were greater than 35 mph, the speed data were aggregated over 5-min intervals. When vehicle speeds were below 35 mph, the speed data were aggregated over 1-min intervals. At the same time interval, queue length was recorded in feet. Queue length was measured from the beginning of the taper by using visible markers. These markers could be easily detected by the camera. Four sites had a queue length more than 1 mile whereas 10 sites did not have any measurable queue length. There were three types of lane drop including 2-to-1, 3-to-2, and 4-to-2. To improve the data range, the data for the sites that had the same configuration were combined. Since data were not enough for the other cases, this study concentrated on the sites with two lanes open under normal conditions.

Satflo2 is the program utilized to extract the time and type of each vehicle. Using these data, headways and the consequent PCE were computed. Values of 1.44 for recreational vehicles (RVs) including bus, passenger cars with trailer and straight truck, and 1.93 for heavy trucks were computed as the average PCE for rolling terrain. The sample size for passenger cars, RVs, and heavy truck was equal to 11,423, 505, and 2,246, respectively.

Based on 1-hr data, a linear speed-density relationship was developed with $R^2$ of 0.93. Therefore, the following model was suggested to represent the variation of flow with respect to speed.

$$q=-2.05\,s^2 + 109.7\,s \tag{3.8}$$

From this model, a value of 1,467 pcphpl was determined as the capacity of 2-to-1 work zones. In the meantime, the data were not sufficient to investigate the effects of grade and adverse weather on the capacity. Length of work zone and degree of activity were represented by two dummy variables in a regression model, but the model did not show any significant effect.

Finally, the following model was suggested to estimate the capacity:

$$C_{WZ} = (1460+I)*f_{HV}*N \tag{3.9}$$

where

$C_{WZ}$ = “Estimated capacity of short-term work zone (veh/h)”,

$f_{HV}$ = “Heavy vehicle adjustment factor”,

N = “Number of lanes open through the work zone”,

I = “Adjustment factor for type, intensity, length, and location of the work activity (A range of “I” from -146 to +146 is suggested for South Carolina interstate work zones)”.

Al-Kaisy et al. (2003) analyzed long-term work zones located in six freeways. Capacity was defined as “the mean queue discharge flow rate”. The value of 2000 pcphpl was suggested as the base capacity which was computed for the following ideal conditions: familiar drivers, daytime, no work activity on site, good weather, level terrain and 12-foot lane width. The PCE values suggested for heavy vehicles were equal to 2.4 and 3.0 for level terrain and for a 3% one-kilometer upgrade, respectively. A capacity reduction of 7% was suggested for off-peak hours compared to peak hours whereas a capacity reduction of 16% was proposed for weekends compared to weekday peak hours. A capacity decrease of 5% due to darkness was observed compared to good lighting conditions. The capacities of work zones with right-side lane closure were higher by around 6% than those with left-side lane closure. A capacity reduction of 4.4% to 7.8% was suggested for the effect of light rain. A range of capacity reduction from 1.85% to 12.5% was reported due to the effect of work activity. Using the collected data, a multiplicative and an additive model were developed to estimate the capacity. Finally, based on the results from those models and the engineering judgment of the authors, the following model was proposed:

$$C=C_b \times f_{HV} \times f_d \times f_w \times f_s \times f_r \times f_l \times f_i \qquad (3.10)$$

where $C_b$ is the base capacity (2000 pcphpl) and C is the work zone capacity (vphpl).

Other adjustment factors and their recommended values are as follows:

$f_{HV}$= Adjustment factor for heavy vehicles which is computed from the relevant formula proposed by HCM 2000. PCE for trucks and buses was suggested as 2.4 for level terrain and 3.0 for 3% upgrade with 1-km length. Linear interpolation was suggested for 1-km long grades which fall between level terrain and +3%. For other grades with similar lengths, an adjustment proportional to the values proposed by HCM 2000 was suggested.

$f_d$= Adjustment factor due to driver population (1 for weekday peak-hours,0.93 for weekday off-peak hours and 0.84 for weekends)

$f_w$= Adjustment factor for work activity (1.00 if there is no work activity at site; otherwise, 0.93 is used)

$f_s$= Adjustment factor for the side of lane closure (1.00 if there is right-side closure, 0.94 if there is left-side closure)

$f_r$=Adjustment factor for rain (1.00 if there is no rain, 0.95 for light to moderate rain, 0.90 for heavy rain)

$f_l$=Adjustment factor the effect of the Lighting conditions (1 for daytime and 0.96 for night with good Lighting conditions)

$f_i$=Interactive effects (1.03 for left-side lane closure during weekday off-peak hours, 1.08 for weekends when work activity is present, 1.02 for left-side closures during weekends, 1.05 for rain during weekends, 1.00 for all other conditions)

To investigate the factors affecting work zone capacity, Venugopal and Tarko (2001) collected data from a long-term work zone with heavy work activity. This work zone was located on a four-lane rural freeway. There was a left-side lane closure in one direction and a right-side lane closure in the other direction of the freeway. The investigated factors were rain, wind, heavy vehicles, the location of lane closure (shoulder or median) police presence and “Indiana Late Merge System (ILMS)”. ILMS is a new control device “to improve the use of traffic lanes on work zone approaches”. Speed, volume and vehicle type were aggregated over 20-min intervals. Loop detectors were deployed after the end-transition area to collect traffic volume data. These devices were used before the transition area and after the end-transition area to record the speed data. All detectors could detect the type of each vehicle. Wind speed and weather conditions (rainy or sunny) were obtained from the “Earth and Atmospheric Sciences Department of Purdue University”. Data were collected in two states: presence and absence of ILMS. Authors did not gather data between these states for two weeks because they believed that two weeks were needed for drivers to be familiar with new conditions. Congestion was observed only at weekends. To detect capacity conditions, time-series diagram for speed and flow rate was plotted. A sharp drop in speed with a considerable period of low speed indicated capacity conditions. Capacity values varied in a wide range. The mean capacity value was equal to 1320 veh/hr. Analysis of Covariance (ANCOVA) indicated that ILMS, weather conditions (rainy or sunny), heavy vehicles and presence of police were the only effective variables on the work zone capacity at 5% significance level. The highest wind speed recorded during data collection period was equal to 24 km/hr which was not very strong. Based on these results, both an additive model and a multiplicative model were proposed. Both of them gave similar results. Therefore, only the multiplicative model is presented as below:

$$C = C_0 \times f_M \times f_R \times f_H \times f_P \qquad (3.11)$$

where:
C= Work zone capacity under existing conditions,
$C_0$ = Work zone capacity under ideal conditions (no ILMS, absence of police, no truck and no rain),
$f_M$ = Adjustment factor for presence of ILMS,
$f_R$ = Adjustment factor for rainy weather,
$f_P$ = Adjustment factor for police presence,
$f_H$ = Adjustment factor for heavy vehicles

Following the calibration of the model, a value of 1440 veh/hr was obtained for $C_0$ and a value of 1.4 was reported for $E_H$ (PCE value for heavy vehicles) on level terrain. The values of $f_M$, $f_R$ and $f_P$ were equal to 0.94, 0.91 and 0.86, respectively. PCE for trucks and adjustment factor for rain were close to the values proposed by HCM for basic freeways that do not have work zone. However, the authors added that the obtained PCE value for trucks on level terrain might be underestimated because during some intervals, flow breakdown was observed in the traffic stream due to the presence of trucks. They also expected that ILMS and police presence would increase the capacity but the results did not show such effects. Overall, police was present for 13 days at the site during the data collection period (4 months). They noted that this number of observations might not be sufficient to get a significant result regarding the police presence. During the presence of police, motorists drove more cautiously. They also mentioned that a 2-week period might not be sufficient for drivers to become familiar with the new conditions (i.e. presence or absence of ILMS) because congestion occurred only during weekends when most of the users were not commuters. The data collection procedure was highly expensive ($20,000) for this site and therefore, they did not extend their studies to other locations.

Kim et al. (2001) developed a new model to estimate capacity for short-term work zones. To reach this point, 12 sites were selected. All sites were located on eight-lane freeways. The following factors were enumerated as the parameters which may affect the work zone capacity: number of closed lanes and open lanes, location of lane closure (shoulder or median), percentage of heavy vehicles, familiarity of drivers with the work zone, presence of on-ramp near the work zone, lateral clearance, length and grade of the work zone, severity of work activity (low, medium, and heavy), the length of time that work zone exists (short-term or long-term), weather conditions, and work time (night or day time).

Traffic data were collected after p.m. and a.m. peak hours, so it was assumed that drivers were not familiar with the sites. To record volume data, a video camera was installed after the end-transition area. A laser speed gun was used to collect speed data at 1-min intervals. Queues were observed only at upstream of work zones. Vehicles were classified into passenger cars and heavy trucks. In addition to the traffic data, characteristics of work zones were also recorded. These characteristics zone and the corresponding capacity values for each work are shown in the next section. Intensity of work activity for each site was indicated with regard to the type of construction, number of workers and size of equipment. Based on the results of the correlation matrix between the factors, which may affect the capacity of work zones, the following model was proposed to estimate work zone capacity:

$$\text{CAPACITY} = 1857 - 168.1\text{NUMCL} - 37.0\text{LOCCL} - 9.0\text{HV} \qquad (3.12)$$
$$+\ 92.7\text{LD} - 34.3\text{WL} - 106.1\text{WI}_H - 2.3\text{WG*HV} \quad (R^2 = 0.993)$$

Where:

CAPACITY = Work zone capacity in vphpl,
NUMCL = "Number of lane closures,"
LOCCL = "Dummy variable for the location of lane closure (in the case of right-side lane-closure, LOCCL = 1; otherwise, LOCCL =0),"
HV = "Proportion of heavy vehicles,"
LD = "Lateral distance to the open travel lane,"
WL ="Work zone length,"
$WI_H$ =" Dummy variable for heavy work activity (for heavy work activity, $WI_H$=1; otherwise, $WI_H$=0),"
WG*HV = Work zone grade*proportion of heavy vehicles.

The results indicated that CAPACITY had a high correlation with NUMCL, $WI_H$, and WG*HV.

Karim and Adeli (2003) proposed a model to estimate work zone capacity by using radial basis function neural network (RBFNN). The model was established based on the following factors: number of lanes, number of open lanes, work zone configuration, work zone length, lane width, percentage of trucks, grade, work zone speed, work intensity, darkness and presence of ramp 1500 ft before the transition area or 500 ft after the work zone. In this model, work intensity was determined with regard to the size of equipment, the number of equipment and workers, the amount of dust and noise generated and the distance of the work area to the traveled lane. To obtain data for the training of the model, authors used the capacity values proposed by ODOT for work zones with various characteristics, but before applying them, those data were modified with regard to the HCM 2000 guidelines and engineering judgment of the authors. After that, the model was trained by using 40 examples of work zone capacity. To evaluate the accuracy of the model, nine samples were taken from the North Carolina studies, 12 samples from Indiana studies, and six samples from Maryland studies. However, there was no information for the value of some parameters that affect work zone capacity. Hence, Karim and Adeli assumed a value for the missing information. Comparison of the estimated values with the observed values indicated that the model had a large error in some cases. In all cases with large error, percentage of trucks was high (19-32%). Two possible sources were mentioned for these errors. Firstly, the model can only simulate the effect of trucks on grades but authors mentioned that trucks had another effect on the capacity of work zones, too. This effect is mean speed reduction due to the presence of trucks which can happen even in level terrain. There was not any information in the training data for this effect. Secondly, there was not any information for grades higher than 25%. A root mean square of 165 (veh/hr) was reported for the model.

Adeli and Jiang (2003) proposed a neuro-fuzzy logic model to estimate work zone capacity. It was assumed that the following parameters affect work zone capacity: percentage of trucks, grade, number of lanes, number of closed lanes, lane width, work zone configuration, work intensity, length of work zone, work zone speed, interchange effects, work zone location (urban or rural), work zone duration (short term or long term), work time (day or night), work day (weekday or weekend), weather conditions, pavement conditions (dry, wet, or icy), and driver population.

They used 168 datasets for testing, checking, and training the model. These datasets were collected from North Carolina, Texas, California, Indiana, Maryland, Ohio, and from Toronto, Canada. There was no information available for some of the variables. Some values were assumed for these variables. To evaluate the proposed model, estimated values versus observed values were plotted on one chart. The chart did not show any outliers. An outlier was defined as "any point with an error value 50% larger than the mean error for all the data points." A lower root mean square was reported for the neuro-fuzzy model compared to the models by Krammes et al. and Kim et al (2001).

## 3.4 ANALYSIS OF THE MODELS

In this section, based on the literature review in the previous sections, the known and unknown issues are discussed.

Effects of the following factors on work zone capacity have been partially studied in a limited number of sites:

- Type of work zone (short term or long term)
- Number of open lanes and number of closed lanes
- Position of closed lanes
- Lighting conditions
- Weather conditions
- Proximity to a ramp
- Driver population
- Work intensity
- Lane width
- Lateral Clearance
- Police presence
- Lateral distance of work activity area to the open travel lane
- Speed limit

In addition to these, there are some other important factors such as flagger presence or ITS presence, both of which have not been comprehensively investigated yet.

Several models have been proposed to estimate work zone capacity. However, none of those models reflect the effects of all above-mentioned parameters on capacity.

One of the applications of capacity is in delay estimation for which the speed at capacity is also needed. Therefore, for accurate estimation of delay, speed-flow relationship is required. Sarasua et al. (2004) and Benekohal et al. (2004) proposed different speed-flow relationships for work zones. The model developed by Sarasua et al. is a Greenshield-type model which is known to be too simple to describe the speed-flow relationship. Moreover, the authors did not suggest any method to adjust the speed-flow relationship regarding non-ideal conditions. Although Benekohal et al. (2004) developed a 3-regime model for work zones and that model can be adjusted for non-ideal conditions, the free flow and transition regimes of that model were developed using the HCM 2000 model for basic freeway sections.

Finally, Passenger Car Equivalence (PCE) factor plays an important role in determining the capacity. Effects trucks in work zones traffic flow are different than the basic freeway sections. Although Al-Kaisy et al. (2003) suggested PCE factors for some special cases, PCE values were not estimated comprehensively for all possible cases. Thus, there is still a need to develop new PCE values for work zones.

## 3.5 CAPACITY VALUES

In this section, capacity values available in the literature are reported.

Table 3-1. North Carolina Work Zone Capacities

| Number of Lanes | | Rural or Urban | End of Transition Capacity (vphpl) | Activity Area Capacity (vphpl) | Intensity of Work Activity |
|---|---|---|---|---|---|
| Normal | Open | | | | |
| 2 | 1 | Rural | 1300 | 1210 | Heavy |
| 2 | 1 | Urban | 1690 | 1560 | Moderate |
| | | | | 1490 | Heavy |
| 3 | 1 | Urban | 1640 | 1440 | Moderate |

Source: Capacity for North Carolina Freeway Work Zones, Karen K. Dixon, Joseph E. Hummer, and Ann R. Lorscheider, Transportation Research Record, 1996

| 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 |

Source: *Traffic Capacity, Speed, and Queue-Discharge Rate of Indiana's Four-Lane Freeway Work Zones*, Yi Jiang Transportation Research Record, 1999

Table 3-3. Work Zone Capacity in Iowa

| Date | Traffic Conditions | Unconverted Free-Flow Volumes: Highest Volume (veh/hr) | Unconverted Free-Flow Volumes: Mean of 10 Highest Volumes (veh/hr) | Converted Free-Flow Volumes: Highest Volume (pcph) | Converted Free-Flow Volumes: Mean of 10 Highest Volumes (pcph) |
|---|---|---|---|---|---|
| 6/19/98 | Free Flow | 1284 | 1216 | 1542 | 1374 |
| 7/2/98 | Free Flow | 1392 | 1302 | 1542 | 1442 |
| 7/10/98 | Free Flow | 1524 | 1438 | 1680 | 1630 |
| 8/7/98 | Free Flow | 1572 | 1375 | 1752 | 1493 |

Source: Capacity of Freeway Work Zone Lane Closures, Maze, T H; Schrock, S D; Kamyab, A, Mid-Continent Transportation Symposium, 2000

Table 3-4. Work Zone Capacity on Gardiner Expressway in Ontario Expressway on April 12 (weekend) and July 11(weekday), 1997,

| | Eastbound (Median lane closed) ( upgrade) Flow (vphpl) | Westbound (Shoulder lane closed) (downgrade) Flow (vphpl) | Weather Condition from "Globe and Mail" |
|---|---|---|---|
| 12 April 1997 | 1631 | 1644 | Wet snow, freezing rain, shower, High near 3°C |
| 11 July 1997 | 1952 | 2102 | Mainly sunny, High near 27°C |

Source: *New Insights into Freeway Capacity at Work Zones Empirical Case Study,* Ahmed Al-Kaisy, Miao Zhou, and Fred Hall, Transportation Research Record, 2000

Table 3-5. Capacity Values during Night and Day Time in Ontario

| Work zone | Length of work zone(m) | Mean capacity value (pcphpl) | |
|---|---|---|---|
| | | Day time | Night time |
| Work zone #1 | 800 | 2247 | 2079 |
| Work zone #2 | 500 | 1853 | 1793 |

Source: Effect of darkness on The Capacity of Long-term Freeway Reconstruction Zones, Al-Kaisy, A F; Hall, FL, Transportation Research Circular, 2000

Table 3-6. Effects of Weather Conditions and Driver Population on Work Zone Capacity in Ontario

| Data Set | Date | Time | Capacity (pcphpl) | Weather | Work Activity |
|---|---|---|---|---|---|
| 1 | 13 April-99 | Weekday-AM | 1967 | Dry | Yes |
| 2 | 14 April-99 | Weekday-AM | 2013 | Dry | Yes |
| 3 | 15 April-99 | Weekday-AM | 2038 | Dry | No |
| 4 | 16 April-99 | Weekday-AM | 1930 | Dry | No |
| 5 | 20 April-99 | Weekday-AM | 2021 | Dry | No |
| 6 | 21 April-99 | Weekday-AM | 1998 | Rainy | No |
| 7 | 22 April-99 | Weekday-AM | 1850 | Dry | No |
| 8 | 23 April-99 | Weekday-AM | 2052 | Dry | No |
| 9 | 26 April-99 | Weekday-AM | 1657 | Dry | Yes |
| 10 | 27 April-99 | Weekday-AM | 1900 | Dry | No |
| 11 | 28 April-99 | Weekday-AM | 1879 | Dry | No |
| 12 | 29 April-99 | Weekday-AM | 1991 | Dry | No |
| 13 | 30 April-99 | Weekday-AM | 1952 | Dry | No |
| 14 | 3 May-99 | Weekday-AM | 1933 | Dry | No |
| 15 | 4 May-99 | Weekday-AM | 1952 | Dry | No |
| 16 | 16 April-99 | Weekday-PM | 1829 | Dry | No |
| 17 | 20 April-99 | Weekday-PM | 1952 | Dry | No |
| 18 | 21 April-99 | Weekday-PM | 1936 | Dry | No |
| 19 | 22 April-99 | Weekday-PM | 1774 | Rainy | No |
| 20 | 23 April-99 | Weekday-PM | 1856 | Dry | No |
| 21 | 26 April-99 | Weekday-PM | 1633 | Dry | Yes |
| 22 | 27 April-99 | Weekday-PM | 1773 | Dry | No |
| 23 | 28 April-99 | Weekday-PM | 1824 | Dry | No |
| 24 | 29 April-99 | Weekday-PM | 1809 | Dry | No |
| 25 | 30 April-99 | Weekday-PM | 1811 | Dry | No |
| 26 | 4 May-99 | Weekday-PM | 1905 | Dry | No |
| 27 | 17 April-99 | Weekend | 1745 | Dry | No |
| 28 | 18 April-99 | Weekend | 1685 | Dry | No |
| 29 | 24 April-99 | Weekend | 1634 | Dry | Yes |
| 30 | 25 April-99 | Weekend | 1598 | Dry | Yes |
| 31 | 2 May-99 | Weekend | 1659 | Dry | No |

Source: Examination of Effect of Driver Population at Freeway Reconstruction Zones, Ahmed Al-Kaisy and Fred Hall Transportation Research Record, 2001 d Studies

| Site | # of closed lanes | Loc. of closed lanes | # of opened lanes | Heavy vehicle (%) | Driver pop. | On-ramp at work | Lateral distance (feet) | Work zone length (mile) | Grade (%) | Work intensity | Work duration (short, long) | Weather (sun, rain) | Work time (day, night) | Avg. Speed (mph) | Capacity (vphpl) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | Right | 3 | 8.2 | 0 | Yes | 0.5 | 1.2 | - 2 | Shoulder pavement (Low) | Short | Sun | Day | 22 | 1612 |
| 2 | 1 | Right | 3 | 8.1 | 0 | Yes | 0.5 | 0.45 | - 2 | Shoulder pavement (Low) | Short | Sun | Day | 37 | 1627 |
| 3 | 1 | Right | 3 | 9.0 | 0 | Yes | 0 | 0.15 | + 3 | Bridge repair (Med) | Short | Sun | Day | 31 | 1519 |
| 4 | 1 | Left | 3 | 10.3 | 0 | N/A | 0.5 | 0.15 | - 5 | Median barrier repair (Low) | Short | Sun | Day | 31 | 1790 |
| 5 | 1 | Left | 3 | 8.0 | 0 | N/A | 0.5 | 0.18 | - 5 | Median barrier repair (Low) | Short | Sun | Day | 30 | 1735 |
| 6 | 1 | Left | 3 | 10.1 | 0 | Yes | 1.0 | 1.9 | - 3 | Median barrier repair (Low) | Short | Sun | Day | 37 | 1692 |
| 7 | 2 | Right | 2 | 14.3 | 0 | Yes | 1.0 | 1.8 | 0 | Pavement (Heavy) | Short | Sun | Night | 23 | 1290 |
| 8 | 2 | Right | 2 | 8.5 | 0 | Yes | 0 | 2.2 | 0 | Pavement (Heavy) | Short | Sun | Night | 21 | 1228 |
| 9 | 2 | Left | 2 | 11.0 | 0 | Yes | 0.5 | 1.3 | 0 | Pavement (Med) | Short | Sun | Night | 22 | 1408 |
| 10 | 2 | Left | 2 | 11.3 | 0 | Yes | 0 | 0.9 | 0 | Pavement (Heavy) | Short | Sun | Night | 24 | 1265 |
| 11 | 2 | Left | 2 | 4.6 | 0 | Yes | 0.5 | 2.0 | 0 | Pavement (Med) | Short | Sun | Night | 17 | 1472 |
| 12 | 2 | Left | 2 | 9.9 | 0 | Yes | 0 | 0.9 | 0 | Pavement (Heavy) | Short | Sun | Day | 20 | 1298 |

Note) Driver population: commuter=1, otherwise=0

Source: A New Methodology to Estimate Capacity for Freeway Work Zones, Taehyung Kim, David J. Lovell, Martin Hall, Jawad Paracha, TRB, CD ROM, 2001

Table 3-8. Work Zone Capacity in Ontario for Commuter traffic on Day Time and Clear Weather Conditions

| Site | Type of closure | Mean capacity | Data (h) |
|---|---|---|---|
| Gardiner Expressway—WB | 3 ---- 2 | 2,102 vphpl | 2.3 |
| Gardiner Expressway—EB | 3 ---- 2 | 1,950 vphpl | 2.3 |
| HWY 403—WB | Right Shoulder | 2,252 pcphpl | 10.5 |
| Queen Elizabeth Way at Burlington—WB | Left & Right Shoulders | 1,853 pcphpl | 6.7 |
| Queen Elizabeth Way at Burlington Bay Skyway—Toronto-bound | 4 ---- 2 | 1,989 pcphpl | 33 |
| Queen Elizabeth Way at Burlington Bay Skyway—Niagara-bound | 4 ---- 2 | 1,985 pcphpl | 18 |

Source: Guidelines for Estimating Capacity at Freeway Reconstruction Zones, Ahmed Al-Kaisy and Fred Hall, Journal of Transportation Engineering, 2003

Table 3.9. Characteristics of Data Collection Locations in South Carolina

| Site # | Date | Time | | Location | | | Type of Work | Closure Geometry | Taper Length | Equipment Activity | Length of Work Zone | Weather Conditions |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Start | End | Interstate | Direction | MPM | | | | | | |
| 1 | 09/12/01 | 19:15 | 21:15 | 85 | N | 32 | Median Cable Guardrail | Inside lane of 2 closed | 863 | Light | Short | Warm, Clear |
| 2 | 09/13/01 | 19:45 | 20:45 | 26 | W | 54 | Median Cable Guardrail | Inside lane of 2 closed | 795 | Light | Short | Warm, Clear |
| 3 | 09/16/01 | 19:40 | 21:15 | 85 | S | 8.5 | Median Cable Guardrail | Inside lane of 2 closed | 600 | Light | Short | Warm, Clear |
| 4 | 09/30/01 | 19:05 | 22:30 | 85 | N | 0 | Median Cable Guardrail | Inside lane of 2 closed | 665 | Light | Short | Warm, Clear |
| 5 | 10/01/01 | 9:00 | 18:00 | 77 | N | 80 | Paving (OGFC) | Inside 2 lanes of 4 closed | 675, 1475, 850 | Heavy | Long | Warm, Clear |
| 6 | 10/03/01 | 17:00 | 22:30 | 385 | N | 40 | Paving (surface) | Outside lane of 2 closed | 446 | Heavy | Long | Warm, Clear |
| 7 | 11/05/01 | 20:00 | 22:00 | 26 | W | 208 | Final Striping | Outside 2 lanes of 3 closed | 668, 1544, 684 | Heavy | Short | Cold, Clear |
| 8 | 01/31/02 | 15:30 | 16:00 | 26 | E | 178 | Concrete Pavement Repair | Outside lane of 2 closed | 800 | Heavy | Medium | Cool, Clear |
| 9 | 03/11/02 | 16:00 | 18:10 | 385 | W | 2 | Median Cable Guardrail | Inside lane of 2 closed | 950 | Light | Long | Cool, Clear |
| 10 | 04/03/02 | 8:30 | 10:30 | 26 | E | 104 | Median Cleanup | Inside lane of 3 closed | - | Light | Short | Warm, Clear |
| 11 | 04/08/02 | 8:42 | 11:10 | 26 | E | 107 | Median Cleanup | Inside lane of 4 closed | 575 | Light | Short | Warm, Clear |
| 12 | 06/03/02 | 19:00 | 21:15 | 85 | S | 28 | Paving | Inside lane 1 of 3 closed | 800 | Light | | Clear |
| 13 | 06/04/02 | 19:00 | 20:30 | 85 | S | 28 | Rumble Strips | Inside lane 1 of 3 closed | - | Light | | Clear |
| 14 | 06/06/02 | 19:00 | 19:00 | 85 | S | 28 | | Inside lane 2 of 3 closed | 800 | Light | | Clear |
| 15 | 06/07/02 | | | 85 | S | | data collection cancelled due to weather conditions, after completion of equipment set-up | | | | | Rain |
| 16 | 06/13/02 | 19:00 | 21:00 | 85 | S | 28 | | Inside 1 lane of 3 closed | | Heavy | | Warm, Clear |
| 17 | 06/14/02 | 19:00 | 21:20 | 85 | S | 28 | Concrete Paving | Outside lane of 2 closed | - | Heavy | Long | Warm, Clear |
| 18 | 06/20/02 | 20:00 | 22:00 | 85 | S | 28 | Concrete Paving | Outside lane of 2 closed | 800 | Heavy | Long | Warm, Clear |
| 19 | 07/09/02 | 19:15 | 20:15 | 85 | S | 2 | Bridge Maintenance | Outside lane of 2 closed | | Light | Long | Warm, Clear |
| 20 | 07/21/02 | 19:03 | 21:08 | 85 | N | 179 | Bridge Maintenance | Outside lane of 2 closed | | Light | Long | Warm, Clear |
| 21 | 07/22/02 | 18:56 | 20:30 | 85 | N | 179 | Bridge Deck Maintenance | Outside lane of 2 closed | | Light | Long | Clear |
| 22 | 08/23/02 | 21:00 | 22:00 | 26 | W | | Concrete Paving | Outside 2 lanes of 3 closed | 800 | Light | Long | Clear |
| 23 | 08/14/02 | 19:17 | 21:00 | 95 | N | 165 | Barrier Wall Erection | Inside 1 lane of 2 closed | 800 | Light | Long | Clear |

MPM = milepost marker; OGFC = open-graded friction course.

Source: Evaluation of Interstate Highway Capacity for Short-Term Work Zone Lane Closures, Wayne A. Sarasua, William J. Davis, David B. Clarke, Jayaram Kottapally, and Pawan Mulukutla, Transportation Research Record, 2004

Table 3.10.Capacity of Short- term Work Zones in South Carolina

| Site # | Location | PC% | T% | Vehicles: 1 min Hourly Volume | | Vehicles: 5 min Hourly Volume | | Vehicles: Hourly Volume | | Passenger Car Equivalents: 1 min Hourly Volume | | Passenger Car Equivalents: 5 min Hourly Volume | | Passenger Car Equivalents: Hourly Volume | | Queue? | Max Queue Length |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min | | |
| 1 | I-85 N MPM 32 | 64.33% | 35.67% | 1980 | 60 | 1056 | 648 | - | | 3060 | 60 | 1560 | 1044 | - | | None | - |
| 2 | I-26 W MPM 54 | 71.05% | 28.95% | 1320 | 120 | 648 | 324 | 497 | 445 | 1680 | 180 | 882 | 492 | 702 | 640 | None | - |
| 3 | I-85 S MPM 8.5 | 87.25% | 12.75% | 2160 | 300 | 1572 | 636 | 1221 | 767 | 2700 | 300 | 1824 | 726 | 1414 | 918 | Few | 3200 |
| 4 | I 85 N MPM 0 | 82.63% | 17.37% | 2100 | 120 | 1440 | 324 | 1320 | 995 | 2280 | 120 | 1728 | 534 | 1540 | 1243 | Continuous | >1 mile |
| 5 | I-77 N MPM 80 | 84.56% | 15.44% | 1410 | 450 | 1140 | 636 | 930 | 802 | 1770 | 555 | 1389 | 765 | 1112 | 954 | None | - |
| 6 | I-385 N MPM 40 | 96.83% | 3.17% | 1140 | 120 | 744 | 60 | 553 | 458 | 1500 | 120 | 768 | 60 | 572 | 479 | None | - |
| 7 | I-26 W MPM 208 | 87.62% | 12.38% | 1800 | 360 | 1308 | 576 | 1124 | 735 | 2160 | 360 | 1506 | 666 | 1310 | 871 | None | - |
| 8 | I-26 E MPM 178 | 84.45% | 15.55% | 1680 | 360 | 1128 | 720 | 927 | 871 | 2010 | 450 | 1416 | 864 | 1107 | 1059 | None | - |
| 9 | I-385 N MPM 2 | 84.49% | 15.51% | 1320 | 0 | 696 | 276 | 565 | 509 | 1710 | 0 | 918 | 312 | 689 | 608 | None | - |
| 10 | I-26 E MPM 104 | 88.60% | 11.32% | 2200 | 1140 | 2016 | 1266 | 1041 | 1041 | 2565 | 1245 | 2262 | 1446 | 1170 | 1170 | Continuous | >4500 |
| 11 | I-26 E MPM 107 | 91.06% | 8.94% | 1722 | 678 | 1480 | 1044 | 1308 | 1152 | 1968 | 738 | 1620 | 1152 | 1437 | 1284 | None | - |
| 12 | I-85 S MPM 28 | 68.61% | 31.39% | 1740 | 180 | 1284 | 636 | 1090 | 820 | 2520 | 180 | 1758 | 1056 | 1518 | 1217 | None | - |
| 13 | I-85 S MPM 28 | 72.68% | 27.32% | 2220 | 180 | 1668 | 756 | 1251 | 976 | 3510 | 270 | 2232 | 960 | 1640 | 1428 | Discontinuous | 500 |
| 14 | I-85 S MPM 28 | 73.69% | 26.31% | 2100 | 480 | 1524 | 1008 | 1357 | 1141 | 2790 | 660 | 2202 | 1428 | 1836 | 1574 | Discontinuous | 8000 |
| 16 | I-85 S MPM 28 | 73.42% | 26.58% | 2160 | 540 | 1500 | 936 | 1341 | 1047 | 2790 | 210 | 2100 | 1296 | 1844 | 1441 | Discontinuous | >1 mile |
| 17 | I-85 S MPM 28 | 82.79% | 17.21% | 2280 | 120 | 1680 | 660 | 1504 | 1240 | 2640 | 120 | 2070 | 768 | 1793 | 1504 | Continuous | >1 mile |
| 18 | I 85 S MPM 28 | 69.67% | 30.33% | 1800 | 360 | 1452 | 732 | 1110 | 916 | 2550 | 450 | 1998 | 1056 | 1552 | 1331 | Continuous | 3000 |
| 19 | I -85 S MPM 02 | 66.93% | 33.07% | 1800 | 240 | 1236 | 636 | 672 | 672 | 2070 | 330 | 1674 | 930 | 995 | 995 | None | - |
| 20 | I -85 N MPM 179 | 85.96% | 14.04% | 1980 | 120 | 1032 | 648 | 903 | 799 | 2670 | 210 | 1500 | 978 | 1332 | 1198 | Continuous | >1 mile |
| 21 | I -85 N MPM 179 | 65.57% | 34.43% | 1800 | 300 | 1548 | 384 | 1339 | 867 | 2190 | 360 | 1830 | 558 | 1536 | 1065 | None | - |
| 22 | I -26 W | 90.40% | 9.60% | 2100 | 420 | 1104 | 948 | 920 | 131 | 2550 | 420 | 1338 | 1110 | 1038 | 149 | Discontinuous | |
| 23 | I-95 N MPM 165 | 69.35% | 30.65% | 1380 | 300 | 1032 | 648 | 907 | 815 | 2130 | 390 | 1500 | 924 | 1276 | 1179 | Discontinuous | 5000 |
| Average | | 70.18% | 20.82% | 1831 | 316 | 1286 | 650 | 1042 | 819 | 2355 | 351 | 1640 | 860 | 1306 | 1065 | | |

PC – passenger car; T – truck; MPM – milepost marker.

Source: Evaluation of Interstate Highway Capacity for Short-Term Work Zone Lane Closures, Wayne A. Sarasua, William J. Davis, David B. Clarke, Jayaram Kottapally, and Pawan Mulukutla, Transportation Research Record, 2004

# CHAPTER 4 FIELD DATA COLLECTION AND REDUCTION

In this chapter, the data collection sites are introduced, various parameters affecting traffic stream are discussed, and the methodology for data collection and reduction is presented. Finally the types of data extracted from the field data are explained.

### 4.1 DATA COLLECTION SITES

Six sites were selected for data collection after consulting with the TRP and visiting a number of work zone sites. Figure 4.1 shows the location of the sites. General information such as the geometric characteristics, traffic, and work zone length is given in Table 4.1. Specific information about data collection activities and work zone conditions are shown in Table 4.2.


Figure 4-1. Data collection sites.
(Source:**http://blogs.suntimes.com/cornerkicks/Illinois_map.jpg**)

Three short term and three long term work zones were among data collection sites. The criteria to indicate whether a work zone is long term or short term, is in compliance with the relevant MUTCD definition. The length of all the work zones but I74EB and I57NB was more than a mile. Besides, all the sites had two lanes open under normal conditions and one lane open during construction.

Table 4-1. General Information about Data Collection Sites

| Site | MP | AADT | WZ Type | Length (mile) | Total No. of lanes | No. of open lanes | Position of closed lane |
|---|---|---|---|---|---|---|---|
| I57-NB | 305-306 | 17,700 | Short term | 0.5 | 2 | 1 | Left |
| I39NB | 54-55 | 18,300 | Long term | 2.7 | 2 | 1 | Right |
| I72-EB | 95-96 | 32,800 | Long term | 1.9 | 2 | 1 | Left |
| I80-EB | 65-66 | 22,200 | Short term | 5.5 | 2 | 1 | Right |
| I80-WB | 65-66 | 22,200 | Short term | 5.5 | 2 | 1 | Right |
| I74-EB | 141-142 | 18,000 | Long term | 0.5 | 2 | 1 | Right |

Table 4.2 shows the prevailing conditions for the data collection sites. Traffic condition in all the sites, except I57-NB, was either in queuing condition or close to that. Moderate traffic volume and no work activity close to travelled lane did not allow the traffic state to become congested on I57-NB and therefore, this site was excluded from further analysis. A flagger was present in three sites, namely I39NB, I80EB and I80WB. The speed limit on I39 NB was 45 mph when the flagger was on the site (till 3:00 p.m.). After 3 p.m. the speed limit was switched back to 55 mph because the flagger was not there anymore. For the other sites, the speed limit was constant over time and is reported in Table 4.2. It should be noted that the speed limit shown in Table 4.2 is the posted speed limit at the location of data collection. The weather was clear for most of the sites during the data collection. However, it was intermittent light rainy on I74EB. The only site where the traffic stream was influenced by police presence was I72EB.

Conditions within the sites, speed limit, hourly volume, and percentage of heavy vehicles are given in Table 4.3. As previously mentioned, I57NB was deleted from further analyses because the hourly volume of this site, around morning peak was 488 vehicles.

It is also worth clarifying why hourly volume of the data set with queue is lower than those of the other data sets of I39-NB which do not include queuing conditions. This work zone was over a long bridge and concrete barriers used to separate the work activity area from the travel lane. Most of the time, work activity did not significantly influence the traffic stream. However, for about 20 minutes, a truck was parked between the concrete barriers and the travel lane (occupying right shoulder). During that time, the workers were working very close to the travel lane, so that work activity along with the flagger presence caused a capacity drop as well as about a 25-minute queuing condition. It should be noted that the data set “AM, Flagger, queue” for I39NB is longer than 25 minutes and the volume reported in Table 4.3 does not represent the queue discharge rate under the condition defined for this data set.

For each set, between 44 to 48 minutes video was reduced from traffic data collected in the field. The next sections elaborate on the data collection and reduction procedure.

Table 4-2. Prevailing Conditions on Data Collection Sites

| Site | Date | Day | Time | Work Activity | Weather | Presence of Flagger | Speed Limit (mph) | Queuing condition | Police Presence |
|---|---|---|---|---|---|---|---|---|---|
| I57-NB | Aug-13-2008 | Wednesday | 6:50-13:00 | No | Clear | No | 45 | Moderate Volume and no queue | No |
| I39-NB* | Aug-15-2008 | Friday | 10:19-15:42 | Yes | Clear | Till 15:00 | 45 | Around 25 minutes Queuing | No |
| I72-EB | Aug-18-2008 | Monday | 6:56-12:11 | Yes | Clear | No | 45 | Close to having queue | Yes, Till noon |
| I80-EB | Aug-19-2008 | Tuesday | 7:23-16:54 | Yes | Clear | Yes | 45 | Sometimes Queuing | No |
| | Aug-20-2008 | Wednesday | 7:25-13:49 | No | Clear | No | 45 | Close to having queue | No |
| I80-WB | Aug-20-2008 | Wednesday | 7:38-14:33 | No | Clear | No | 45 | Close to having queue | No |
| | Aug-25-2008 | Monday | 8:49-13:22 | Yes | Clear | Yes | 45 | Sometimes Queuing | No |
| | | | 15:20-17:35 | Yes | Clear | Yes | 45 | Sometimes Queuing | No |
| I74-EB | Aug-21-2008 | Thursday | 7:47-9:59 | Yes | Clear | No | 55 | Close to having queue | No |
| | | | 15:45-18:12 | No | Drizzle rain | No | 55 | Close to having queue | No |

Table 4-3. Data Sets Information

| Site | Data Set | Speed Limit | Hourly Volume (vph) | Percentage of HV |
|---|---|---|---|---|
| I39NB | AM, flagger, queue | 45 | 790 | 28 |
| | PM, flagger, no queue, dynamic speed feedback sign | 45 | 838 | 21 |
| | PM, no flagger, no queue, dynamic speed feedback sign | 55 | 914 | 22 |
| I72EB | Morning peak, police, no queue | 45 | 877 | 12 |
| | Noon peak, no police, no queue | 45 | 854 | 14 |
| I80EB | AM, flagger, and queue | 45 | 627 | 43 |
| | PM, flagger, and queue | 45 | 723 | 31 |
| | No work activity, no queue | 45 | 655 | 45 |
| I80WB | AM, flagger, and queue | 45 | 633 | 39 |
| | PM, flagger, and queue | 45 | 669 | 43 |
| | No work activity, no queue | 45 | 732 | 38 |
| I74EB | AM, no work activity, no queue | 55 | 708 | 22 |
| | PM, no work activity, no queue, drizzle rain | 55 | 813 | 28 |

## 4.2 DATA COLLECTION

Traffic data and parameters that are likely to influence the traffic stream are listed below:

Traffic Data:

1-Headway for all vehicles
2-Speed of all vehicles
3-Vehicle type
4-Whether or not a vehicle is in platoon
5-Queue Length

Influencing Parameters:

1-Presence of police
2-Presence of flagger
3-Weather conditions
4-Lane width
5-Type of work zone (short term or long term)
6-Number of open lanes and closed lanes
7-Position of the closed lane
8-Type of the traffic control devices used on site
9-Location of the traffic control devices
10-Speed limit
11-Length of work zone
12-Proximity of ramp to work zone

Based on previous experiences, the research team decided to videotape traffic stream to obtain the traffic data for each vehicle. Figure 4.2 shows the typical set up for the camera and markers. Some factors such as the visibility of markers and proximity to the bottleneck were checked to find an optimum location for the camera. After site observations, the research group deduced that a bottleneck is likely to happen close to either the location of the flagger or the work area. Therefore, the camera should be placed close to these locations. On the other hand, data collection should not interfere with construction activity and traffic stream. Regarding these constraints, the camera was installed such that the beginning of queue is visible on the screen and the flow rate is close to queue discharge rate.

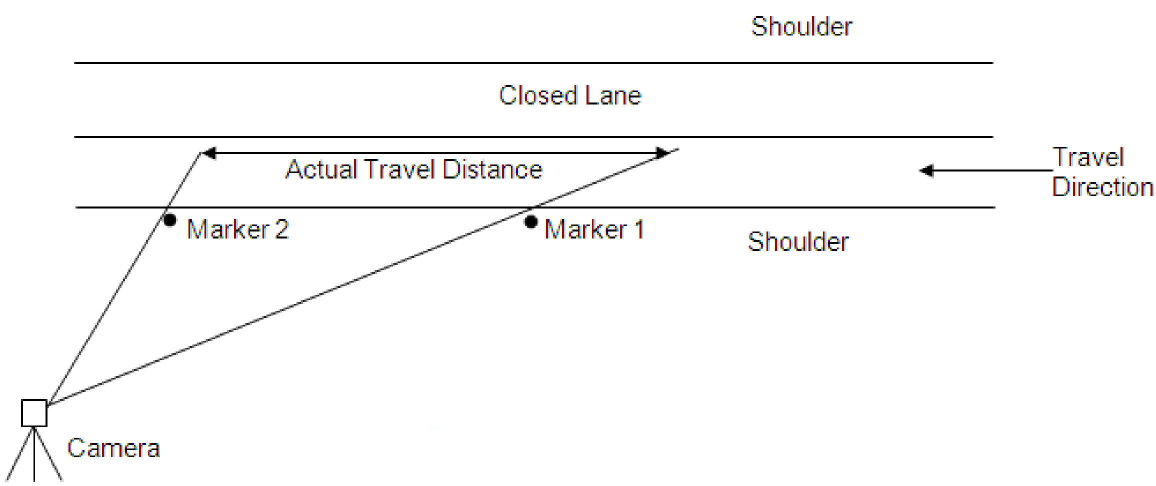


Figure 4-2. Typical set up for markers and camera

Queue length at the end of each minute was recorded on forms previously prepared for this purpose. As previously mentioned, queuing condition was observed on three sites where the beginning of the queue location was visible in the video and almost stable. In the meantime, one person was observing the beginning of the queue to record its location just in case. Another person was writing down the location of the back of the queue at the end of each minute.

To record the conditions in work zone, it was videotaped so information such as location of traffic control devices and geometric characteristics are recorded. The length of the work zone was determined using the odometer of the car used in videotaping.

## 4.3 DATA REDUCTION

In order to have data for individual vehicles, the videos images were viewed and the following information were extracted: Headway, speed, vehicle type, and whether a vehicle is in platoon or not.

### 4.3.1 Headway

In order to determine a vehicle's headway, the time that a particular point of the vehicle (e.g. front bumper) passed marker 1 was recorded. The corresponding time difference between two successive vehicles provided the headway for the vehicle following vehicle. For each vehicle, an alternative headway value can be obtained if one does the same calculation based on marker 2. So two different headway values are available for each vehicle; one being calculated based on marker 1 and the other being computed based on marker2. However, the headway based on the more visible marker was used for analyses because it is more accurate. Depending on the particular location of the camera on a site, marker 1 might be more visible, and vice versa. It should be noted that the accuracy of time is 1/30$^{th}$ of a second.

### 4.3.2 Speed

To find speed, the distance the vehicle traveled was divided by its travel time. The travel time is difference between times when the vehicle passed marker 1 and 2. The accuracy of travel time is 1/30$^{th}$ of a second and yielded a speed accuracy within ±1-mph.

### 4.3.3 Vehicle Type

Vehicles were categorized into three groups, namely passenger cars, large single unit trucks, and tractor semi trailer trucks. In data analysis, the large trucks and semi trucks were combined into a single group of heavy vehicles.

### 4.3.4 In-platoon Vehicles

To determine whether a vehicle is in platoon or not, two criteria were used. A vehicle is considered in platoon if its headway is less than 4 seconds or if its spacing is less than 250 ft. In other words, each vehicle maintaining a headway greater than or equal to 4 seconds and a spacing greater than or equal to 250 ft is considered traveling under free flow condition. Generally, the headway criterion alone is used to determine platooning vehicles. However, this criterion cannot handle the cases in which vehicles are in queue and travel at very low speeds. Under such conditions, because of the very low vehicle speeds, vehicle headways might be longer than 4 seconds while they are still in platoon due to the short spacings. Thus, the spacing criterion should be applied in conjunction with the headway criterion to take care of queuing conditions. It should also be noted that the computation of both the speed and headway yields the spacing for each vehicle.

# CHAPTER 5 CAPACITY ESTIMATION USING FIELD DATA

In this chapter, four different ways of finding capacity are discussed. First the maximum 15-minute flow rate is computed for each data set, and then it is discussed that this approach for estimating capacity generally works when the traffic is close to capacity conditions. Since some of the data sets include traffic volumes that are high but still not at the capacity level, there is a need to develop a new capacity estimation method applicable to traffic volumes less than the capacity level. Thus, in order to achieve reliable capacity estimation for those sites, three alternative approaches are developed and applied to those sites. Then based on the results obtained from the three alternative approaches, a capacity estimation method is proposed. Finally, the capacity values estimated by using the proposed method are reported.

### 5.1 THE MAXIMUM 15-MINUTE FLOW RATE

Based on the capacity definition of the HCM 2000, capacity is “the maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions.” Flow rate could be computed over various time intervals such as 2-minute, 5-minute, and 15-minute intervals.

In order to estimate the capacity based on the HCM 2000 definition, flow rate was calculated for all 15-minute intervals advanced by 1-min increments. For example, the flow rate from the beginning of minute 1 to the end of minute 15 was computed; and then the flow rate from the beginning of minute 2 to the end of minute 16 was calculated. This procedure was repeated to the last minute of the data. Once all the 15-minute flow rates were computed, their maximum value was identified and shown in Table 5.1. A passenger car equivalency (PCE) factor of 1.5 was used to estimate the capacity in passenger cars/ hour/ lane (pcphpl). This value is suggested by HCM 2000 for basic freeway sections on level terrain.

For some cases, the maximum 15-minute flow rate did not lead to a reasonable capacity estimation. For example, for the sites with a speed limit of 45 mph and no work activity, i.e.  the work zones on I80EB, I80WB and I72EB, the maximum 15-minute flow rate varies from 889 to 993 pcphpl while the capacity model of the HCM 2000 returns a capacity of 1600 pcphpl for a work zone with no work intensity and no adverse ramp effect. Hence, the 15-minute maximum flow rate may not represent the capacity condition in these sites. The 15-min max flow rate method for capacity estimation returns reasonable results only when the traffic demand is high enough and close to capacity conditions. If the traffic volume is relatively high but still not close to capacity conditions, some vehicles are likely to maintain large headways (several seconds or more). The large headways results in lower traffic volume and an operation that represents less than capacity conditions. Under such circumstances, the capacity can indeed be higher than the values computed by the maximum 15-minute flow rate method.

### 5.2 “h-n” METHOD

In order to overcome the effects of large gaps in traffic stream on capacity estimation, a new method called “h-n”, i.e. “h minus n”, is examined. The main idea of this method is that road is almost fully utilized by vehicles under capacity conditions. At high traffic volumes that are not close to capacity conditions, some vehicles maintain large gaps. Part of those large gaps is assumed to be the time during which the road is “underutilized.” In order to make a reliable estimation of capacity, the underutilized time is excluded from the computations, and the flow rate is computed by using the time that is efficiency utilized. The underutilized time is computed as below:

$$\text{Underutilized Time} = \begin{cases} 0 & \text{if h} < 8\ second \\ \text{h} - \text{n} & \text{otherwise} \end{cases} \quad (5.1)$$

Where

h = headway in second, and

n=headway threshold for free flow traffic which is 4 seconds, as discussed in the data collection and reduction chapter. The logic behind the method is explained with the aid of an example. Under high and moderate speed conditions, the vehicles whose headways are equal to or greater than 4 seconds are moving under free flow condition. Suppose that a vehicle is traveling under free flow condition with a headway of 10 seconds. Since a 4-second headway is enough to move under free flow condition, this vehicle does not need the extra 6-second headway which is the difference between the 10-second headway and the free flow threshold headway of 4 seconds. During this 6-second, one free flowing vehicle could be processed, but did not happen. Hence, this 6-second extra headway is considered as unutilized time for the roadway.

If the same vehicle maintained a 7-second headway, the extra headway would be 7 seconds minus 4 seconds, which is equal to 3 seconds. Nevertheless, this extra headway of 3-seconds is not enough to serve one more free-flowing vehicle since it is less than the free flow threshold of 4 seconds. Thus, the unutilized time for this vehicle is zero. Therefore, the unutilized portion of the time is computed for vehicles that maintain a headway equal to or greater than 8 seconds.

In order to estimate the capacity, the raw data was aggregated over 15-minute intervals that jump by 1-minute increments. After the total unutilized time was excluded from each 15-minute interval, flow rate was computed based on the utilized time, and then the corresponding 15-min flow rate was computed. Afterwards, the maximum flow rate computed by the “h-n” method is considered as the capacity value. The results are shown in Table 5.1. A PCE value of 1.5 was used to convert trucks to car to estimate the capacity in pcphpl.

The capacity values resulted from the “h-n” method are in general higher than the maximum 15-minute flow rates computed by method 1; however, some of the estimated capacity values still turned out to be too low. For example, the “h-n” method returned a capacity of 1315 pcphpl for the work zone on I74EB that had a speed limit of 55 mph, a short distance lane closure and no work activity. The returned capacity of 1315 pcphpl is not reasonable for such a site based on our field observation. Therefore, although the flow rate which is computed by excluding the unutilized time is less affected by the presence of large gaps in the traffic stream, the “h-n” method can still underestimate the capacity when there is not enough sustained demand. So the following method is used to see if it addresses the lack of demand issue.

Table 5-1. Capacity Values Obtained from Different Methods

| Traffic Condition | Site | Data Collection Time | Capacity Estimation Method | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 15-min max sustained flow | | "h-n" method | | All in-platoon Headways | | All in-platoon Headways and platoon size>4 | |
| | | | VPHPL | PCPHPL | VPHPL | PCPHPL | VPHPL | PCPHPL | VPHPL | PCPHPL |
| Flagger, queue, SL=45 | I39NB | AM | 958 | 1079 | 981 | 1105 | 937 | 1064 | 937 | 1064 |
| | I80EB | AM | 720 | 879 | 1023 | 1245 | 1215 | 1444 | 1166 | 1413 |
| | I80EB | PM | 772 | 882 | 1118 | 1291 | 1360 | 1541 | 1376 | 1551 |
| | I80WB | AM | 674 | 824 | 1035 | 1220 | 1105 | 1271 | 1091 | 1262 |
| | I80WB | PM | 734 | 874 | 868 | 1069 | 946 | 1115 | 957 | 1120 |
| Low work activity, dynamic speed feedback sign, flagger, no queue, SL=45 | I39NB | PM | 974 | 1067 | 1297 | 1393 | 1556 | 1650 | 1558 | 1661 |
| No work activity, no queue, SL=45 | I80EB | AM | 726 | 889 | 1181 | 1453 | 1608 | 1927 | 1563 | 1893 |
| | I80WB | AM | 830 | 993 | 1350 | 1604 | 1611 | 1871 | 1609 | 1878 |
| | I72EB | Noon peak | 880 | 944 | 1316 | 1401 | 1863 | 1969 | 1897 | 2005 |
| No work activity, with police, no queue, SL=45 | I72EB | Morning peak | 974 | 1028 | 1331 | 1426 | 1764 | 1846 | 1738 | 1827 |
| No work activity, dynamic speed feedback sign, no queue, SL=55 | I39NB | PM | 992 | 1103 | 1264 | 1397 | 1635 | 1743 | 1660 | 1772 |
| No work activity, no queue, short distance work zone, SL=55 | I74EB | AM | 804 | 895 | 1191 | 1314 | 1746 | 1855 | 1748 | 1870 |
| | I74EB | PM* | 848 | 956 | 1155 | 1315 | 1631 | 1829 | 1641 | 1853 |

*Drizzle rain

## 5.3 CAPACITY BASED ON ALL IN-PLATOON VEHICLES

In order to eliminate the effect of large gaps in the traffic stream, one can exclude all free-flowing vehicles from the data set and compute the capacity based on the average headway of all the remaining vehicles called in-platoon vehicles. It should be noted that the leader of a platoon is in free flow condition and is not considered as an in-platoon vehicle. At least one of the following criteria should be satisfied in order to consider a vehicle as an in-platoon vehicle for capacity calculations:

1- The headway is less than 4 seconds.
2- The spacing is less than 250 feet.

Then capacity is then estimated by using Equation 5.2:

$$\text{Capacity (vphpl)} = \frac{3600}{\bar{\text{h}}_\text{p}} \qquad (5.2)$$

Where,

$\bar{\text{h}}_\text{p}$ = Average headway of all in-platoon vehicles in second.

The results are tabulated in Table 5.1. A PCE value of 1.5 was used to convert trucks to passenger cars to estimate the capacity in pcphpl. The capacity values determined based on all in-platoon headways are obviously higher than the two aforementioned methods. This method considers all vehicles in platoon including those vehicles in small platoons. Based on field observation and data, the authors believe that the vehicles in moderate and large platoons should be considered in capacity calculations, but not the vehicles in small platoons (Less than 5). This is because in some of the small platoons, successive vehicles were maintaining very short headways, and such a traffic flow condition cannot be sustained when a roadway is operating near or at capacity conditions. Moreover, the vehicles traveling in moderate and large platoons are closer to capacity conditions than those traveling in small platoons.

## 5.4 CAPACITY BASED ON VEHICLES IN MODERATE AND LARGE PLATOONS

As mentioned in the previous section, the vehicles traveling in small platoons should be excluded from the capacity calculations to eliminate the effects of vehicles maintaining very short headways. For this method, the capacity is computed using equation 5.3:

$$\text{Capacity (vphpl)} = \frac{3600}{\bar{\text{h}}_{\text{p}>4}} \qquad (5.3)$$

Where,

$\bar{\text{h}}_{\text{p}>4}$ = Average headway (sec) of vehicles in platoons that had more than four vehicles. The results of this method are very close to the capacity calculated based on all vehicles in platoon. The results are shown in Table 5.1. A PCE value of 1.5 was used to convert the results to pcphpl. In addition, they are higher than the results obtained from the “h-n” method and the maximum 15-minute flow rates.

By applying this method, the effects of very large gaps are excluded from the capacity calculation. Nevertheless, the underlying assumption of the method that a road is fully utilized by vehicles under capacity conditions may not hold true in reality since considerable gaps, though not very large, can still be observed between some vehicles under capacity conditions. In this text, this phenomenon is referred to as the platooning effect on capacity. Due to the aforementioned assumption, this capacity estimation method tends to overestimate the capacity. Thus, capacities obtained from this method can be named “potential capacity”. Potential capacity means that if all vehicles were moving in a large and single platoon, in other words if the road was fully utilized by vehicles; the flow rate would be practically equal to the potential capacity. In the next section, a method is proposed to take care of the platooning effect and make a reasonable estimation for capacity.

## 5.5 PROPOSED CAPACITY ESTIMATION METHOD

So far, 4 methods of capacity estimation have been examined. The maximum 15-minute flow rate and “h-n” method tend to underestimate the capacity and therefore, the results obtained from those methods can be regarded as lower limits of observed capacity. On the other hand, the results based on the average headway of either all platooning vehicles or those in medium and large platoons tend to overestimate the capacity. Thus, they can be considered as the upper limit of observed capacity. When the traffic volume is high and the road is operating at capacity conditions, these upper and lower limits converge to the observed capacity values. The goal of this section is to propose a capacity estimation method which returns a reasonable capacity value when it is applied to moderate and high traffic volume.

As mentioned before, the potential capacity should be adjusted due to so-called platooning effect to get an estimate of observed capacity. For that purpose, it is proposed that a platooning factor be used in order to reflect the proportion of time during which the road is utilized by vehicles at capacity conditions.

Through Equation 5.4, one can apply the platooning factor to potential capacity to get a reasonable estimate of capacity:

$$C_E = C_P * f_P \tag{5.4}$$

Where
$C_E$ = Estimated operating capacity in pcphpl
$C_P$= Potential capacity on pcphpl
$f_P$= Platooning factor

Since the potential capacity tends to overestimate capacity, the platooning factor is expected to be less than or equal to 1.0. It was previously explained how to estimate potential capacity but it is not known yet how to estimate the platooning factor using field data.

### 5.5.1 Platooning Factor Estimation

Based on Equation 5.4, the platooning factor is computed as below:

$$f_P = \frac{C_E}{C_P} \tag{5.5}$$

Where
$C_E$ = Estimated operating capacity in pcphpl
$C_P$= Potential capacity in pcphpl
$f_P$= Platooning factor

In order to determine $f_p$ by using Equation 5.5, both $C_E$ and $C_P$ should be given. In the above equation, although $C_p$ can be computed directly from the field data, $C_E$ is still unknown. One can use an estimate of $C_E$ instead of $C_E$ itself, and obtain an estimate of $f_p$ . If the capacity value estimated by the “h-n” method is used rather than $C_E$ then the estimated platooning factor would be:

$$\tilde{f}_P = \frac{C_{h-n}}{C_P} \tag{5.6}$$

Where

$C_{h-n}$ = Estimated capacity in pcphpl by the "h-n" method

$C_P$= Potential capacity in pcphpl

$\tilde{f}_P$ = Estimated platooning factor

As mentioned earlier, capacity values estimated by the "h-n" method is equal to or less than the operating capacity, $C_E$. Hence $\tilde{f}_P$ is equal to or less than actual platooning factor,$f_P$, and can be treated as a lower bound for $f_p$. Depending on the traffic conditions this lower bound might be close to actual platooning factor or considerably less than that.

Estimated platooning factor can be directly applied to the data when it is close to the actual platooning factor. It is expected that whenever traffic is operating under queuing conditions, $C_{E,}$ $C_P$, and $C_{h-n}$ are close to each other, and both estimated and actual platooning factor are close to 1. Such a condition happened for data sets with an average speed of less than 30 mph and queuing condition

For non-queuing conditions, $C_{h-n}$ is less than $C_E$ and as a consequence, $\tilde{f}_P$ is noticeably less than $f_P$. For these cases, adjustments based on the observation of the field data and the platooning trends were applied to estimate a closer value to actual platooning factor than what was estimated by using equation 5.6. Regarding the platooning trend on the non-queuing sites, values obtained by equation 5.6 were divided by 0.95, and the resulting value was used in equation 5.4.

After adjusting the platooning factor obtained by equation 5.6, a platooning factor of 85% for short term work zones, 90% for long term and long-distance work zones, and 95% for long term and short distance work zones are suggested to reliably estimate the capacity. These suggested platooning factors were applied to potential capacity values obtained from the field data and the results were rounded to the nearest multiple of 50 and shown in Table 5.2.

## 5.6 SUGGESTED CAPACITY VALUES FOR EACH SITE

The suggested capacity and suggested speed at capacity for each traffic condition are reported in Table 5.2. All the sites have "ideal" geometric conditions. For example, the suggested capacity for a typical work zone with ideal geometric characteristics, flagger, queue and 45-mph speed limit is 1200 pcphpl, and the suggested speed at this capacity is 27 mph. These values come from five data sets as shown in Table 5.2. In addition to this information, the average speed and the percentage of heavy vehicles for each data set can be read from Table 5.2.

For the sake of simplicity, a site with ideal geometric conditions and no work activity is considered as the site with base conditions. Based on the tabulated information, the suggested capacity for the base condition at a site with 45-mph speed limit is 1550 pcphpl. For a site with the base condition plus police patrol presence, the suggested capacity is 1450 pcphpl. If flagger is added to the base conditions and there is no queue, the suggested capacity is 1400 pcphpl. For a site with flagger and queue in addition to the base conditions, a capacity value of 1200 pcphpl is suggested. The above values are shown in Table 5.2 for sites with 45 mph speed limit.

Capacities for sites with the base condition and speed limit of 55 mph are generally higher than the capacities for sites with 45 mph. A capacity value of 1600 pcphpl is suggested for a long-distance work zone with 55 mph speed limit and dynamic speed feedback sign, no work activity and no queue. The suggested capacity is based on the data collected from I39NB at which three dynamic speed feedback signs were deployed and work activity was taking place over a bridge. The location of data collection was next to the second dynamic speed feedback sign that was at the beginning of the bridge. On the other hand, a short-distance work zone with 55-mph speed limit and base conditions can process a maximum flow rate of 1750 pcphpl. It should be noted that a short-distance work zone is a work zone where drivers are able to see the end of the work zone when they enter the transition area and consequently, it requires less-than-a-minute travel time at the posted speed limit to exit the work zone. The authors believe that the speed reduction due to the change in geometric characteristics in a short distance work zone is less than the corresponding speed reduction in a long distance work zone when other conditions remain the same. As a result, somewhat higher capacity is expected in short-distance work zones compared to long-distance work zones. Hence, the capacity of a long-distance work zone with speed limit of 55 and base conditions should be lower than 1750 pcphpl but it is still expected to be higher than 1600 pcphpl, i.e. the suggested capacity for the same site with dynamic speed feedback sign. Thus, a capacity value of 1700 pcphpl is suggested for a long (distance) work zone with based conditions and 55 mph speed limit.

The suggested speeds at capacity are based on the average speeds for all vehicles, and all in-platoon vehicles in large and medium platoons (platoon size>4). The suggested speed ranges from 35 to 44 mph for the data sets with speed limit of 45 mph and no queuing condition whereas it is between 46 and 52 for the data sets with speed limit of 55 mph and no queuing condition. Besides, a speed of 27 mph is suggested for the data sets with speed limit of 45 mph and queuing condition.

Table 5-2. Estimated and Suggested Capacity Values

| Traffic Condition | Site | Data collection time | All Vehicles | | In-platoon vehicles where platoon size>4 | | Potential Capacity (pcphpl)* | Capacity (pcphpl)** | Suggested capacity (pcphpl) | Suggested speed (mph) at capacity |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Speed (mph) | %HV | Speed (mph) | %HV | | | | |
| Flagger, queue, SL=45 | I39NB | AM | 16.02 | 28 | 15.9 | 27 | 1064 | 1163 | 1200 | 27 |
| | I80EB | AM | 33.8 | 43 | 30.6 | 42 | 1413 | 1282 | | |
| | I80EB | PM | 36.9 | 31 | 36 | 25 | 1551 | 1367 | | |
| | I80WB | AM | 28 | 39 | 27 | 31 | 1262 | 1275 | | |
| | I80WB | PM | 25.2 | 43 | 25.1 | 34 | 1120 | 1131 | | |
| Low work activity, flagger, dynamic speed feedback sign no queue, SL=45 | I39NB | PM | 37.2 | 19 | 35.3 | 13 | 1661 | 1476 | 1400 | 35 |
| No work activity ,no queue, SL=45 | I80EB | AM | 47.5 | 45 | 46.5 | 42 | 1893 | 1503 | 1550 | 44 |
| | I80WB | AM | 44 | 38 | 42.2 | 33 | 1878 | 1696 | | |
| | I72EB | Noon peak | 44.7 | 14 | 43.715 | 11 | 2005 | 1502 | | |
| Police, no work activity, , no queue, SL=45 | I72EB | Morning peak | 43.2 | 12 | 41.9321 | 10 | 1827 | 1485 | 1450 | 42 |
| Dynamic speed feedback sign no work activity, , no queue, SL=55 | I39NB | PM | 47.3 | 22 | 45.5 | 13 | 1772 | 1495 | 1600 | 46 |
| Short distance work zone, no work activity, no queue, SL=55 | I74EB | AM | 53.7 | 21 | 52.4 | 14 | 1870 | 1760 | 1750 | 52 |
| | I74EB | PM*** | 53 | 28 | 51.7 | 26 | 1853 | 1777 | | |
| No work activity, no queue, SL=55 | Typical | PM | | | | | | | 1700**** | 50**** |

*Computed for vehicles in medium and large platoons
**After applying platooning factor on potential capacity
***Drizzle rain
****Suggested based on the data sets with speed limit of 55 mph

# CHAPTER 6 SPEED-FLOW RELATIONSHIP FOR WORK ZONES

In this chapter, speed-flow models are developed for different conditions in work zones. First, data sets which were used for this purpose are introduced and then all steps to construct the final speed-flow curve are explained. Raw data from these data sets were aggregated. Thereafter, the aggregated data were categorized into three groups each of which represents a particular traffic region. After data cleaning was done, the data were ready to be used for regression analyses. General forms of models used in regression analyses are presented and thereafter, the regression parameters of the models for each data set are reported. Two types of speed-flow curves are proposed: three-regime speed flow curves and four-regime speed-flow curves.

As it will be discussed later, three-regime speed-flow curves have a sharp turning point at the capacity level. The change in slope at this point is rather abrupt. To avoid this issue, four-regime speed flow curves were established based on the field data and findings of from three-regime speed-flow curves. The three regime and four regime curves are identical for the most part, except when flow is near capacity. The four-regime speed-flow curves have smoother change of slope at the point of peak flow compared to the three-regime speed-flow curves.

## 6.1 DATA SETS

Data from 16 sites were available to develop speed-flow relationship. All of them were 2-lane-to-1-lane work zones. Data from five sites were especially collected for this project and are called "new data." Information on these sites is given in the data collection and reduction chapter. Data for the rest of the sites were available from previous studies in Illinois (old data). Three out of 11 sites in the old data had queuing conditions. On the other hand, the traffic volume on the other eight sites was not high and thus, the traffic was under free flow condition. Detailed information about these 11 sites is available in Benekohal et al. (2003). Since the data sets included several sites under free flow condition and with ideal lane width, the sites with less-than-ideal lane width which did not have queuing conditions were eliminated from further analyses. The remaining sites were categorized based on the prevailing conditions on the sites. Four major factors considered in site categorization are:

1-Speed limit
2-Presence of queue
3- Presence of treatment like police presence or flagger
4- Work activity

Conditions and corresponding sites are shown in Tables 6.1 and 6.2 for 45 mph and 55 mph speed limit, respectively. In terms of the above-mentioned factors, some sites were under the same condition, but because of the different speed-flow patterns, their data were not combined. For instance, data sets 4.1 and 4.2 both had 45-mph speed limit, no queue, no work activity and no treatment but they were not combined. Also, data sets 9.1 and 9.2 were kept separate although they had 55-mph speed limit, no treatment, no queue and no work intensity. Further detail on the speed-flow relations will be elaborated later, but for these sites, a linear equation was fitted to the data. As shown in Table 6.3, the slope for data set 4.1 is statistically significant while for data set 4.2, it is not. A similar case can be observed in Table 6.6 for data set 9.1 and 9.2.

Table 6-1. Information on Data Sets with Speed Limit of 45 mph

| SL | Data Set | Condition | Site | | | Number of data points for each region | | |
|---|---|---|---|---|---|---|---|---|
| | | | Name | MP | New data or old data? | Free flow region | Transition region | Congested region |
| 45 | 1 | Flagger, no queue, very low work intensity | I39NB | 54 | New data | 10 | 22 | – |
| | 2 | Flagger, no queue, high work intensity | I55NB | 244 | Old data | 31 | 13 | – |
| | 3 | Police, no queue, very low work intensity | I72EB | 95 | New data | 5 | 17 | – |
| | 4.1 | No treatment, no queue, no work activity | I80 EB | 65 | New data | 41 | – | – |
| | | | I80WB | 65 | | | | |
| | 4.2 | No treatment, no queue, no work activity | I57NB | 250 | Old data | 60 | – | – |
| | 5 | No treatment, no queue, low work intensity | I57NB | 271 | Old data | 101 | 4 | – |
| | 6 | No treatment, with queue, moderate work intensity, idealized | I55SB | 55 | Old data | – | 14 | 53 |
| | 7 | Flagger, with queue, high work intensity | I39NB | 54 | New data | – | – | 99 |
| | | | I80EB (AM data) | 65 | | | | |
| | | | I80EB (PM data) | 65 | | | | |
| | | | I80WB (AM data) | 65 | | | | |
| | | | I80WB (PM data) | 65 | | | | |

Table 6-2. Information of Data Sets with Speed Limit of 55 mph

| SL | Data Set | Condition | Site | | | Number of data points for each condition in | | |
|---|---|---|---|---|---|---|---|---|
| | | | Name | MP | New data or old data? | Free flow region | Transition region | Congested region |
| 55 | 8 | 8-No treatment, no queue, no work activity, short distance work zone | I74EB (AM data) | 141 | New data | 45 | _ | _ |
| | | | I74EB (PM data) | 141 | | | | |
| | 9.1 | 9.1-No treatment , no queue, no work activity | I80WB | 39 | Old data | 58 | _ | _ |
| | 9.2 | 9.2-No treatment , no queue, no work activity | I70EB | 145 | Old data | 56 | _ | _ |
| | 10 | 10-No treatment , no queue, high work intensity | I80EB | 43 | Old data | 44 | 14 | _ |
| | 11 | 11-No treatment , with queue, no work intensity | I55NB | 55 | Old data | _ | 18 | 115 |
| | | | I74EB Idealized | 5 | | | | |

## 6.2 DATA AGGREGATION

After data reduction, space-mean speed and flow were aggregated over 2-minute intervals, and the flow was converted to hourly flow rate. Shorter-than-2-minute intervals were not used to avoid large fluctuations in estimation of flow and speed. On the other hand, larger intervals such as 5-minute and 15-minute intervals were not appropriate for use as some important changes in traffic flow may be masked by averaging data over a long interval.

Different aggregation methods were used for different regions. For the free flow and transition regions, the number of vehicles was counted over each 2-minute interval and was converted to hourly flow rate. Also, the corresponding space-mean speed was computed for each interval. The aforementioned method is the one traditionally used for data aggregation. For congested regions, in-platoon vehicles were detected during each 2-minute interval. The criteria to detect in-platoon vehicles are presented in the data reduction and collection chapter. For in-platoon vehicles, the average headway (sec) and the space-mean speed were computed. Finally, the hourly flow rate (vph) was computed based on the average headway. The reason for using such an aggregation method is that some moderate or large gaps might still exist between the vehicles in congested traffic conditions. The intervals which include those gaps do not represent capacity condition properly. The effect of those gaps in flow calculations is magnified when 2-minute flow is converted to the hourly flow rate.

It should be noted that a PCE value of 1.5, suggested by the HCM 2000 for basic freeway sections was used to estimate the average flow rate in pcphpl.

## 6.3 DATA CLUSTERING

Aggregated data were divided into three groups, each of which is associated with one particular traffic conditions. In the following section, the traffic regions are defined and based on the definition, the data are clustered accordingly.

### 6.3.1 Free Flow Region

Free flow traffic region represents a state in which the speed is high and the volume is low. Under free flow condition, speed is close to a maximum speed allowed by a particular geometric condition and traffic control device such as the posted speed limit. In order to detect low volume condition, a threshold was set for flow, under which the traffic volume is considered to be low. In the first attempt the volume of 800 pcphpl was chosen as the free flow threshold. The capacity and the free flow threshold for a basic freeway section with speed limit of 70 mph are 2400 and 1300 pcphpl respectively. On the other hand, based on the HCM 2000 model the capacity of a work zone with no work activity and no ramp effect is 1600 pcphpl and the relevant free flow threshold is a number close to 800 pcphpl assuming that there is a linear relationship between capacity and free flow threshold. After that different data sets were investigated to see how much this threshold is reasonable. Investigation showed that a value close to 800 pcphpl is a good estimation for sites with work activity since generally after that volume the rate of speed decrease was higher than free flow condition. For sites without work activity, a free flow threshold of 1200 pcphpl was selected because generally the speed-flow relationship for volume lower than 800 pcphpl was the same as the relevant relationship for volume lower than 1200 pcphpl.

### 6.3.2 Congested Region

In the congested region, flow rate is below the capacity level and speed is below the optimal speed. In order to identify the congested data, time series plot of speed and flow for each site were investigated. Intervals with low speed and low departure rate were identified as congested intervals. Field notes were also checked to make sure that the traffic had been under queuing condition for selected intervals.

### 6.3.3 Transition Region

The state between the free flow and congested region is called transition region. In the transition regions, as volume increases, speed decreases and the rate of decrease is non-linear. Field data that did not belong to the free flow or to the congested regions was considered as the transition data.

## 6.4 DATA CLEANING

For each region, few data points which were not following the general trend were detected as the outliers and removed. These points had considerably either higher or lower flow rate than other points with a speed close to speed of the outlier. Moreover, the data sets were examined to find the cases which are less likely to happen under stationary traffic flow conditions, such as a slow moving truck which causes a flow-break down or low speed condition for few minutes in the transition regions. The data which were “contaminated” by the effect of slow moving truck were deleted.

As mentioned in the data aggregation section, in order to exclude the effects of large gaps in the congested regions data, the average headway and the corresponding flow rate were computed using the data for in-platoon vehicles with sufficient sample size. The criteria for sufficiency was set based on the number of platooning vehicles (excluding leader of platoons) and corresponding cumulative headway. For a site without flagger, if the number of platooning vehicles in the 2-minute interval was more than 25 and the cumulative headway for these vehicles was longer than 50 seconds, then that interval was considered as a valid data point for the congested regions. Otherwise, it was deleted and not considered for further analyses. For sites with flagger, if the number of platooning vehicles in the 2-minute interval was at least 10, then the data point was considered valid. The reason for different criteria is that it was observed that when a flagger is present, the number of vehicles in queue may not be large but queues are more frequent.

## 6.5 GENERAL FORM OF THE SPEED-FLOW MODELS FOR THREE-REGIME MODELS

Speed-flow models developed for each traffic region are the bases of the speed-flow curves that will be proposed.

### 6.5.1 Free Flow Regime

For free flow region, a linear type model is used as below:

$$U = a + b * Q \tag{6.1}$$

Where
U: Operating speed of passenger cars (mph),
Q: Flow rate (pcphpl),
a, b : Coefficient obtained from regression analysis.
The parameter a could be considered as the free flow speed.

### 6.5.2 Transition Regime

For the transition region, the following equation was used to find a non-linear relationship.

$$U = a - b * \left( \frac{Q - c}{d} \right)^{e} \tag{6.2}$$

U: Operating speed of passenger cars (mph),
Q: Flow rate (pcphpl),
a, b, c, d, e: Regression constants.

The regression constant "c" is the flow (pcphpl) threshold below that the traffic is assumed to be in free flow condition. As mentioned previously, this threshold was set based on the data analyses. The value of "c" is 800 pcphpl for work zones with work activity and it is 1200pcphpl for work zones without work activity. These thresholds may be fine-tuned for different work zone conditions.

It should be noted that the general form of the transition equation is basically similar to the one suggested by the HCM 2000 for the transition regime of basic freeway sections. However, the constants and threshold are different to reflect work zone data. It should be noted that the value of "e", which is the exponent of Equation 6.2, is equal to 2.6 in the HCM 2000 model for basic freeway sections. For work zones, four different exponents (2.1, 2.6, 3.1, and 3.6) were used. Regression results revealed that equations with the exponent 3.6 had better fit to the work zone data. The higher exponent indicates a higher drop in speed when flow reaches capacity level.

### 6.5.3 Congested Regime

The power function form proposed by Benekohal et al. (2004) was re-examined using the new data and was found to be the suitable form for the congested region. New parameters were obtained based on the new data sets. The general form of the model is:

$$Q = a * U^{b} \tag{6.3}$$

Where
U: Operating speed of passenger cars (mph),
Q: Flow rate (pcphpl),
a, b: Regression constants.

## 6.6 SELECTED MODELS FOR THE THREE-REGIME SPEED-FLOW CURVES

In this section, the speed-flow models are proposed for work zones under different prevailing conditions. Different conditions and the corresponding sites are shown in Table 6.1 and two for 45 mph and 55 mph speed limit, respectively. The number of the data points in each regime that remained after data cleaning is also shown in those tables. No model was fit for data sets which have less than 10 data points. Free flow, transition and congested models for the speed limit of 45 mph are tabulated in Table 6.3, 6.4 and 6.5, respectively. Models for sites with the speed limit of 55 mph are reported in Table 6.6, 6.7, and 6.8. Speed-flow models are proposed for following conditions:

1- A site with speed limit of 45 mph without flagger
2- A site with speed limit of 45 mph and flagger
3- A site with speed limit of 55 mph.

The speed-flow relationships should be for ideal conditions. A work zone with ideal conditions is the one with ideal geometric characteristics and no work activity. When a site with the ideal conditions was not available, the model for the non-ideal sites is carefully used to develop speed-flow relationship for ideal conditions.

Both the selected non-ideal model and the proposed ideal model are plotted on one chart, see Figures 6.1, 6.2, and 6.3. Considering the relative trend of the ideal model versus the non-ideal model, an interpolation/ extrapolation method is proposed to develop speed-flow relationships for different non-ideal conditions.

### 6.6.1 A Site with Speed Limit of 45 mph and with Flagger

The data for work zone shows that even for low and moderate traffic volume speed decreased as volume increases. For this case, see Figure 6.1, two free flow models are available. Data set 1 and 2 are associated with the sites with flagger and work activity, but the former one had very low work activity whereas the latter had very high work activity. In the data set 1, workers were working behind concrete barriers and there was a wide shoulder between the flagger and the concrete barriers, hence the workers were far from the travel lane and practically can be considered that there was no work intensity.

The results of the regression analyses given in Table 6.3 show that the slope is statistically significant for the data sets 4.1 and 5. It indicates that the speed significantly decreased as the volume increased. The slopes of the free flow models for data sets 1 and 2, the data sets associated with flagger, are not statistically significant however the models show that the speed decrease as the volume increased. Based on the information on Table 6.3, the authors believed that there is a decreasing trend between volume and speed and it was decided to keep slope of the free flow models for the data sets 1 and 2. The slopes of these two models are not significantly different than each other. Thus, the authors decided to use a slope of 0.0031 for both models. This finding is different than the trend for speed-flow curve for basic freeway sections proposed by the HCM 2000. In the HCM model, at low to moderate traffic conditions (up to 1300 pcphpl) speed does not decrease as traffic volume increases.

For the transition regime, the model obtained from data set 1 was preferred over the model obtained from data set 2 because the curvature of the model was more reasonable.

There is only one congested model for a site with flagger. That model comes from data set 7.

### 6.6.2 A Site with Speed Limit of 45 mph without Flagger

For the free flow regime, the model of the data set 4.1 was selected. As shown in Table 6.1, both the data sets 4.1 and 4.2 have a speed limit of 45 mph, no treatment and no work intensity. The model obtained from the data set 4.1 has a statistically significant slope at 90 percent confidence level while the slope of the model for the data set 4.2 is not statistically significant. The model with the significant slope is selected since the authors believe that as traffic volume increases, speed significantly decreases in work zones with work activity (as most of the work zones with speed limit of 45 mph had work activity) under free flow conditions. A witness of this claim could be the model obtained from the data set 5 which involves work activity on-site. The slope for this model is statistically significant. Hence, the model of the data set 4.1 is selected. One may argue that the model for data set 5 could be chosen instead of the one for data set 4.1. However, the value of the slope of model 5 is a little higher compared to the slope of the models for the other data sets with speed limit of 45 mph and it seems more reasonable to select the model for data set 4.1.

Data set 4.1 contains data from I80EB (MP=65) and I80WB (MP=65). These sites had one-foot left and right lateral clearance at the location of data collection which was over a short distance bridge. The speed reduction due to one-foot left and one-foot right lateral clearance was estimated to be 4.6 mph (Benekohal et al. 2003). The free flow line based on the actual data from I80EB (MP=65) and I80WB (MP=65) was shifted 4.6 mph upward. So the intercept of the new curve became 55.6 mph. The intercept of the new curve is rounded down and 55 mph is used for the intercept of the free flow model for work zones with speed limit of 45 mph and with ideal geometric conditions, no work activity and no flagger.

The only transition model for a site with speed limit of 45 mph and no flagger is the one for the data set 6. However, the model was not selected because the model had to be extrapolated beyond the range of the available data. Instead, the model of the data set 1 which was already used for a site with speed limit of 45 mph and flagger was used.

For the congested regime, the model for data set 7 was used. This model comes from the data collected in the work zone on I55SB (MP=55) which was the only site with congestion data and without any treatment.

### 6.6.3 A Site with Speed Limit of 55 mph

For the free flow regime, almost none of the models given in Table 6.6 had significant slope. On the other hand, the authors believe that since usually no severe work activity occurs in work zones with speed limit of 55 mph, the operation of such a work zone is similar to that of a basic freeway section. Therefore, a slope of zero is selected for the free flow regime.

For the transition regime, the model for data set 11 shown in Table 6.7 was used. This model was the only model with speed limit of 55 mph which did not have work intensity.

The only model available for the congested regime is the one for data set 11.

Table 6-3. Models for the Free Flow Regime and Speed Limit of 45mph

| SL | Data Set | Condition | Free Flow Regime | | |
|---|---|---|---|---|---|
| | | | **Model** | **$R^2$** | **P-value for slope** |
| 45 | 1 | 1-Flagger, no queue, very low work intensity | Speed=42.81-0.0031*Flow | 0.007 | 0.822 |
| | 2 | 2-Flagger, no queue, high work intensity | Speed=33.38-0.0021*Flow | 0.006 | 0.6706 |
| | 3 | 3-Police, no queue, very low work intensity | – | – | – |
| | 4.1 | 4.1-No treatment, no queue, no work activity | Speed=51.052-0.004*Flow | 0.082 | 0.0686 |
| | 4.2 | 4.2-No treatment, no queue, no work activity | Speed=47.0837-0.0008Flow | 0.0004 | 0.8793 |
| | 5 | 5-No treatment, no queue, low work intensity | Speed=45.38-0.007*Flow | 0.1401 | 0.0001 |
| | 6 | 6-No treatment, with queue, moderate work intensity, idealized | – | – | – |
| | 7 | 7-Flagger, with queue, high work intensity | – | – | – |

Table 6-4. Models for the Data Sets with Transition Regime and Speed Limit of 45 mph

| SL | Data Set | Condition | Transition Regime | | |
|---|---|---|---|---|---|
| | | | Model | $R^2$ | RMS |
| 45 | 1 | Flagger, no queue, very low work intensity | $\text{Speed} = 38.6 - 59.48 * \left(\frac{\text{Flow} - 800}{1163.70}\right)^{3.6}$ | 0.08 | 3.38 |
| | 2 | Flagger, no queue, high work intensity | $\text{Speed} = 26.6 - 16 * \left(\frac{\text{Flow} - 800}{558.34}\right)^{3.6}$ | 0.10 | 3.52 |
| | 3 | Police, no queue, very low work intensity | $\text{Speed} = 43.4 - 66.60 * \left(\frac{\text{Flow} - 800}{1369.90}\right)^{3.6}$ | 0.07 | 1.85 |
| | 4.1 | No treatment, no queue, no work activity | – | – | – |
| | 4.2 | No treatment, no queue, no work activity | – | – | – |
| | 5 | No treatment, no queue, low work intensity | – | – | – |
| | 6 | No treatment, with queue, moderate work intensity, idealized | $\text{Speed} = 42.4 - 38.74 * \left(\frac{\text{Flow} - 800}{1544.76}\right)^{3.6}$ | 0.17 | 3.14 |
| | 7 | Flagger, with queue, high work intensity | – | – | – |

Table 6-5. Models for the data Sets with Congested Regime, and Speed Limit 45 mph

| SL | Data Set | Condition | Congested Regime | | |
|---|---|---|---|---|---|
| | | | **Model** | **$R^2$** | **RMS** |
| 45 | 1 | Flagger, no queue, very low work intensity | – | – | – |
| | 2 | Flagger, no queue, high work intensity | – | – | – |
| | 3 | Police, no queue, very low work intensity | – | – | – |
| | 4.1 | No treatment, no queue, no work activity | – | – | – |
| | 4.2 | No treatment, no queue, no work activity | – | – | – |
| | 5 | No treatment, no queue, low work intensity | – | – | – |
| | 6 | No treatment, with queue, moderate work intensity, idealized | $\text{Flow} = 109.30 \times \text{Speed}^{0.7594}$ | 0.87 | 105.4 |
| | 7 | Flagger, with queue, high work intensity | $\text{Flow} = 211.56 \times \text{Speed}^{0.5472}$ | 0.65 | 138.7 |

Table 6-6. Models for the data Sets with Free Flow Regime, and Speed Limit 55 mph

| SL | Data Set | Condition | Free flow regime | | |
|---|---|---|---|---|---|
| | | | Model | $R^2$ | P-value for slope |
| 55 | 8 | No treatment, no queue, no work activity | Speed=54.187-0.0007*Flow | 0.003 | 0.7156 |
| | 9.1 | No treatment, no queue, no work activity | Speed=54.8190-0.0020Flow | 0.015 | 0.366 |
| | 9.2 | No treatment, no queue, no work activity | Speed=52.196-0.0032Flow | 0.067 | 0.055 |
| | 10 | No treatment, no queue, high work intensity | Speed=49.88-0.003*Flow | 0.011 | 0.49 |
| | 11 | No treatment, with queue, no work intensity | – | – | – |

Table 6-7. Models for the data Sets with Transition Regime, and Speed Limit 55 mph

| SL | Data Set | Condition | Transition Regime | | |
|---|---|---|---|---|---|
| | | | Model | $R^2$ | P-value for slope |
| 55 | 8 | No treatment, no queue, no work activity | | | |
| | 9.1 | No treatment, no queue, no work activity | | | |
| | 9.2 | No treatment, no queue, no work activity | | | |
| | 10 | No treatment, no queue, high work intensity | $\text{Speed} = 47.8 - 69.26 * \left(\frac{\text{Flow} - 800}{965.0}\right)^{3.6}$ | 0.36 | 4.04 |
| | 11 | No treatment, with queue, no work intensity | $\text{Speed} = 47.9 - 37.69 * \left(\frac{\text{Flow} - 800}{1498.02}\right)^{3.6}$ | 0.17 | 2.78 |

Table 6-8. Models for data Sets with Congested Regime, and Speed Limit 55 mph

| SL | Data Set | Condition | Congested Regime | | |
|---|---|---|---|---|---|
| | | | Model | $R^2$ | P-value for slope |
| 55 | 8 | No treatment, no queue, no work activity | – | – | – |
| | 9.1 | No treatment, no queue, no work activity | – | – | – |
| | 9.2 | No treatment, no queue, no work activity | – | – | – |
| | 10 | No treatment, no queue, high work intensity | – | – | – |
| | 11 | No treatment, with queue, no work intensity | Flow = $271.43 \times \text{Speed}^{0.4868}$ | 0.61 | 158.2 |

**6.7 FINE-TUNING**

The speed-flow curves selected for each case so far are not continuous. In order to have a continuous speed-flow relationship, the free flow model of a particular case should be connected to the transition model of the same case. Likewise, the transition model should intersect with the congestion model. There are two major concerns about the connection of the curves. First, in most of the cases, the free flow models and transition models do not cross each other and some vertical gap occurs between those curves. Second, the crossing point of the transition curve with the congestion curve should be comparable with the capacity value suggested based on the field data. In order to solve the issue, all the 3 regimes for a particular case were first plotted on one chart. The free flow and congestion model were set to be fixed whereas the transition model was slightly moved vertically and horizontally to obtain a smooth connection at the intersection points. Moreover, the transition curve was fine-tuned such that the flow rate at the intersection point of the congestion and the transition model is comparable with the capacity value estimated by using the field data. For fine-tuning, first transition curve was extended beyond the range of the data by which the curve has been established. Then the intersection point of the extended transition curve and congestion model was compared with the capacity value estimated by filed data. If there was a close agreement between the intersection point and the capacity value based on the field data then no more fine-tuning was made on the models otherwise the transition curve was shifted horizontally and vertically in a way to achieve a reasonable capacity value compared to the one estimated by the field data and at the same time to have a smooth pattern at the crossing point of the free flow model and the transition model. The final curves were shown in Figure 6.1, 6.2 and 6.3. The corresponding equations are also reported in Table 6.9.

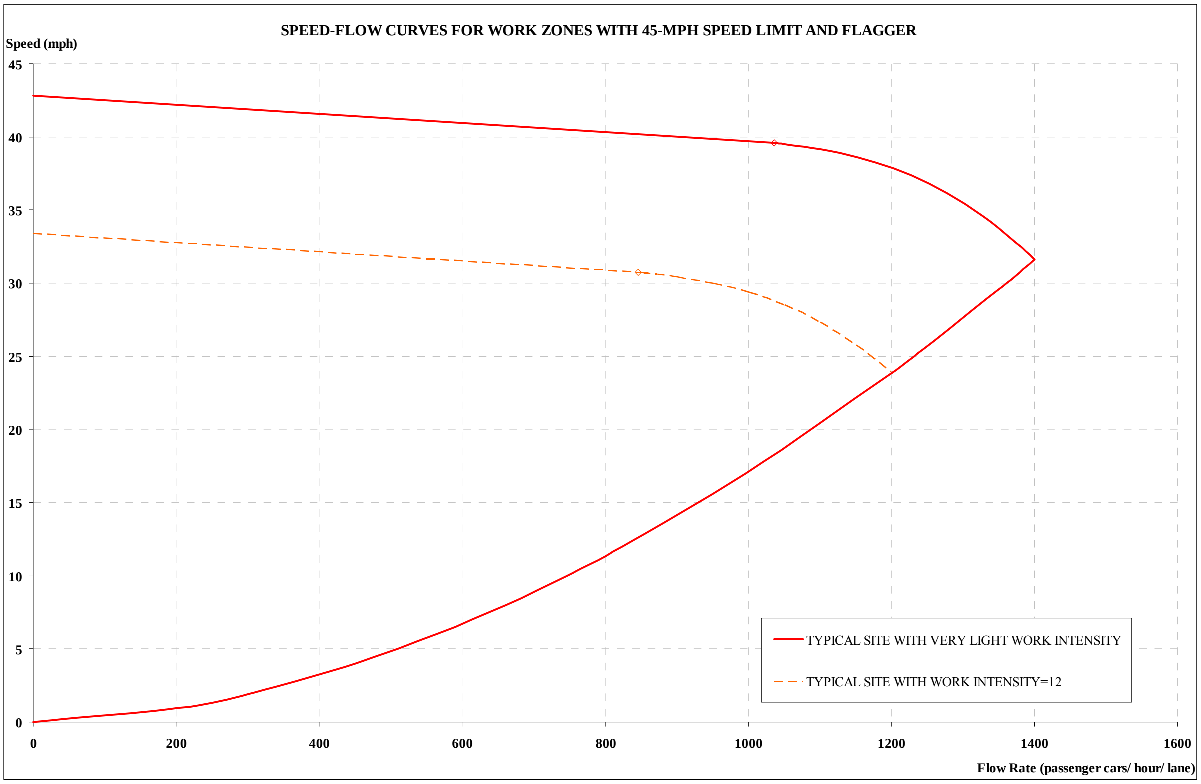


Figure 6-1.Three-regime speed-flow curve for a site with speed limit of 45 mph and flagger.

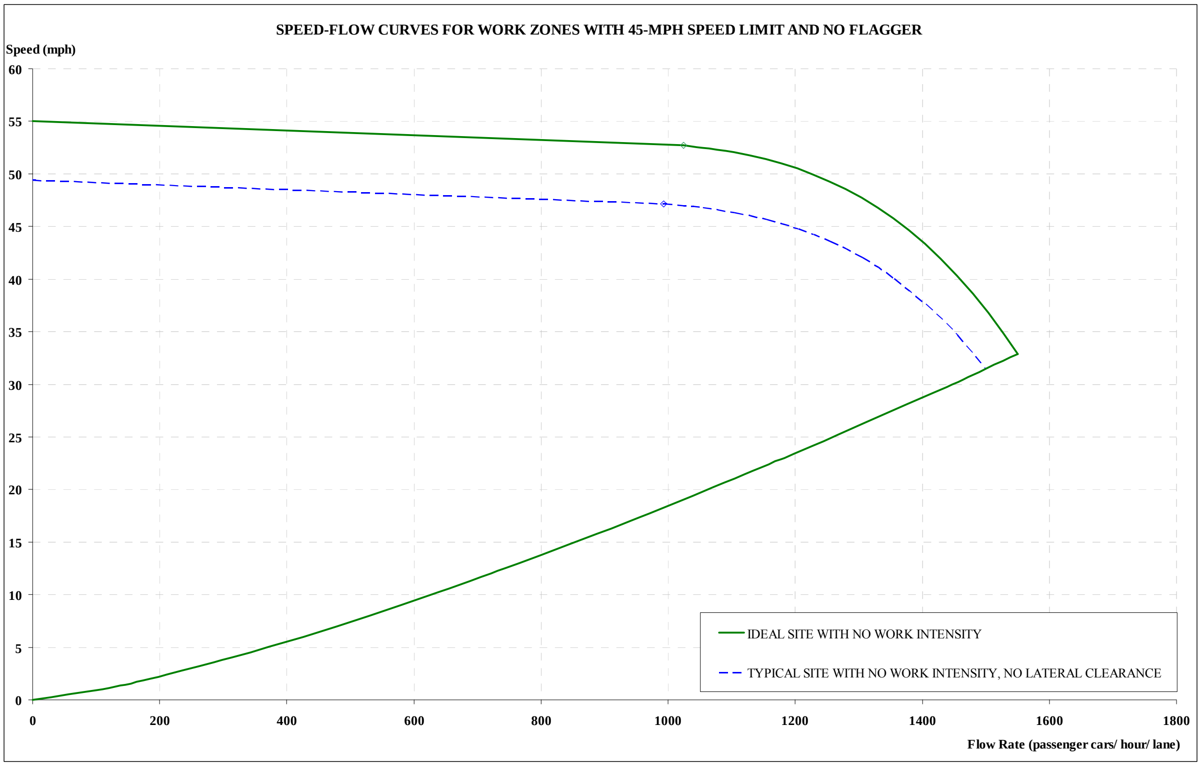

Figure 6-2.Three-regime speed-flow curve for a site with speed limit of 45 mph and no flagger.

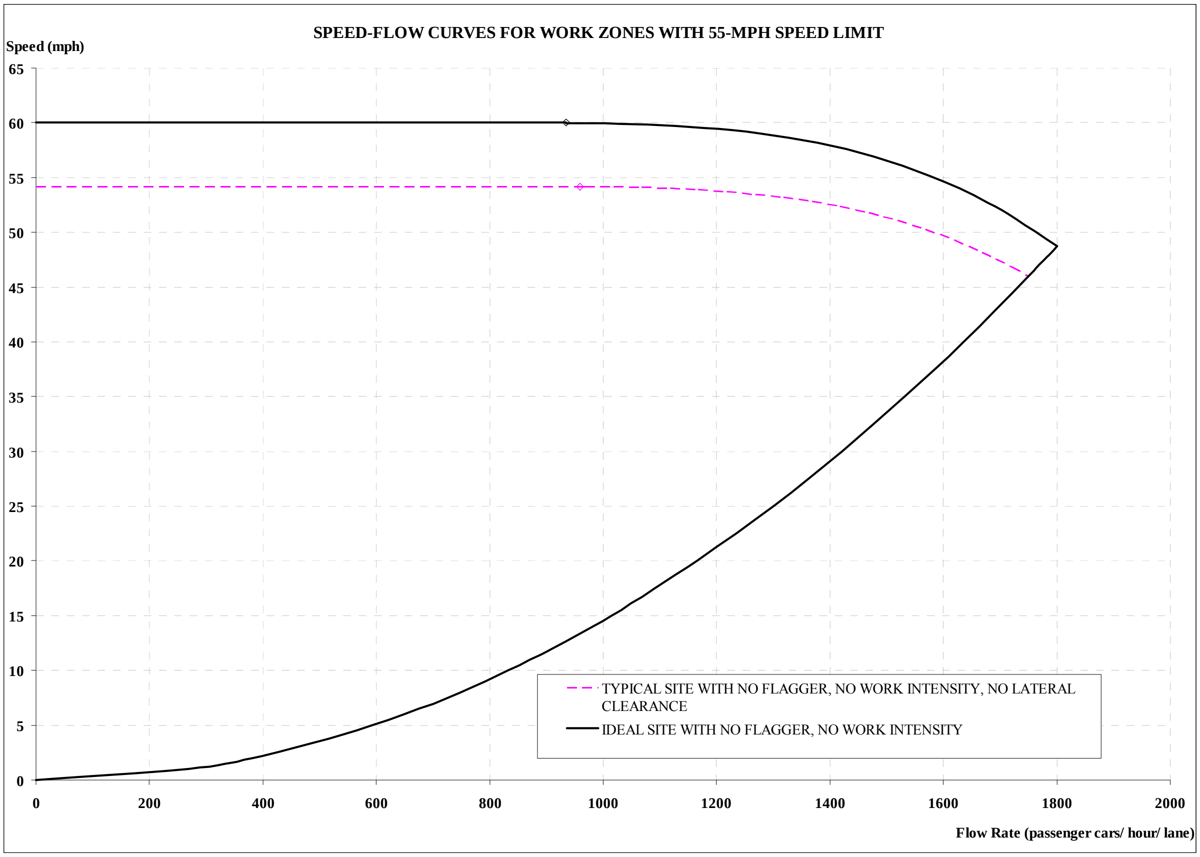


Figure 6-3**.** Three-regime speed –flow curve for a site with speed limit of 55 mph.

Table 6-9. Proposed Speed-Flow Relationships for Three-Regime Models

| CASE | FREE-FLOW EQUATION | TRANSITION EQUATION | CONGESTION EQUATION |
|---|---|---|---|
| Typical site with SL=45 mph, flagger, very low work intensity | Speed*=42.81-0.0031*Flow** | $\text{Speed} = 39.95 - 64.9 * \left(\frac{\text{Flow} - 778}{1100}\right)^{3.6}$ | $\text{Flow} = 211.56 * \text{Speed}^{0.5472}$ |
| Typical site with SL=45 mph, flagger, with high work intensity | Speed = 33.38 - 0.0031 * Flow | $\text{Speed} = 31.04 - 64.9 * \left(\frac{\text{Flow} - 603}{1100}\right)^{3.6}$ | $\text{Flow} = 211.56 * \text{Speed}^{0.5472}$ |
| Ideal site with SL=45 mph, no flagger, no work intensity | Speed = 55 - 0.00226 * Flow | $\text{Speed} = 53.10 - 64.9 * \left(\frac{\text{Flow} - 754}{1100}\right)^{3.6}$ | $\text{Flow} = 109.30 * \text{Speed}^{0.7594}$ |
| Typical site with SL=45 mph, no flagger, no work intensity, no lateral clearance | Speed = 49.37 - 0.00226 * Flow | $\text{Speed} = 47.39 - 64.9 * \left(\frac{\text{Flow} - 755}{1100}\right)^{3.6}$ | $\text{Flow} = 109.30 * \text{Speed}^{0.7594}$ |
| Typical site with SL=55 mph, no flagger, no work intensity, no lateral clearance | Speed=54.187 | $\text{Speed} = 54.21 - 73 * \left(\frac{\text{Flow} - 768}{1800}\right)^{3.6}$ | $\text{Flow} = 271.43 * \text{Speed}^{0.4868}$ |
| Ideal site with SL=55 mph, no flagger, no work intensity | Speed=60 | $\text{Speed} = 60.03 - 73 * \left(\frac{\text{Flow} - 728}{1800}\right)^{3.6}$ | $\text{Flow} = 271.43 * \text{Speed}^{0.4868}$ |

### 6.8 BRIEF DISCUSSION ON THE THREE-REGIME MODELS

A three-regime speed-flow curve was developed for each for work zone traffic condition. Figure 6-1 shows one of the three-regime speed-flow curves after fine-tuning. Figure 6-1 indicates that the change in slope is rather abrupt at the point of maximum flow rate. In order to solve the issue, the authors decided to use a four-regime speed-flow curve that is still based on the models obtained for each traffic regime. The three-regime models and the four-regime models are identical, except near capacity. The four-regime models have smoother change of slope near capacity, but are slightly more complicated than the three-regime models.

### 6.9 INTRODUCTION TO FOUR REGIME MODELS

The four regimes are named as free flow, upper-transition, lower transition, and congested regime. The four regimes are shown in Figure 6-4. The upper transition regime is the part of the speed-flow curve which connects the free-flow regime to the lower transition regime. The lower transition regime connects the upper transition regime to the congested regime. The free flow and the congested regimes in the four-regime models are identical to the corresponding free flow and the congested regimes in the three-regime models. For a given work zone traffic condition, the range spanned by the lower transition and congested regimes in four-regime speed-flow curves is practically the same as the range spanned by the congested regime in three-regime models.

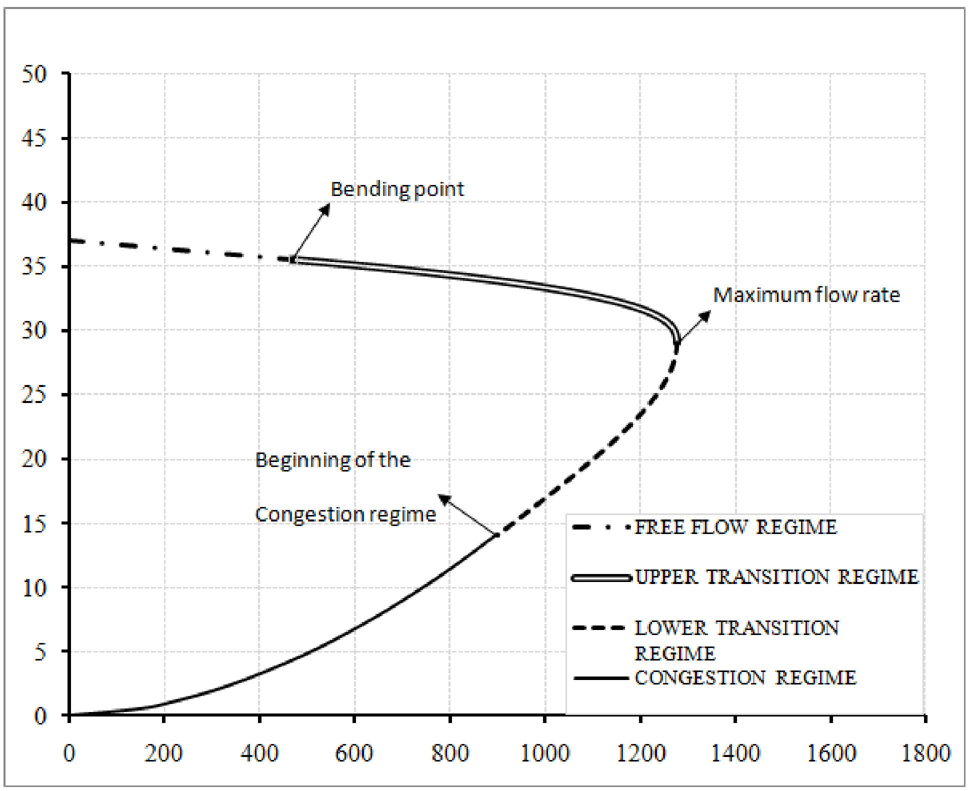


Figure 6-4. General form of four-regime speed –flow curves.

## 6.10 GENERAL FORM OF FOUR-REGIME SPEED-FLOW CURVES

The linear and the power functions introduced in three-regime models are also used for the free flow and congested regimes in the four-regime models, respectively.

On the other hand, polynomial functions of degree four were used for the upper and lower transition regimes. Equation 6.4 shows the general form of these functions.

$$Q = A * U^4 + B * U^3 + C * U^2 + D * U + E \tag{6.4}$$

Where

$Q$ and $U$ are the flow rate (pcphpl) and speed (mph), respectively.

$A$, $B$, $C$, $D$, $E$ are the coefficients of the polynomial function.

How to find the coefficients and domain of these functions are discussed as below.

### 6.10.1 Calculations of the Polynomial Coefficients

The coefficients of the free flow models and the congested models are based on the regression analyses achieved for the three-regime speed-flow curves. However, the intercept of the free flow models may change slightly for some cases after the curves are fine-tuned. The amount of change is less than or equal to 1 mph and the intercepts does not exceed the average speed of all free flowing vehicles.

The coefficients of the functions for upper transition and lower transition regimes might be different. Given the points at which two adjacent regimes are intersecting, the coefficient were calculated such that the functions of the two adjacent regimes have equal value, equal first derivative and equal second derivative at the intersecting points. In addition, the first derivative of the upper and lower transition regimes at the point with the maximum flow rate was set to be zero.

It should be noted that the slope of the free flow regimes in the speed-flow curves for work zones with speed limit of 55 mph is zero. Thus, the slope of the upper transition curve at the bending point should theoretically be zero. Nevertheless, the upper transition curves obtained to satisfy this requirement do not display reasonable trend. To overcome this issue, it was decided to use 1/400 as the slope of the upper transition curve at the bending point. The discontinuous change in flow at the bending point from zero to 1/400 is practically negligible.

### 6.10.2 Determining the Domains of Polynomial Functions

Once the intersection points of the adjacent regimes are determined, the domain of each function becomes known.

The intersection points were determined based on engineering judgment and trial and error. First, it was decided to examine potential bending points between 600-700 pcphpl for work zones with speed limit of 45 mph, and between 800-900 pcphpl for work zones with speed limit of 55 mph. The authors believe that work zones with a traffic flow rate lower than the above-mentioned ranges are in free flow condition, and traffic gradually goes into the transition regime at higher flow rates.

The lower points with flow rate of 900 pcphpl and 1200 pcphpl were taken as the initial trail points for the intersection of the lower transition and the congested curve. These values are equal to 75% of the lowest capacity obtained using the field data for work zones with the given speed limit. Referring to Table 5-2, the lowest suggested capacity for work zones with speed limit of 45 and 55 mph are 1200 and 1600 vph and 75 percent of these values are 900 vph and 1200 vph, respectively. The authors believe that these are good initial trail points since selecting very high flow rates does not provide smooth change in slope as desired. Besides, choosing a too low flow rate causes considerable diversion from the congested curve constructed based on the field data for the three-regime models.

The following rules were developed to determine the intersection point of the regimes:

1- The intersection point of the free flow regime and the upper transition regime is called the bending point. All bending points have a density of 13.25 veh/mile. In other words, all bending points lay on the following line.

$$Flow = 13.25 * Speed \tag{6.5}$$

2-The upper transition regime and lower transition regime are connected at the point with maximum flow rate. Similar to the three-regime models, the suggested capacity values and the suggested speed at capacity reported in Chapter 5 were used to determine the point with the with maximum flow rate for each work zone traffic condition.

3-The flow rate at the intersection point of the lower transition regime and the congested regime is 900 vph for work zones with speed limit of 45 mph. On the other hand, it is equal to 1200 mph for work zones with speed limit of 55 mph for intercepts greater than or equal to 58 mph, and then it reduces for lower speed intercepts.

The above-mentioned rules to determine the intersecting points provide continuous change in slope as well as close agreement between the 4$^{th}$-degree polynomial functions and the three-regime models.

In Appendix A, look-up tables are provided so that one can determine the intersection points of different regimes for different speed intercepts.

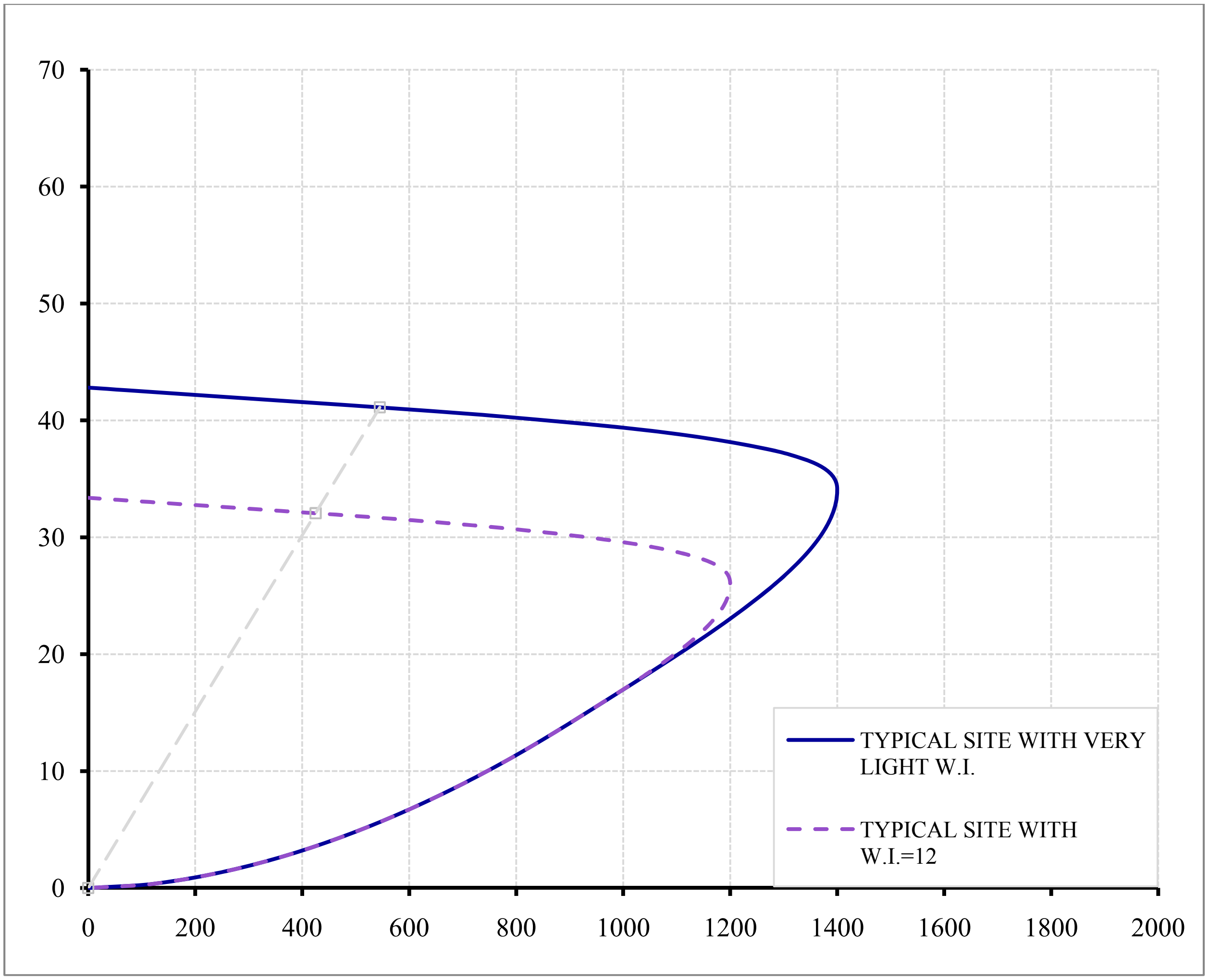


Figure 6-5. Four-regime speed-flow curves for a site with speed limit of 45 mph and flagger.

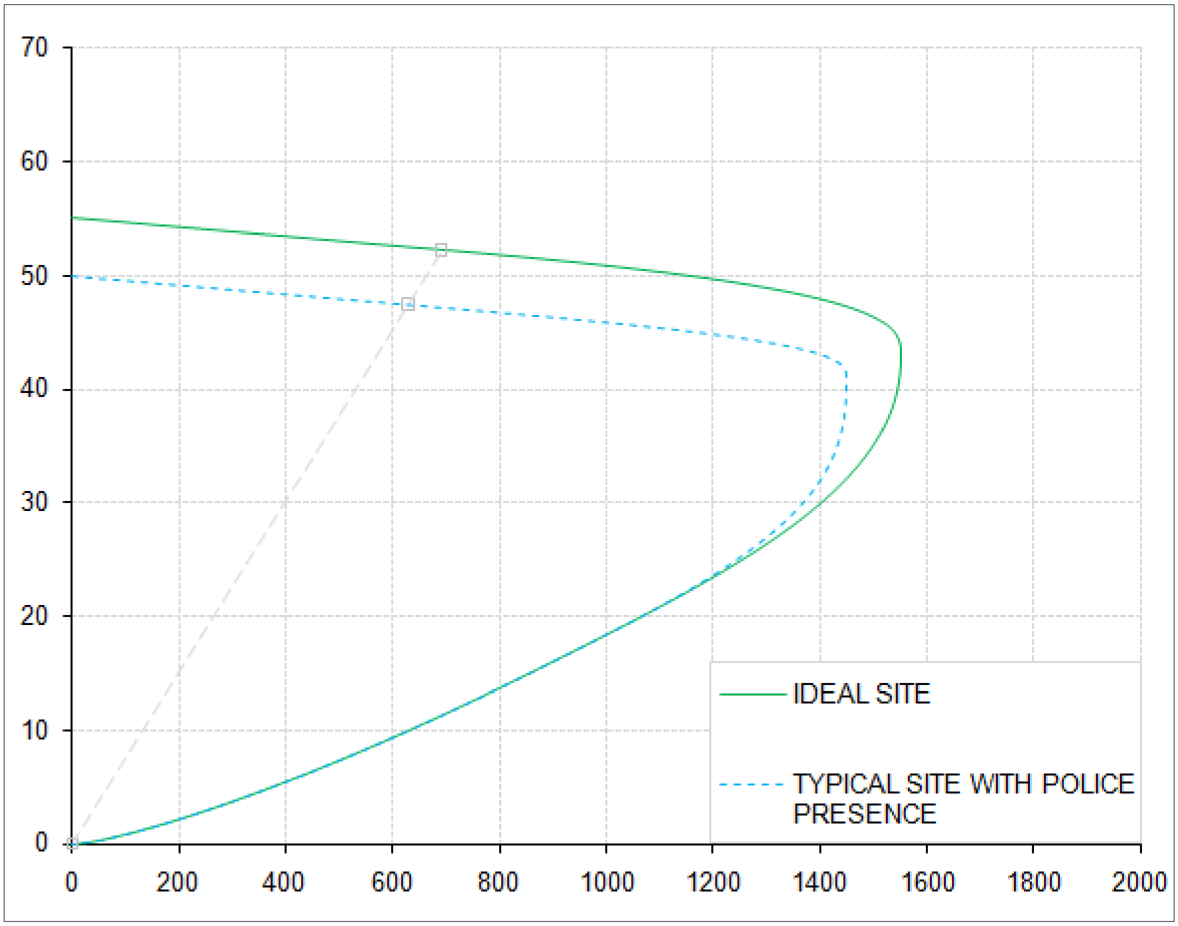

Figure 6-6. Four-regime speed-flow curves for a site with speed limit of 45 mph and no flagger.

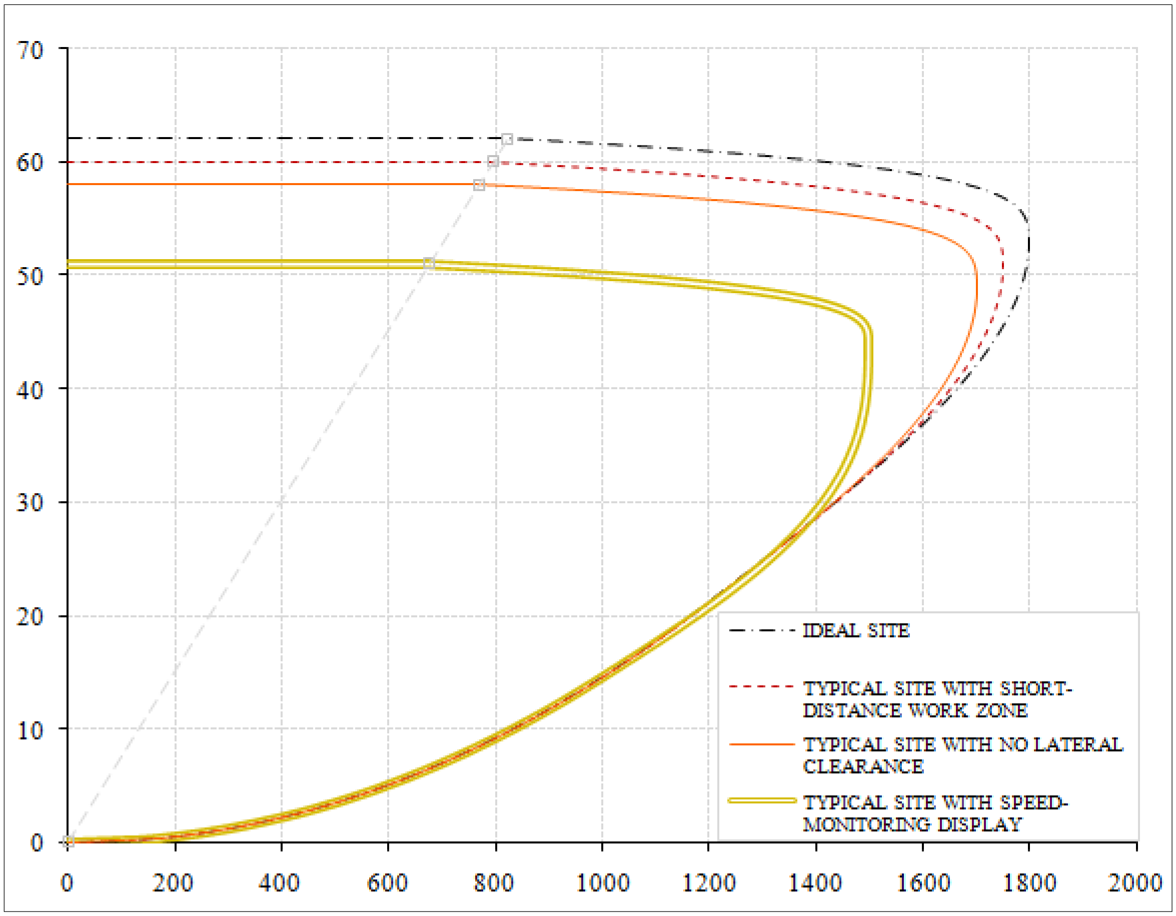

Figure 6-7. Four-regime speed-flow curves for a site with speed limit of 55 mph.

Table 6-10. Proposed Speed-Flow Relationships for Four-regime Models

| Traffic Condition | Free-Flow Equation | Upper Transition Equation | Lower Transition Equation | Congestion Equation |
|---|---|---|---|---|
| Typical site with very low work activity, flagger, no queue, SL=45 | $U = 42.81 - 0.0031 * Q$ | $Q = 0.0588 * U^4 - 10.5 * U^3 + 656 * U^2$ $-17{,}622 * U + 174{,}046$ | $Q = -0.000698 * U^4 + 0.0284 * U^3$ $-0.371 * U^2 + 36.3 * U + 410$ | $Q = 211.56 * U^{0.5472}$ |
| Typical site with flagger, queue, high work intensity, SL=45 | $U = 33.38 - 0.0031 * Q$ | $Q = 0.167 * U^4 - 21.2 * U^3 + 975 * U^2$ $-19{,}365 * U + 142{,}666$ | $Q = -0.00341 * U^4 + 0.164 * U^3$ $-2.87 * U^2 + 56.4 * U + 352$ | $Q = 211.56 * U^{0.5472}$ |
| Ideal site with no work activity, no queue, no flagger, SL=45 | $U = 55.00 - 0.0040 * Q$ | $Q = 0.0292 * U^4 - 6.31 * U^3 + 490 * U^2$ $-16{,}387 * U + 202{,}788$ | $Q = 0.000651 * U^4 - 0.0848 * U^3$ $+3.08 * U^2 - 1.46 * U + 438$ | $Q = 109.30 * U^{0.7594}$ |
| Typical site with police, no work activity, no queue, no flagger, SL=45 | $U = 50.00 - 0.0040 * Q$ | $Q = 0.472 * U^4 - 83.4 * U^3 + 5{,}503 * U^2$ $-160{,}568 * U + 1{,}751{,}244$ | $Q = 0.00122 * U^4 - 0.142 * U^3$ $+4.95 * U^2 - 26.8 * U + 561$ | $Q = 109.30 * U^{0.7594}$ |
| Ideal site with no work activity, no queue, SL=55 | $U = 62.00$ | $Q = -0.113 * U^4 + 23.8 * U^3 - 1{,}877 * U^2$ $+65{,}771 * U - 862{,}991$ | $Q = -0.0000445 * U^4 - 0.00342 * U^3$ $+0.337 * U^2 + 19.6 * U + 675$ | $Q = 271.43 * U^{0.4868}$ |
| Typical site with no work activity, no queue, short distance work zone, SL=55 | $U = 60.00$ | $Q = -0.124 * U^4 + 25.1 * U^3 - 1{,}916 * U^2$ $+64{,}954 * U - 824{,}543$ | $Q = -0.00000749 * U^4 - 0.00941 * U^3$ $+0.618 * U^2 + 14.3 * U + 710$ | $Q = 271.43 * U^{0.4868}$ |
| Typical site with no work activity, no queue, SL=55 | $U = 58.00$ | $Q = -0.134 * U^4 + 26.3 * U^3 - 1{,}938 * U^2$ $+63{,}445 * U - 777{,}496$ | $Q = 0.0000565 * U^4 - 0.0188 * U^3$ $+1.04 * U^2 + 6.61 * U + 760$ | $Q = 271.43 * U^{0.4868}$ |
| Typical site with no work activity, dynamic speed monitoring sign, no queue, SL=55 | $U = 51.00$ | $Q = -0.138 * U^4 + 22.8 * U^3 - 1{,}412 * U^2$ $+38{,}646 * U - 393{,}218$ | $Q = 0.000622 * U^4 - 0.0735 * U^3$ $+2.42 * U^2 + 1.98 * U + 656$ | $Q = 271.43 * U^{0.4868}$ |

U= Speed in mph
Q= Flow in pcphpl

# CHAPTER 7 APPLICATION OF SPEED-FLOW RELATIONSHIP

Operating speed and capacity can be determined by using speed-flow relationship. These parameters are used in delay and queue length estimation. This chapter demonstrates how to find capacity and operating speed by using the speed-flow curves. Two cases are considered: when traffic is operating without the traffic stream stopping, which is called Normal Case, and when traffic has to stop intermittently which is called Interrupted Case.

In Normal Case, there is no external component (such as police or flagger to stop traffic) to make drivers to stop intermittently, so drivers move through the work zone without stopping. In Interrupted Case drivers are asked to stop for a certain time period and then move (e.g. flagger stops the traffic for short time to allow construction activities to progress or an incident happens and blocks the open lane temporarily).

First, it is elaborated how to determine the capacity for a Normal Case when the speed-flow relationship is given. Thereafter, the parameters affecting the operating speed are introduced and then it is explained how to quantify them and find the operating speed for Normal Case. Finally, a model is proposed to estimate the capacity and the corresponding operating speed interrupted Case.

## 7.1 CAPACITY ESTIMATION FOR NORMAL CASE

When a speed-flow curve is given for a site, the capacity is the flow rate at the intersection point of the upper transition and lower transition curves. In other words, the capacity is the maximum possible flow rate which can be read from speed-flow curve. As shown in Figure 7.1, for a site with speed limit of 45 mph, a flagger and free flow speed of 42.81 mph, the intersection point of the upper transition and lower transition curves is point A and the flow rate at this point is equal to 1400 pcphpl.

The flow rate read from the speed-flow curve is in pcphpl. In order to incorporate the adverse effect of heavy vehicles on capacity, Equation 7.1 is used.

$$C_{adj} = C \times f_{HV} \qquad (7.1)$$

Where
$C_{adj}$ = Adjusted capacity (vphpl),
C = Capacity under the prevailing conditions in the work zone (pcphpl),
$f_{HV}$= Heavy vehicle adjustment factor.

Heavy vehicle adjustment factor is computed as below:

$$f_{HV} = \frac{1}{1 + P_T\,(PCE - 1)} \qquad (7.2)$$

Where
$f_{HV}$= Heavy vehicle adjustment factor
$P_T$ =Percentage of heavy vehicles (entered as decimal)
PCE=Passenger car equivalence factor determined using Table 7.1 when no grade is long or steep enough to cause a significant speed reduction on trucks (when no one grade of 3% or greater is longer than 0.25 mile or where no one grade of less than 3% is longer than 0.5 mile). Otherwise, PCE should be obtained from Exhibit 23-9 of the Highway Capacity Manual.)

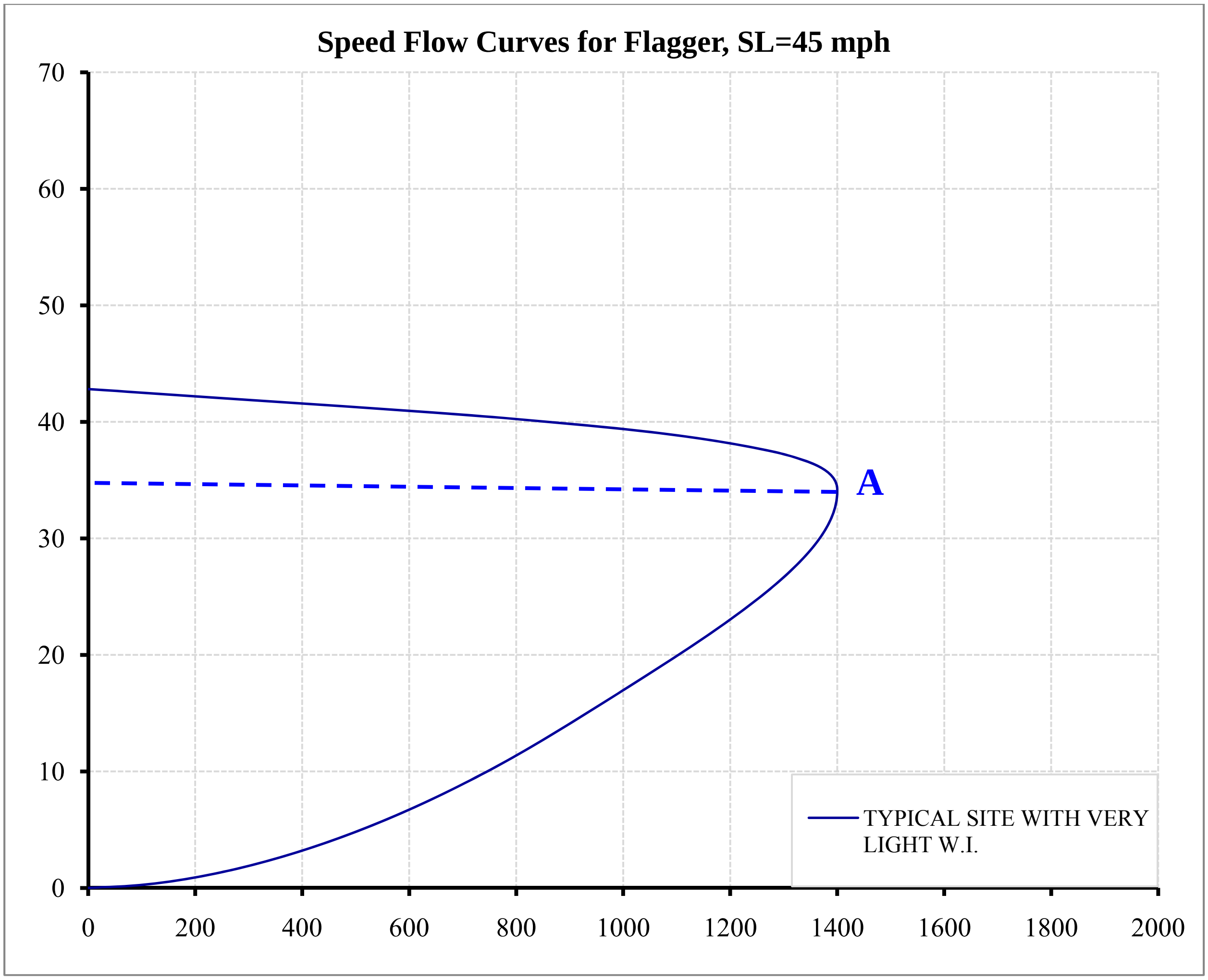

Figure 7-1. The speed-flow curve for a site with 45-mph speed limit and flagger.

Table 7-1. Passenger Car Equivalence

| Passenger Car Equivalence | Type of Terrain | | |
|---|---|---|---|
| | Level | Rolling | Mountainous |
| Trucks and Buses | 1.5 | 2.5 | 4.5 |

### 7.2 OPERATING SPEED FOR NORMAL CASE

Operating speed is the speed at which traffic travels through the work zone under the prevailing conditions. Conceptually, the procedure to determine the operating speed consists of three steps: First, the free-flow speed is adjusted due to non-ideal geometric conditions, work intensity, presence of a treatment and other speed-reducing factors. Second, the speed-flow curve corresponding to the adjusted free-flow speed is chosen. Finally, the operating speed at a given flow rate can be read from the speed-flow curve. Each of these three steps is explained in the following.

### 7.2.1 Adjusted Free Flow

Adjusted free flow is computed by Equation 7.3:

$$AFFS = FFS - R_{WI} - R_{LW} - R_{LC} - R_T - R_o \quad (7.3)$$

Where

AFFS= Adjusted free flow speed,

FFS =Free-flow speed (when there is no field data, FFS=62 mph for speed limit of 55 mph, FFS=55 mph for speed limit of 45 mph and no flagger, and FFS=43 mph for speed limit of 45 mph if there is a flagger in the work zone)

$R_{WI}$ = Reduction in speed due to work intensity (mph),

$R_{LW}$ = Reduction in speed due to lane width (mph),

$R_{LC}$ =Reduction in speed due to lateral clearance (mph),

$R_T$= Speed reduction due to treatment (mph), and

$R_o$ = Reduction in speed due to all other factors that may reduce speed (mph) (including those that may cause a flow breakdown).

In the following subsections, it is explained how to quantify these factors.

*7.2.1.1 Speed Reduction due to Less-than-ideal Lane Width and Lateral Clearance*

Speed reductions due to lateral clearance and lane width are determined from Table 7.2.

Table 7-2. Adjustment due to Lane Width and Lateral Clearance

| Adjustment for lane width | | | | |
|---|---|---|---|---|
| **Lane width (ft)** | **Reduction in speed (mph)** | | | |
| **12 ft or more** | 0.0 | | | |
| **11** | 1.9 | | | |
| **10** | 6.6 | | | |
| **9** | 15.0* | | | |
| **8** | 25.0* | | | |
| **Adjustment for left shoulder** | | | | |
| **Left shoulder (ft) width** | **Reduction in speed (mph)** | | | |
| **2 ft or more** | 0 | | | |
| **1** | 1 | | | |
| **0** | 2 | | | |
| **Adjustment for right shoulder** | | | | |
| **Right shoulder width (ft)** | **Reduction in speed (mph)** | | | |
| | **No of Lanes in one direction (without work zone)** | | | |
| | **2** | **3** | **4** | **>= 5** |
| **6 ft or more** | 0 | 0.0 | 0.0 | 0.0 |
| **5** | 0.6 | 0.4 | 0.2 | 0.1 |
| **4** | 1.2 | 0.8 | 0.4 | 0.2 |
| **3** | 1.8 | 1.2 | 0.6 | 0.3 |
| **2** | 2.4 | 1.6 | 0.8 | 0.4 |
| **1** | 3.6 | 2.0 | 1.0 | 0.5 |
| **0** | 3.9 | 2.4 | 1.2 | 0.6 |

*: Based on the authors' best estimate

*7.2.1.2 Speed Reduction due to Work Intensity*

In order to quantify the speed reduction due to work intensity, the level of work intensity should be determined first. Three levels of work intensity, namely low, moderate and high, are defined. Knowing the number of construction equipment and workers in the work activity area and the lateral clearance of the work activity area to the open lane of travel, the level of work intensity can be determined by look-up Tables 7.3 and 7.4 for short term and long term work zones, respectively. After determining the level of work intensity, the speed drop corresponding to the level of work intensity is read from Table 7.5 for short term work zones and from Table 7.6 for long term work zones.

Tables 7.3 through 7.6 were originated from a study by Benekohal et al (2004). Based on this study, work intensity is computed by using Equation 7.4.

$$WI_r = \frac{w + e}{p} \tag{7.4}$$

Where
w = Number of workers working together as a group in the work activity area (w varies from 0 to a maximum of 10),
e = Number of large construction equipment in work activity area near the workers group (e varies from 0 to a maximum of 5)
p = Lateral distance between the work area and the open lane (feet) (p varies from 1 to a maximum of 9 ft).

After determining the work intensity, the speed reduction due to work intensity is computed by using Equation 7.5 for short term work zones and Equation 7.6 for long term work zones as follows:

$$SRs = 11.918 + 2.676 \ln(WIr) \tag{7.5}$$

Where
$SR_s$ = Speed reduction due to work intensity in short-term work zone (mph)

$WI_r$ = Work intensity ratio

$$SRL = 2.6625 + 1.2056 \ln(WIr) \tag{7.6}$$

Where,
SRL = Speed reduction due to work intensity in long term work zones (mph)
WIr = Work intensity ratio

The input parameters to use Equations 7.4 to 7.6 are the number of workers and equipment and the lateral clearance to the open lane of travel. Tables 2 to 5 were prepared based on Equations 7.4 to 7.6 in case the analyzer does not know the input parameters but he/she subjectively knows the level of work intensity. In such a case, since the analyzer knows the level of work intensity, then it is no longer needed to use Table 2 and 3 to find the level of work intensity. When the analyzer knows the input parameters, then Tables 2 and 3 can be used to determine the level of work intensity in a more accurate way. The range of speed reduction for each level of work intensity is reported in Table 4 and 5. Based on those speed ranges, a speed reduction value is suggested for each level.

Table 7-3. Work intensity in short term work zones

| | (# of workers) + (# of large construction equipment) in the work activity area | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Lateral distance between the work activity area and the edge of the open lane (ft) | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| 9 | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 8 | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 7 | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 6 | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 5 | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI |
| 4 | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI |
| 3 | LO | LO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI |
| 2 | LO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |
| 1 | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |

LO=Low work intensity MO=Moderate work intensity HI=High work intensity

Table 7-4. Work Intensity in Long Term Work Zones

| Lateral distance between the work activity area and the edge of the concrete barrier closer to the activity area (ft) | (# of workers) + (# of large construction equipment) in the work activity area: 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 9 | LO | LO | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO |
| 8 | LO | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO |
| 7 | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 6 | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 5 | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| 4 | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI |
| 3 | LO | LO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI |
| 2 | LO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI |
| 1 | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |

LO=Low work intensity MO=Moderate work intensity HI=High work intensity

Table 7-5. Speed Reduction due to Work Intensity in Short Term Work Zones

| **Work Intensity** | **Estimated Speed Reduction Range (mph)** | **Suggested Speed Reduction (mph)** |
|---|---|---|
| LOW | 6.04 - 10.80 | 8.00 |
| MODERATE | 10.81 - 14.40 | 12.00 |
| HIGH | 14.41 - 19.16 | 16.00 |

Table 7-6. Speed Reduction due to Work Intensity in Long Term Work Zones

| **Work Intensity** | **Estimated Speed Reduction Range (mph)** | **Suggested Speed Reduction (mph)** |
|---|---|---|
| LOW | 0.01 - 2.65 | 2 |
| MODERATE | 2.66 - 3.80 | 3 |
| HIGH | 3.81 - 5.93 | 5 |

*7.2.1.3 Speed Reduction due to Treatment*

Generally, presence of a treatment like police or Speed Photo Enforcement (SPE) reduces vehicle speeds. Table 6 provides speed reduction data for several types of treatments. These data are extracted from several studies in this area. The range of speed reduction for each type of treatment is reported in Table 6 and based on that a speed reduction value is suggested.

Table 7-7. Speed Reduction due to Treatment

| **Speed Control Technique** | **Observed Range of Speed Reduction (mph)** | **Suggested Speed Reduction (mph)** |
|---|---|---|
| Changeable Message Signs[1] | 1.4 - 4.7 | 3.0 |
| Drone Radar[2] | 1.2 - 9.8 | 2.5 |
| Police Presence[3] | 4.3 - 5.0 | 4.5 |
| Speed Photo Enforcement[3] | 3.4 - 7.8 | 5.0 |
| MUTCD Flagging[4*] | 3.0 - 12.0* | 7.0* |
| Innovative Flagging[4*] | 4.0 - 16.0* | 10.0* |
| Changeable Message Signs with Radar[5] | 4.0 - 8.0** | 5.0** |
| Speed Monitoring Display[5] | 4.0 - 5.0** | 4.0** |

**: In the case of flagger, the authors recommend the use of the speed-flow curves for a speed limit of 45 mph and flagger in the work zone rather than the suggested speed reduction shown in the table.*

1 Benekohal et al (1992)
2 Benekohal et al (1993)
3 Benekohal et al (2008)
4 Richards et al (1985)
5 Mattox et al (2007)

### 7.2.2 Selecting a Proper Speed-Flow Curve

In the previous chapter, speed-flow curves were proposed for three different types of work zones: work zones with speed limit of 45 mph, work zones with speed limit of 45 mph and a flagger, and work zones with speed limit of 55 mph. According to the prevailing conditions in the work zone, one will select one of the speed-flow curves mentioned above.

After selecting the appropriate speed-flow curve, the adjusted free flow speed in the work zone should be estimated. For each of these work zones, two different speed-flow curves were developed based on field data. For instance, as shown in Figure 7.2, for a work zone with speed limit of 45 mph and a flagger, two different speed-flow relationships are presented: one for free flow speed of 42.81 mph and the other for free flow speed of 33.38 mph. When adjusted free flow speed is either 42.81 or 33.38 mph, the analyzer can easily select the corresponding speed-flow curve. When adjusted free flow speed is between 42.81 and 33.38 mph, then the new speed-flow curve is obtained by interpolation between the existing curves. For adjusted free flow less than 33.38 mph, one may draw a curve similar to that of 33.38 mph. The details on finding the intersection points for a given intercept are provided in Appendix A, but for the purpose of discussion, it is assumed in the following sections that the speed-flow curve is given.

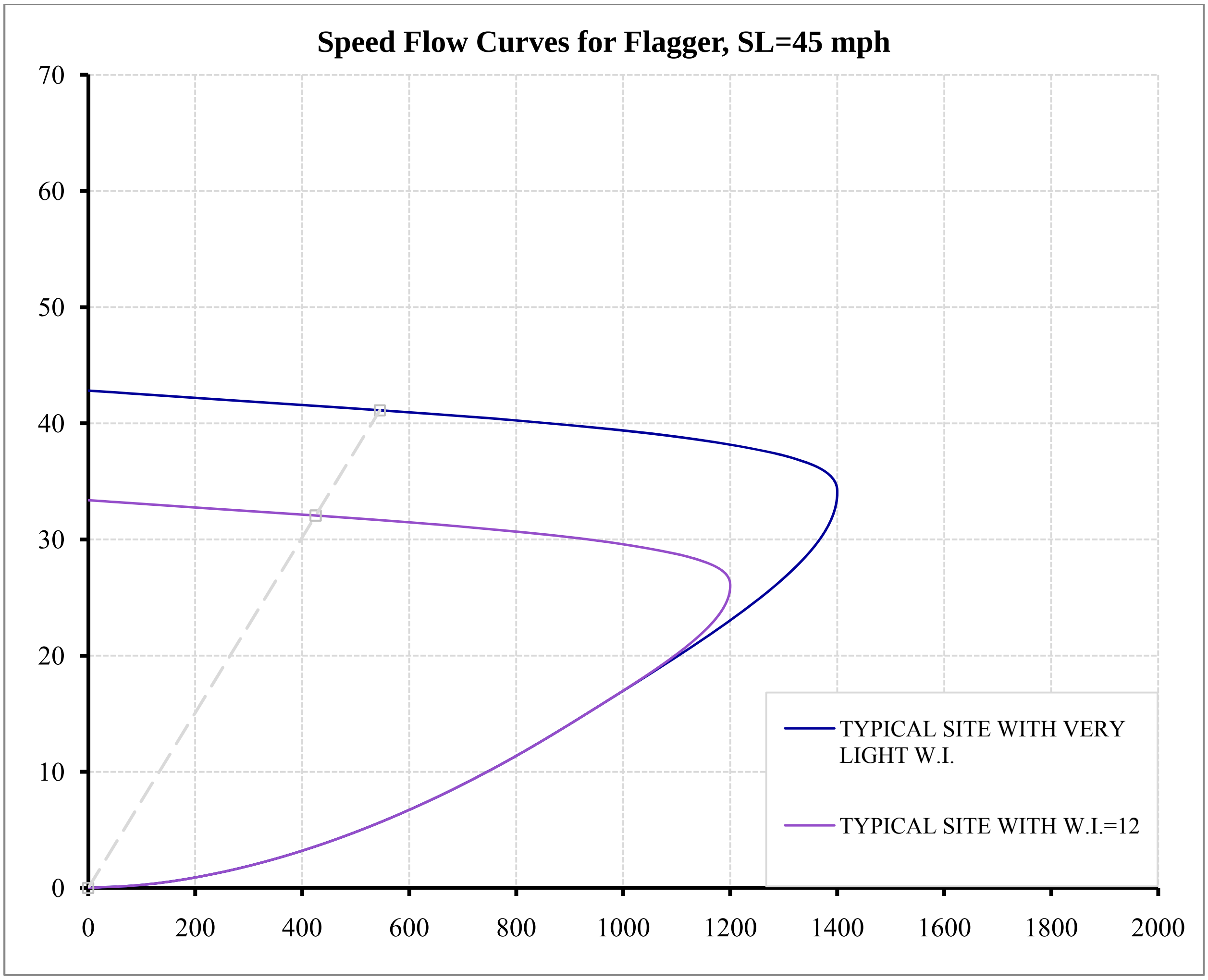

Figure 7-2. Speed-flow curves for work zones with speed limit of 45 mph and a flagger.

### 7.2.3 Reading the Operating Speed from the Speed-Flow Curve

In this section, the operating speed is obtained for a site with a given traffic flow rate and speed-flow curve. Two conditions can occur: undersaturated conditions or oversaturated conditions. In order to differentiate these conditions, the capacity should be read from the speed-flow curve and should then be converted to vehicle per hour. If the traffic volume is less than the capacity, then the traffic will be in undersaturated conditions; otherwise, it is in oversaturated conditions.

When the traffic is in undersaturated conditions, the operating speed is the speed on upper branch of the speed-flow curve corresponding to the traffic flow rate. As shown in Figure 7.2, if the free flow speed is 33.38 mph for a site with speed limit of 45 mph and a flagger, then the operating speed at flow rate of 600 pcphpl will be 31.52 mph.

In the case of oversaturated conditions, a speed-flow curve corresponding to the adjusted free flow speed needs to be drawn and it is assumed that the flow rate is equal to the capacity obtained from the speed-flow curve and the operating speed is corresponding to the speed at capacity. As shown in Figure 7.1, for a site with flagger and speed limit of 45 mph and adjusted free flow speed of 42.82 mph, the operating speed is 31.61 mph when the flow rate is 1400 pcphpl.

## 7.3 CAPACITY AND OPERATING SPEED FOR INTERRUPTED CASE

It is assumed to be known that during $t_1$ minutes of the study interval, vehicles are stopped and during the remaining interval, $t_2$ minutes, vehicles are moving. It is assumed that the flow rate during $t_1$ is equal to the maximum flow rate which is equal to the capacity under normal conditions. Therefore there are two different time intervals with different traffic conditions. Then the capacity when traffic is asked to stop for a certain time interval is the weighted average of the flow rate during these two time intervals and can be determined by using Equation 7.7.

$$\bar{C} = \frac{(60 - t_1) * C_{Max}}{60} \tag{7.7}$$

Where

$\bar{C}$ = Average capacity (pcphpl) when traffic is asked to stop

$C_{Max}$= Maximum flow rate (pcphpl) which is equal to the capacity in normal conditions.

$t_1$ = Interval during which traffic is stopped

In order to find the average operating speed, one may use Equation 7.8.

$$\bar{U} = \frac{(60 - t_1) * U_O}{60} \tag{7.8}$$

Where

$\bar{U}$ = Average speed (mph) when traffic is asked to stop

$U_O$= Operating speed (mph) at maximum flow rate (pcphpl)

$t_1$ = Interval during which traffic is stopped

# CHAPTER 8 DELAY AND USER'S COST IN WORK ZONES

This section presents methodologies to estimate delay and user's cost in work zones. The reference speed for delay calculations is the posted speed limit. Hence, the vehicles going slower than the reference speed are assumed to experience delay. Drivers may encounter delay under either undersaturated or oversaturated conditions. For each case, a delay computation model is suggested, and then, users' cost is computed based on the delay that drivers experience in a work zone.

## 8.1 DELAY MODEL FOR COMPLETELY UNDERSATURATED INTERVAL

Vehicles in undersaturated conditions may experience delay due to any factor that causes speed reduction such as work intensity, less-than-ideal lane width, and lateral clearance. For such a condition, Equation 8.1 estimates the average delay for a link.

$$d_u = \begin{cases} \left(\dfrac{E}{U_o} - \dfrac{E}{SL}\right) & \text{if } U_o < SL \\ 0 & \text{Otherwise} \end{cases} \qquad (8.1)$$

Where

$d_u$= Delay experienced by each vehicle due to the an operating speed less than the speed limit (hour/vehicle),

E= The distance from the end of the buffer space to the end of the activity area (link length).

$U_0$=Operating speed (mph) on the link,

SL=Speed limit (mph) of the link

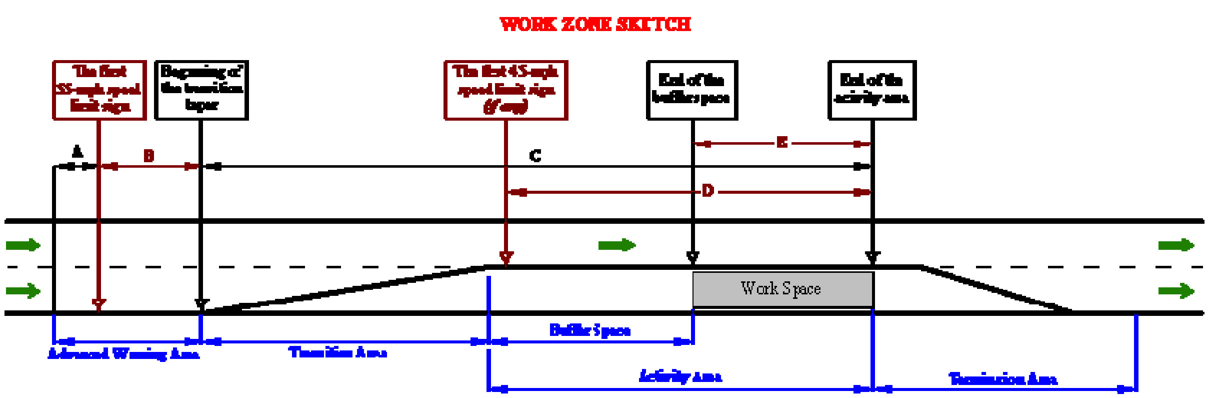


Figure 8-1. Work zone sketch.

Total delay is also obtained by Equation 8.2:

$$D_u = d_u * V \qquad (8.2)$$

Where

$D_u$=Total delay in undersaturated condition (hour)

$d_u$ =Delay experienced by each vehicle in undersaturated condition (hour/Vehicle)

$V$=Number of vehicles arrived in the interval (vehicles)

Total delay for each data set in which the traffic was in undersaturated condition is estimated and shown in Table 8.1. Average speed in each minute was computed and Equation 8.1 and 8.2 were used to compute total delay.

## 8.2 VALIDATION OF THE DELAY ESTIMATION FOR DATA SETS WITH UNDERSATURATED CONDITIONS

As mentioned in the previous section, total delay was computed for each data set with undersaturated condition. Those results were based on the average speed and arriving volume during each minute; however, one may have these data for larger intervals (5-minute or 10-minute intervals). How the results are sensitive with respect to the interval length was investigated. In particular results based on the three-minute and five-minute intervals will be compared with the total delay computed using one-minute data. The same procedure (see section 8.1) used for one-minute data was also used to compute total delay for larger intervals. The results are shown on Table 8.2.

Table 8-1.Total Delay in Undersaturated Condition Based on Field Data

| Site | Data Set | SL* (mph) | Average Speed of Traffic (mph) | Length of the Work Space (mile) | Counted Volume (vehicle) | Total Delay (hr) |
|---|---|---|---|---|---|---|
| I39NB | PM, very low work activity, with flagger and no queue, | 45 | 37.98 | 1.5 | 690 | 4.8 |
| | PM, no work activity, no queue, and no flagger | 55 | 48.10 | 1.5 | 645 | 2.9 |
| I80EB | No work activity, no queue, no treatment | 45 | 47.95 | 3.6 | 485 | 0.3 |
| I80WB | No work activity, no queue, no treatment | 45 | 45.33 | 1.5 | 730 | 1.7 |
| I72EB | Morning peak, no work activity, no queue, police | 45 | 43.59 | 0.1 | 657 | 0.1 |
| | Noon Peak, no work activity, no queue, no treatment | 45 | 44.67 | 0.1 | 621 | 0.0 |
| I74EB | AM, no work activity, no queue, no treatment | 55 | 54.20 | 0.1 | 549 | 0.0 |
| | PM, no work activity, no queue, no treatment, intermittently rainy | 55 | 53.64 | 0.1 | 600 | 0.1 |

* Work zone speed limit in work space

The percentage of error in three-minute estimations ranges from 0% to 67% and it varies between 0% and 100% for five-minute interval estimations. As interval length gets longer, the percentage of error becomes larger or stays the same. If one ignores the results of I80EB, the percentage of error will be less than 11.8% for both three-minute and five-minute intervals. Hence five-minute-input data can be used with low amount of error.

Table 8-2.The Estimated Total Delay for Different Interval Lengths

| Site | Data Set | One-minute Intervals | Three-minute Intervals | | Five-Minute Intervals | |
|---|---|---|---|---|---|---|
| | | Total Delay (hr) | Total Delay (hr) | % error | Total Delay (hr) | % error |
| I39NB | PM, very low work activity, with flagger and no queue, with dynamic feedback sign | 4.8 | 4.8 | 0.0 | 4.8 | 0.0 |
| | PM, no work activity, no queue, and no flagger, with dynamic feedback sign | 2.9 | 2.9 | 0.0 | 2.8 | 3.4 |
| I80EB | No work activity, no queue, no treatment | 0.3 | 0.1 | 66.7 | 0.0 | 100 |
| I80WB | No work activity, no queue, no treatment | 1.7 | 1.5 | 11.8 | 1.5 | 11.8 |
| I72EB | Morning peak, no work activity, no queue, police | 0.1 | 0.1 | 0.0 | 0.1 | 0.0 |
| | Noon Peak, no work activity, no queue, no treatment | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| I74EB | AM, no work activity, no queue, no treatment | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| | PM, no work activity, no queue, no treatment, drizzle rain | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |

## 8.3. DELAY MODEL FOR COMPLETELY OVERSATURATED INTERVALS

In oversaturated conditions, demand is greater than capacity of the road section and vehicles experience queuing and delay. Three types of queuing conditions are considered:

**1-Stopped queue**: Vehicles in queue are stopped or their speed is so low that they can be assumed as practically stopped.

**2-Moving queue:** Vehicles in queue are moving at considerable speed. For the purpose of modeling, it is assumed that the vehicles in queue are moving at constant speed.

**3-Combination of stopped and moving queue:** Traffic stops for a certain time period and then moves. An example of this condition is a work zone where traffic is intermittently stopped to allow construction vehicles to get in and out of the work space.

For all queuing types, one needs to determine the number of vehicles in queue. This is discussed in the next section.

### 8.3.1 Number of Vehicles in Queue

In order to compute the queue length, the number of vehicles in queue is computed as follows:

$n_i=n_{i-1}+V_i-C_{adj}*N_{op}$ (8.3)

Where
$n_i$ = Number of vehicles in queue at the end of the $i^{th}$ interval
$n_{i-1}$ =Number of vehicles in queue at the end of the $(i-1)^{th}$ interval
$V_i$ = Total arriving volume in the $i^{th}$ interval
$C_{adj}$= Departure capacity of a single lane of the work zone, adjusted due to heavy vehicles (See Equation 7.1) during the time interval
$N_{op}$= Number of open lanes in the work zone

The number of vehicles in queue is used to estimate the queue length and corresponding delay as discussed in the following section.

### 8.3.2 Queue Length and Delay in Stopped Queue

Knowing the number of vehicles in queue (from equation (8.3)), the queue length in a work zone with one lane open is estimated using Equation 8.4:

$Q_i = n_i * l/5280$ (8.4)

Where
$Q_i$= Queue length at the end of the $i^{th}$ interval, usually an hour (mile)
$n_i$ = Number of vehicles in queue at the end of the $i^{th}$ interval.
l = Average spacing between vehicles (ft)

The average spacing between vehicles, l, is computed considering the percentage of heavy vehicles. Equation 8.5 is used to compute in stopped queue.

$l=P_T*l_T+P_C*l_C+\text{buffer space}$ (8.5)

Where,
$P_T$= Percentage of heavy vehicles
$l_T$=Length of heavy vehicles (ft)
$P_C$= Percentage of passenger cars
$l_C$=Length of passenger cars (ft)
Buffer space=Spacing gap between two successive vehicles. For stopped queue it is assumed to be 10 ft

Delay in stopped queue is estimated using Equation 8.6:

$D_{qi} = \frac{n_i+n_{i-1}}{2}(\text{interval length})$ (8.6)

Where,
$D_{qi}$=Total delay due to stopped queue during the $i^{th}$ interval
$n_{i-1}$ and $n_i$ are the number of vehicles in queue at the end of the $(i-1)^{th}$ and $i^{th}$ interval of congestion, respectively.

### 8.3.3 Queue Length and Delay in Moving Queue

Similar to the stopped queue condition, Equation 8.4 is used to estimate the queue length in a work zone with one lane open. However, the average spacing for moving queue is computed from the following equation:

$$l = \frac{U_o}{C_{adj}} * 5280 \tag{8.7}$$

Where

l= Average spacing between vehicles (ft),

$C_{adj}$ = Adjusted capacity of the work zone (vphpl). It was discussed in section 7.1 how to determine adjusted capacity.

To compute the average delay for vehicles in moving queues, Equation 8.8 is used.

$$d_q = \frac{\overline{Q}}{U} + \overline{n}_c \overline{\gamma} - \overline{FFT} \tag{8.8}$$

Where

$d_q$=Average delay per vehicle in queuing condition (hour/vehicle),

$\overline{Q}$= Average queue length (mile),

U = Average speed of vehicles in the moving queue (mph),

$\overline{FFT}$=The time to travel the average queue length at work zone speed limit (hr)

$\overline{n}_c$= Average number of queued vehicles on the closed lane. It will be discussed how to estimate $\overline{n}_c$later.

$\overline{\gamma}$=The additional time for a vehicle to move from the closed lane to the open lane in the same queue position. It is suggested to use $\frac{1}{C_{adj}}$ for $\overline{\gamma}$ where $C_{adj}$ is the adjusted capacity of the work zone in vphpl.

Total delay is obtained by Equation 8.9:

$$D_q = d_q * V \tag{8.9}$$

Where

$D_q$d=Total delay under queuing condition (veh)

$d_q$ =Average queuing delay per vehicle estimated by Equation 8.8 (hour/vehicle)

V=Number of vehicles arrived in the interval (vehicle)

The logic behind Equation 8.8 is explained as follow. The term, $\frac{\overline{Q}}{U} + \overline{n}_c \overline{\gamma}$in Equation 8.8 reflects the average travel time under queuing condition. The first term of Equation 8.8, $\frac{\overline{Q}}{U}$, is equal to the average travel time assuming that U is the average speed of the vehicles in queue and there is no vehicle on the closed lane. However, in reality, some vehicles may be in the closed lane.

It is shown that the second term of Equation 8.8,$\overline{n}_c \overline{\gamma}$, is associated with the delay caused by the vehicles in the closed lane. Figure 8.2 shows a work zone at the time that vehicle "(n)o" joins the queue. The position of each vehicle in queue is shown in parenthesis. If a vehicle is in the open lane, the letter "o" comes after the parenthesis. The letter c is used for closed lane. For example (i)o means that the corresponding vehicle occupies the $i^{th}$ position in queue on the open lane. In Figure 8.2, the shoulder lane is closed and the vehicles in this lane need to move to the open lane.

It is assumed that the queue length shown in Figure 8.2 is equal to the average queue length. It can also be shown that the delay of vehicle (n)o is the average delay. It is also assumed that vehicles in a lower position have higher priority to enter the transition area. For example, after vehicle (i+1)o enters the transition area, vehicle (i+2)o has to slow down to let vehicle (i+1)c shift its lane and enter the transition area. This slowing down, causes vehicles beyond the position (i+1) to experience delay. It is assumed that the average delay caused by each lane shifting is $\bar{\gamma}$ hour. Consequently, vehicle (n)o experiences $\bar{n}_c\bar{\gamma}$, where $\bar{n}_c$is the number of vehicles on the closed lane at the time that vehicle (n)o joins the queue. Since it was assumed that queue length at this time is the average queue length, it can be shown that $\bar{n}$is the average number of vehicles on the closed lane.

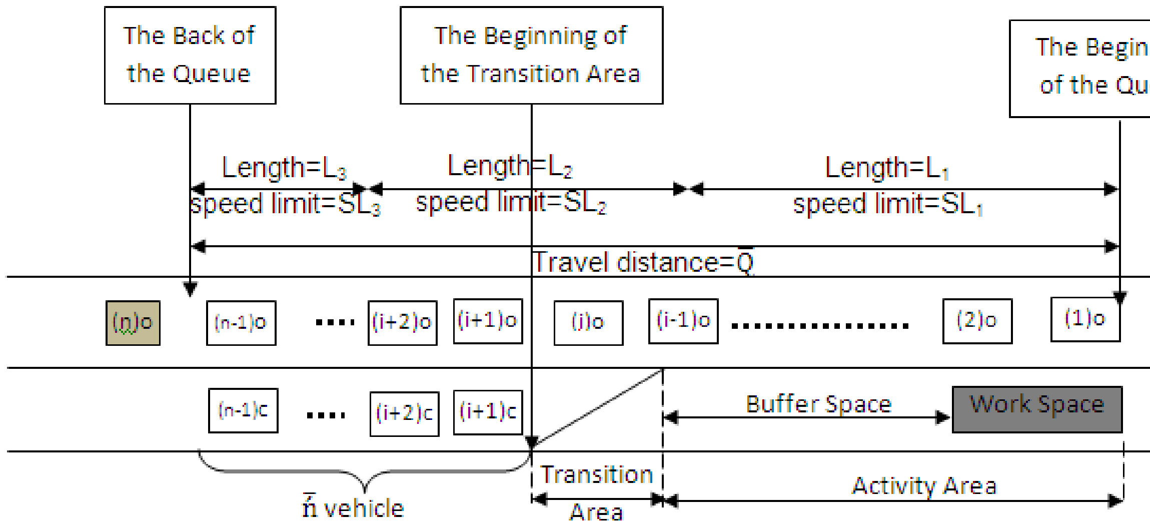


Figure 8-2. Plan of a work zone when the back of the queue extends to the upstream of the beginning of the transition area

If just one speed limit is in effect for the queued vehicles, $\overline{\mathrm{FFT}}$ is estimated as below:

$$\overline{\mathrm{FFT}} = \frac{\bar{Q}}{\mathrm{SL_{WZ}}} \tag{8.10}$$

Where

$\bar{Q}$= Average queue length (mile),

$SL_{WZ}$=Speed limit in the work zone (mph)

However, sometimes more than one speed-limit may be in effect for the vehicles in queue. In that case,$\bar{Q}$ should be decomposed into multiple parts such that each part has a uniform speed limit. Figure 8.2 displays an example of this case. For the case shown in Figure 8.2, $\overline{\mathrm{FFT}}$ can be estimated using Equation 8.11.

$$\overline{\mathrm{FFT}} = \frac{L_1}{SL_1} + \frac{L_2}{SL_2} + \frac{L_3}{SL_3} \tag{8.11}$$

In Equation 8.11, the speed limit of the $L_i$-mile-long section is $SL_i$ mph.

The average queue length, $\bar{Q}$, is estimated using Equation 8.12:

$$\bar{Q} = \frac{Q_b + Q_e}{2} \tag{8.12}$$

Where

$\bar{Q}$= Average queue length (mile),

$Q_b$ = Queue length at the beginning of the interval (mile),

$Q_e$ = Queue length at the end of the interval (mile).

The average number of vehicles in queue, $\bar{n}$, is estimated using 8.13.

$$\bar{n} = \frac{n_b + n_e}{2} \tag{8.13}$$

Where
$\bar{n}$= Average number of the vehicles in queue,
$n_b$ = Number of the vehicles in queue at the beginning of the interval,
$n_e$ = Number of the vehicles in queue at the end of the interval.

The number of vehicles on the closed lane at time t is estimated as below:

$$n_{ct} = \frac{Q_t - C}{l} * 5280 \tag{8.14}$$

Where
$Q_t$= Queue length at time t (mile)
C=The distance (See Figure 8.1) from the beginning of the transition area to the end of the activity area (mile).
l=The spacing (ft) between vehicles in queue which is estimated using Equation 8.7.

The average number of vehicles on the closed lane,$\bar{n}_c$, is estimated using Equation 8.15:

$$\bar{n}_c = \frac{n_{ce} + n_{cb}}{2} \tag{8.15}$$

Where$n_{cb}$ and $n_{ce}$are the number of queued vehicles on the closed lane at the beginning and end of the interval, respectively.

**Queue Length and Delay for Combination of Stopped and Moving Queue**

It is suggested to use the procedure developed for moving queue for the combination of stopped and moving queue; however, the adjustments proposed in section 7.3 must be applied to estimate the capacity and speed.

## 8.4 DELAY MODELS FOR PARTIALLY OVERSATURATED INTERVALS

Up to now, it was assumed that the traffic state does not change in the middle of the study period. In this section, this assumption is released for the case in which queue disappears somewhere in the middle of the interval and thereafter; the traffic stays in under-saturated conditions. As shown in Figure 8.3, it is assumed that there is an initial queue left from the previous interval and the rate of the arrival is less than departure. Hence, the queue is shrinking and it is assumed that somewhere in the middle of the interval, say at time T', the queue vanishes. In Figure 8.3, $C_{adjT}$ and $N_{op}$ are the number of vehicles that can be processed in each lane at the adjusted capacity level during time T and the number of lanes open in work activity area, respectively. The variable V is the volume arriving during time T and $n_0$ denotes the number of vehicles in queue in the beginning of the interval. Prior to time T', vehicles experience queuing delay, whereas after time T' they may only experience delay due to an operating speed less than the speed limit.

As shown in Figure 8.3, the cumulative demand (the solid green line) and the cumulative departure (the solid red line) curves meet each other at time T', which can be obtained from Equation 8.16:

$$T' = \frac{n_0 * T}{C_{adjT} * N_{op} - V_T} \tag{8.16}$$

Where
$T'$=Queuing duration (min)
T=Interval length (min)
$n_0$=Number of vehicles in initial queue
$V_T$=Volume arriving during T.
$N_{op}$=Number of open lanes
$C_{adjT}$= Number of vehicles that can be processed in each lane at the adjusted capacity level during time T and is computed using Equation 8.17.

$$C_{adjT} = C_{adj} * \frac{T}{60} \tag{8.17}$$

$C_{adj}$= Adjusted capacity (vphpl)
The average delay is computed using Equation 8.18:

$$\bar{d} = T'/T * \bar{d}_Q + (1 - T'/T) * \bar{d}_{NQ} \tag{8.18}$$

Where
$\bar{d}$=The average delay (hr) for the time interval [0,T]
$\bar{d}_Q$=The average delay (hr) during the queuing period which lasts until time T'.
$\bar{d}_{NQ}$=The average delay (hr) during the non-queuing period which starts at T' and ends at T.
$T'/T$=The proportion of the study interval,[0,T], with queuing condition and can be estimated using Equation 8.19:

$$\frac{T'}{T} = \frac{n_0}{C_{adjT} * N_{op} - V_T} \tag{8.19}$$

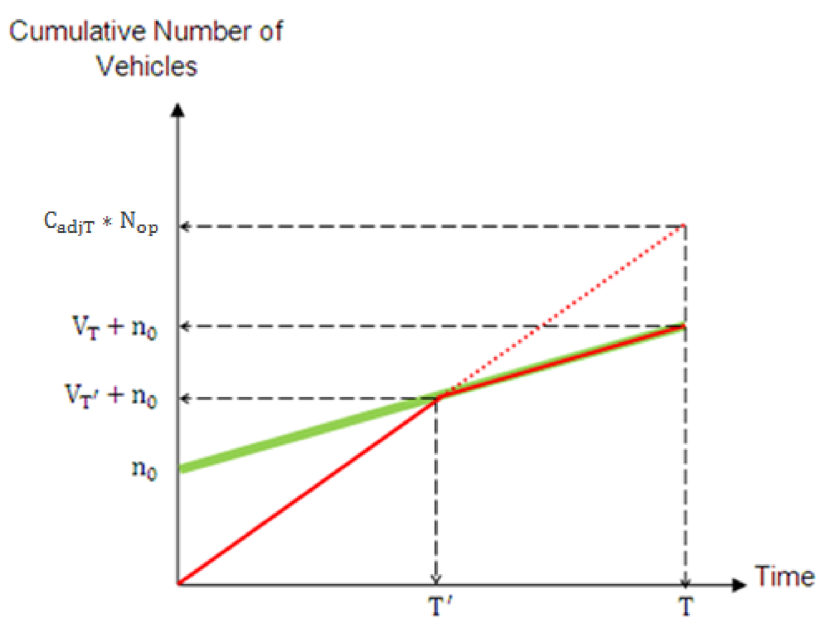


Figure 8-3. The cumulative arrival (solid green line) and departure (solid red line) for a partially oversaturated interval.

## 8.5 DELAY ESTIMATION USING FIELD DATA FOR THE SITES WITH QUEUE

In this section, delay is estimated for each data set with queuing condition. The five data sets, the corresponding sites and the average speed of the vehicles in queue are shown in Table 8.3. Since the speed of the vehicles in queue ranges from 16.9 mph to 37.5 mph, the proposed procedure to estimate total delay in moving queue should be used. This procedure assumes that arriving volume, departing volume and average speed in the queue are known. In the field, the departure volume and queue length data were recorded in one minute intervals. The one minute queue length data was used to find the moving average for queue length (the intervals before and after the current interval was used). Speed of each vehicle was determined from the video tapes taken in the field. Based on the volume and speed data, the average spacing between vehicles was computed for each one minute interval. Then the number of vehicles in queue (in one minute intervals) was computed based on the average spacing and queue length. Finally, arriving volume for each interval (one minute) was computed knowing the departure volume and the number of vehicles in queue.

The following steps were taken to obtain arriving volume for each minute:

1. In-platoon vehicles were detected using the criteria introduced in section 4.3.4. Average headway and speed of in-platoon vehicles were computed. Then the average queue departure rate for in-platoon vehicles was calculated.
2. Spacing of the vehicles in queue is estimated as the average speed of in-platoon vehicles divided by the average queue departure rate for in-platoon vehicles.
3. Number of vehicles in queue is computed by dividing the queue length obtained from field data by the spacing estimated in Step 2.
4. Arriving volume is computed from the following equation:

$$V_i = n_i + AD_i - n_{i-1} \quad (8.20)$$

Where

$n_{i-1}$ = Number of vehicles in queue at the end of the $(i-1)^{th}$ minute

$n_i$ = Number of vehicles in queue at the end of the $(i)^{th}$ minute

$V_i$ = Arriving volume in the $(i)^{th}$ minute (vehicle per minute)

$AD_i$ = Actual departing volume in the $(i)^{th}$ minute (vehicle per minute)

Once the arriving volume is known, one can follow the proposed method to estimate the delay in moving queue.

### 8.5.1 Error when Moving Queue Is Assumed as Stopped Queue

Since some of the delay estimation models assume that vehicles are in stopped queue, it is investigated how much this assumption causes error. Assuming the number of vehicles in stopped queue and moving queue are equal to each other; the total delay and stopped queue length were estimated and displayed in Table 8.3.

The results show that assuming stopped queue instead of moving queue underestimates the queue length by 57% to 81%. The base for the error calculation is the queue length recorded in the field. This underestimation is expected because the number of vehicles in queue on each site is the same for both the moving queue and stopped queue calculations, but the spacing between the vehicles in stopped queue is less than the spacing between the vehicles in moving queue. Therefore, for the same number of vehicles in queue, the stopped queue length is less than the moving queue length.

Delay in stopped queue was compared with the estimated delay (see section 8.4 for more details) based on the field data. The percentage of error for delay in stopped queue varies from38% to 969%. The lowest percentage of error belongs to I39NB for which the average speed of the vehicles in queue was the lowest and the traffic situation was closer to stopped queue condition compared to the other data sets. Moreover, the delay in stopped queue estimated from the PM data of I80EB with flagger and queue involves the largest amount of error. This is expected as this data set has the highest average speed of vehicles in queue and consequently has the farthest traffic condition from the stopped queue condition.

The magnitude of error in the queue length and delay estimation for stopped queue is considerable and thus implies that the designer should select an appropriate method for delay and queue length estimation. Otherwise, a large amount of error would be incorporated into the results.

Table 8-3. Delay and Queue Length Computed Based on the Field Data

| **Site** | I80 EB | | I80WB | | I39NB |
|---|---|---|---|---|---|
| **Data set** | AM, flagger, and queue | PM, flagger, and queue | AM, flagger, and queue | PM, flagger, and queue | flagger, and queue |
| **Average Speed** | 35.1 | 37.5 | 29.9 | 25.7 | 16.9 |
| **Average moving queue length (ft)** | 535 | 585 | 591 | 1244 | 2647 |
| **Average queue length (ft), assuming stopped queue** | 128 | 109 | 219 | 439 | 1128 |
| **Percentage of Error in queue length due to the assumption of stopped queue** | 76 | 81 | 63 | 65 | 57 |
| **Total delay (hr) due to moving queue** | 0.3 | 0.19 | 0.54 | 1.93 | 7.52 |
| **Total delay (hr), assuming stopped queue** | 1.98 | 2.02 | 3.22 | 5.92 | 10.35 |
| **Percentage of error in total delay due to the assumption of stopped queue length** | 562 | 969 | 498 | 207 | 38 |

### 8.6 VALIDATION OF THE MOVING QUEUE LENGTH AND DELAY ESTIMATION

Moving delay and queue length were estimated based on the field data in section 8.4. However, the minute by minute data may not be available and instead, traffic data over larger intervals (say five-minute or ten-minute interval) may exist. In that case, the method is still applicable but it assumes one departure rate and average speed for the entire interval while we may have some fluctuation in the actual data. Then the question is that “how accurately does this method work for larger than 1-minute intervals?”

In this section, queue length and delay are estimated for larger-than-one-minute intervals provided that arrival rate within each interval is already given (estimated) based on the field data. The estimated queue length is compared with the queue length coming from the field notes while the estimated delay is compared with the delay obtained from the minute-by-minute field data. The estimation is done for three-minute and five-minute intervals so that we see how the results change as the selected interval length gets longer. The details of the calculation for three-minute intervals are explained in the next section. The same procedure is used for five-minute intervals.

### 8.6.1 Validation Procedure for 3-minute intervals

For each data set, queuing duration was divided into three-minute intervals. Since the entire queuing duration may not be a multiple of three, the last interval may be less than three minutes. As already mentioned, actual departing volume, estimated arriving volume, and average speed in queue for each interval are the major input data for the purpose of validation. Number of vehicles in queue at the end of the interval was estimated by Equation 8.3

Multiplying the number of vehicles in queue by the spacing between vehicles in queue, yields the queue length at the end of the interval. Spacing between vehicles in queue is estimated by the average speed of vehicles in queue divided by the queue departure rate. Since some of the intervals are partially saturated, especially for the data sets for intermittent queue, actual departure rate and average speed of all vehicles in such an interval are not good indicators of queue departure rate and average speed of vehicles in queue. Instead, in-platoon vehicles were detected in each interval (In-platoon vehicle detection criteria were introduced in the Section 4.3.4) so that their average speed is taken as the average speed of the vehicles in queue while the reciprocal of their average headway is taken as the queue departure rate for spacing calculations.

Total moving delay for such an interval is estimated using Equation 8.9. The same procedure was followed for five-minute intervals.

### 8.6.2 Results and Conclusions

The total delay for the different interval lengths and the corresponding error is shown in Table 8.4. The results for average queue length are given in Table 8.5. When the selected interval length is three minutes, the percentage of error in total delay estimation ranges from 1% to 10% whereas the percentage of error in average queue length is between 2% to 7% When five-minute intervals are used, the percentage of error in total delay varies between 6% to 34% while the percent error for average queue length falls between 0 to 26%. Generally, as the selected interval length gets longer, the percentage of error in both MOEs increases. For the site with moving queue, I39NB, the increase in error was 1% for both average queue length and total delay. For the sites with intermittent queue (the rest of the sites), the increase in error was 0.3% to 27% for total delay and 1% to 21% for average queue length. These results convey this hypothesis that using long intervals in delay and queue length estimation can cause higher error for the sites with intermittent queue than for the site with moving queue (I39NB). However, this statement is based on just one site with moving queue. In order to further evaluate this hypothesis, field data from more sites with moving queue are needed.

Similar to all models, the accuracy of the outputs obtained from the proposed model depends on the accuracy of the input data. In order to use this method, analysts should use accurate input data to get a desirable level of accuracy.

Table 8-4.The Estimated Total Delay for Different Interval Lengths

| Data set | Minute-by-minute Estimation | 3-minute Interval | | 5-minute Interval | |
|---|---|---|---|---|---|
| | Total Delay (hr) | Total Delay (hr) | % error | Total Delay (hr) | % error |
| I39NB | 7.52 | 6.74 | 10 | 6.70 | 11 |
| I80WBAM | 0.54 | 0.57 | 6 | 0.57 | 6 |
| I80WBPM | 1.93 | 1.96 | 1 | 1.57 | 19 |
| I80EBAM | 0.30 | 0.28 | 7 | 0.20 | 34 |
| I80EBPM | 0.19 | 0.19 | 2 | 0.15 | 19 |

Table 8-5.The Estimated Average Queue Length for the Different Interval Lengths

| Data set | Minute-by-minute estimation | 3-minute intervals | | 5-minute intervals | |
|---|---|---|---|---|---|
| | Average Queue Length (ft) | Average Queue Length (ft) | % error | Average Queue Length (ft) | % error |
| I39NB | 2647 | 2456 | 7 | 2443 | 8 |
| I80WBAM | 591 | 574 | 3 | 589 | 0.3 |
| I80WBPM | 1244 | 1180 | 5 | 923 | 26 |
| I80EBAM | 535 | 525 | 2 | 403 | 25 |
| I80EBPM | 585 | 544 | 7 | 466 | 20 |

## 8.7 USER'S COST MODEL

To compute user's cost in a work zone, the delay should be multiplied by the dollar value of time for each driver/vehicle unit. The dollar value of time is different for cars and trucks, so Equation 8.21 is suggested for users' cost calculation to take into account the percentage of heavy vehicles as well as the average occupancy rate of passenger cars.

$$UC_i = (D_{ti})*(P_{SUTi}*C_{SUT} + P_{MUTi}*C_{MUT} + P_{Ci}*C_C*N_{OCC}) \quad (8.21)$$

$UC_i$ = Total users' cost during the $i^{th}$ hour (\$)
$D_{ti}$ = Total delay during the $i^{th}$ hour (hours)
$C_{SUT}$=Hourly delay cost for single unit trucks (\$/hr). The default value is \$70/hr in 2009.
$C_{MUT}$=Hourly delay cost for multiple unit trucks (\$/hr). The default value is \$90/hr in 2009.
$C_C$=Hourly delay cost for each passenger in a car (\$/hr/passenger). Default value is \$20/hr in 2009.
$N_{OCC}$=Average rate of occupancy (passengers/car). Default value is 1.25 passengers/car in 2009.
$P_{SUTi}$=Percentage of single unit trucks during the $i^{th}$ hour. (Entered as decimal)
$P_{MUTi}$=Percentage of multiple unit trucks during the $i^{th}$ hour. (Entered as decimal)
$P_{Ci}$=Percentage of passenger cars during the $i^{th}$ hour. (Entered as decimal) which is found from:

$$P_{Ci} = 1 - P_{MUTi} - P_{SUTi} \quad (8.22)$$

# CHAPTER 9 COMPARISON OF FIELD DATA TO QUICKZONE 2

Quickzone2 is commonly used to analyze work zones. In this chapter, input and output data are introduced, and then the algorithm which QuickZone2 uses to estimate delay is explained. Finally, QuickZone2 is used to analyze the data collection sites and QuickZone2 outputs are compared with the field data.

## 9.1 INPUT DATA

To estimate delay, geometric characteristics of the roadway before and after the construction, project plan, and traffic data are needed. All of this information is explained in the following sections.

### 9.1.1 Geometric Information

QuickZone2 models a network as a set of links and nodes. For each link, the length and the number of open lanes should be specified. Users should also determine whether a link is a mainline link or a work zone link. A mainline link is a link to which no changes have been made during the construction activity.

### 9.1.2 Project Plan

Beginning and end date of the project should be specified. Each project could have several phases. For each phase also, the beginning and end time and date should be defined.

### 9.1.3 Traffic Data

Demand and capacity are the major input data for delay calculation. However, there are other traffic data which are used in QuickZone2's algorithm. This section mainly concentrates on demand and capacity data which are the key variables in delay estimation.

#### *9.1.3.1 Demand*

Demand can be specified either by AADT of the roadway or hourly counts. PCE value and percentage of heavy vehicles are also needed. According to the QuickZone's user manual, when truck percentage is already included in volume, users should leave the PCE-value's field blank in demand module. It should be noted that the effect of the percentage of heavy vehicles on capacity is considered later.

#### *9.1.3.2 Capacity*

Pre-construction capacity and capacity of each work zone link are needed. Pre-construction capacity is a user-specified feature. Capacity of a work zone link can be determined either by user or with respect to the models available in the QuickZone2 for short term work zones. The available models are the HCM 1997 model, the HCM 2000 model and the model proposed by the University of Maryland (UMD model).

When a user chooses the HCM 1997 model, then suggested capacity values for several work zone configurations are displayed and the user selects one of them.

Equation 9.1 indicates the HCM 2000 model for short term work zones.

$$C = (1600+I-R)*f_{HV}*N \qquad (9.1)$$

Where
C = Estimated capacity of a short term work zone (veh/h),
$f_{HV}$ = Heavy vehicle adjustment factor,
N = Number of lanes open through the work zone,
I = Adjustment factor for type, intensity, and location of the work activity (ranges from -160 to +160 pcphpl), and
R= Adjustment for ramps

UMD proposed a model for work zone capacity which is based on Equation 9.2.

$$CAPACITY=1857 - 168.1NUMCL - 37.0LOCCL - 9.0HV + 92.7LD - 34.3WL - 106.1WI_H - 2.3WG*HV \qquad (9.2)$$

Where:
CAPACITY = Capacity of work zone in vphpl,
NUMCL = Number of lane closures,
LOCCL = Dummy variable for location of lane closures (in the case of right-side lane-closure = 1, otherwise=0),
HV = Proportion of heavy vehicles,
LD = Lateral distance to the open travel lane,
WL =Work zone length,
$WI_H$ =Dummy variable for heavy work activity (heavy work activity=1, otherwise=0), and
WG*HV = Work zone grade*proportion of heavy vehicles.

It should be noted that this model was developed by using of data collected from short term work zones.

As already mentioned in Benekohal et al. (2003), there are some concerns about the UMD model. This model was based on the study conducted by Kim et al (2001). The results of the statistical tests are given in Table 9.1 for different parameters of the model. Based on the P-values reported, the parameters of the location of closed lanes, proportion of heavy vehicles, lateral distance to the open lanes, and work zone length do not have significant effect on the work zone capacity at 90 percent confidence level. In the meantime, it is known that these are some important factors which normally affect the traffic flow. The effect of heavy vehicles and lateral distance of work activity to the travel lane on capacity is a known issue. Since there are some concerns about the validity of the UMD model, this model is not used in QuickZone2 evaluation.

Table 9-1. Statistical Analyses for the Parameters of UMD Model

| **Factor** | **Variable** | **P-value** |
|---|---|---|
| | CONSTANT | 1.65 E-05 |
| Number of closed lanes | NUMCL | 0.011 |
| Location of closed lanes | LOCCL | 0.199 |
| Proportion of heavy vehicles | HV | 0.212 |
| Lateral distance to the open lanes | LD | 0.125 |
| Work zone length | WL | 0.166 |
| Intensity of work activity | WI | 0.054 |
| Work zone grade*proportion of heavy vehicles | WG*HV | 0.028 |

It should be noted that the models introduced so far were developed based on short term work zone data. It is up to users to employ these models for long term work zones. However, for long term work zones, HCM 2000 proposed capacity values reported in Table 9.2.

Table 9-2. Capacity Values Proposed by HCM 2000 for Long Term Work Zones

| No. of Normal Lanes | Lanes Open | Number of Studied | Range of Values (vphpl) | Average per lane (vphpl) |
|---|---|---|---|---|
| 3 | 2 | 7 | 1780-2060 | 1860 |
| 2 | 1 | 3 | - | 1550 |

*9.1.3.3 Other Traffic Data*

Other than demand and capacity, there are some other data that should be specified by the user. Free flow speed and the jam density of each link should be determined. Travel behavior data such as percentage of demand that cancels the trip or that chooses another mode is also needed.

## 9.2 OUTPUT DATA

QuickZone2 returns several measures of effectiveness such as delay, user cost, and queue length. User cost has four components, namely user delay cost, vehicle operating cost, inventory cost, and economic cost. User delay cost is calculated based on the results of delay calculation. The algorithm for delay estimation is elaborated in the next section. Vehicle operating costs are estimated separately for cars and trucks. Inventory costs are computed for freight vehicles and economic costs are considered due to the effects of traffic flow reduction on the business of that area.

In addition, travel behavior information is also determined by QuickZone2. In travel behavior information, the proportion of traffic which selects each of these strategies is estimated: trip cancelation, mode changing, travel time shifting, and taking a detour.

## 9.3 QUICKZONE2 ALGORITHM FOR DELAY ESTIMATION

QuickZone2 uses input-output analysis to estimate the number of vehicles in queue at the end of each hour. For each hour when demand is greater than capacity, the difference between volume and capacity is determined as the number of vehicles in the queue. For delay calculation, Quickzone2 assumes that vehicles are stopped within the queue. Consequently, the average length of the queue at the end and beginning of each hour is taken as the total delay (hr) for that hour.

Two major input parameters for delay estimation are demand and capacity which were explained in the previous sections. However, Quickzone2 can model the effect of lane closure on the travel demand. If a user selects this feature, then the demand which was previously input by the user might be updated and the updated demand is used in the delay calculation.

The difference between the number of vehicles in queue before construction versus during construction is considered as the unsatisfied demand. Quickzone2 assumes that the unsatisfied drivers select at least one of these strategies: 1) shifting the trip time 2) choosing detour 3) changing modes 4) trip cancellation.

The proportion of drivers that selects the first strategy is determined by the user. If publicity is provided, the total unsatisfied demand will be evenly distributed to an interval which begins from two hours before the oversaturated duration and ends one hour after the oversaturated duration. If publicity is not provided, the unsatisfied demand of each queuing hour will be evenly assigned to one hour before and after that queuing hour. After this step, demand is updated and unsatisfied demand is calculated again.

The proportion of unsatisfied drivers who select the detour(s) is determined by the software. If VMS is available in advance of the detour, the software will assign the minimum of unsatisfied demand and spare capacity of the detour to the detour. If VMS is not available, Quickzone2 will assign the unsatisfied demand to the detour when the back of the queue in the mainline reaches the access point of the detour. However, the assigned volume is not greater than 90 percent of the spare capacity of the detour. After this step, the demand is updated and the unsatisfied demand is recomputed.

The proportion of demand that chooses another mode or that cancels the trip is determined by the user. The software removes this proportion from the total demand.

Finally, the software computes total delay (veh-hour) for each hour as explained previously.

## 9.4 QUICKZONE2 EVALUATION USING FIELD DATA

In the next sections, the delay and queue length estimated by QuickZone2 are compared with the field data for sites with queuing condition and non-queuing condition.

### 9.4.1 Sites with Queuing Condition

Hourly volume data shown in Table 9.3 for each data set were used as the demand data. Three different capacity values were used for each data set: 1) HCM 2000 capacity 2) Suggested capacity based on the field data for a site with flagger and queue 3) Actual departure rate which is equal to the volume for these data sets.

Table 9-3. QuickZone Input Data for the Sites with Queuing Condition

| Site | Traffic Condition | Hourly Volume (vph) | HCM 2000 Capacity (vph) | Suggested Capacity (vph) | Actual Departure Rate (vph) |
|---|---|---|---|---|---|
| I39NB | AM, Flagger & queue | 778 | 1261 | 1051 | 778 |
| I80EB | AM, Flagger & queue | 610 | 1183 | 986 | 610 |
| | PM, Flagger & queue | 699 | 1244 | 1037 | 699 |
| I80WB | AM, Flagger & queue | 580 | 1205 | 1005 | 580 |
| | PM, Flagger & queue | 641 | 1187 | 989 | 641 |

For all these three capacity options, QuickZone2 returned no delay and no queue length for each data set while each of these data sets had queue in reality. The delay and queue length based on the field data are shown in Table 9.4 for these data sets. All these data sets had less-than-an-hour queuing conditions which occurred in the middle of the hour. In other words, congestion begins at some point within the hour and ends before the end of that hour. The results convey that QuickZone2 is not able to detect queuing condition for these data sets. For further discussion on the QuickZone output for the sites with queuing condition, the reader may refer to Appendix C, in which queues were "created" in QuickZone by changing input data, however the authors do not recommend using this approach.

Table 9-4.Moving Delay and Queue Length Based on the Field Data

| Data Sets | Average Queue Length (ft) | Total Delay (hr) |
|---|---|---|
| I39NB | 2646.64 | 7.52 |
| I80WBAM | 590.96 | 0.54 |
| I80WBPM | 1244.31 | 1.93 |
| I80EBAM | 535.20 | 0.30 |
| I80EBPM | 584.61 | 0.19 |

**9.4.2 Sites without Queuing Condition**

Similar to the data sets with queuing condition, three different capacity values were used to analyze the data set with under-saturated condition. The three capacity values are based on 1) the suggested capacity values in Chapter 5 for these data sets 2) the HCM 2000 model 3) the actual departure rate. Table 9.5 shows the hourly demand, percentage of heavy vehicles and the capacity values which were used as the input data.

Quickzone2 returned no delay and queue for these data sets. Table 9.6 shows the total delay for sites without queuing conditions. The total delay for sites with speed limit of 45 mph and without treatment ranges from 0 to 2.9hr. Only the delay for I39NB without any treatment, (2.9 hr), is somewhat far from zero. For the rest of the data sets, the estimated total delay is close to zero as returned by QuickZone2.

For the data set with police, the total delay of 0.1 (hr) is close to zero. It should be noted that the location of the camera for this site was close to the work space and a little far from the police location. The work activity did not interfere with the traffic during the time of data collection. Since the police location was a little far from the camera, the effect of the police on speed may not be significant in this data set. Thus, the delay is not high. The estimated delay for the site with flagger is 4.8 hr while QuickZone2 returned zero for this site. Clearly, QuickZone2 underestimated the total delay for the site with flagger.

Table 9-5. QuickZone Traffic Input Data for each Data Set with Non-queuing Condition

| Site | Traffic Condition | Hourly Volume (vph) | Capacity (vph) Based on | | |
|---|---|---|---|---|---|
| | | | Suggested value | HCM 2000 Model | Actual Departure Rate |
| I39NB | PM, very low work activity, with flagger and no queue, with dynamic feedback sign | 920 | 1279 | 1461 | 920 |
| | PM, no work activity, no queue, and no flagger, with dynamic feedback sign | 860 | 1535 | 1444 | 860 |
| I80EB | No work activity, no queue, no treatment | 647 | 1579 | 1444 | 647 |
| I80WB | No work activity, no queue, no treatment | 973 | 1535 | 1404 | 973 |
| I72EB | Morning peak, no work activity, no queue, police | 876 | 1263 | 1304 | 876 |
| | Noon Peak, no work activity, no queue, no treatment | 828 | 1303 | 1345 | 828 |
| I74EB | AM, no work activity, no queue, no treatment | 732 | 1368 | 1509 | 732 |
| | PM, no work activity, no queue, no treatment, intermittently rainy | 800 | 1446 | 1493 | 800 |

## 9.6 CONCLUSION

The total delay and queue length, returned by QuickZone2 were compared with the corresponding values based on the field data. Traffic data for one hour was input to the QuickZone and three different capacity values were used:1) The suggested capacity values based on the field data 2)The HCM 2000 model 3) The actual departure rate.

Regardless of which capacity was used, QuickZone2 returned zero delay and queue length for all five data sets that actually had queue. All these five data sets had less-than-an-hour queuing condition and queuing condition ended before the end of the study interval.

For the eight data sets without queue (undersaturated conditions), QuickZone2 returned no delay and queue length while in six of these data sets, the average speed of traffic was below the speed limit and consequently, the drivers experienced some delay.

Table 9-6. Total Delay Computed Based on the Field Data for Non-queuing Data Sets

| Site | Data Set | SL* (mph) | Average Speed of Traffic (mph) | Length of the Work Space (mile) | Counted Volume (vehicle) | Total Delay (hr) |
|---|---|---|---|---|---|---|
| I39NB | PM, very low work activity, with flagger and no queue, with dynamic feedback sign | 45 | 37.98 | 1.5 | 690 | 4.8 |
| | PM, no work activity, no queue, and no flagger, with dynamic feedback sign | 55 | 48.10 | 1.5 | 645 | 2.9 |
| I80EB | No work activity, no queue, no treatment | 45 | 47.95 | 3.6 | 485 | 0.3 |
| I80WB | No work activity, no queue, no treatment | 45 | 45.33 | 1.5 | 730 | 1.7 |
| I72EB | Morning peak, no work activity, no queue, police | 45 | 43.59 | 0.1 | 657 | 0.1 |
| | Noon Peak, no work activity, no queue, no treatment | 45 | 44.67 | 0.1 | 621 | 0.0 |
| I74EB | AM, no work activity, no queue, no treatment | 55 | 54.20 | 0.1 | 549 | 0.0 |
| | PM, no work activity, no queue, no treatment, intermittently rainy | 55 | 53.64 | 0.1 | 600 | 0.1 |

# CHAPTER 10 PROCEDURE TO ESTIMATE CAPACITY AND DELAY IN WORK ZONES

In this chapter, a step-by-step procedure is presented to estimate capacity and delay in work zones. This chapter basically presents the practical results of Chapters 6, 7, and 8. Finally, an example problem is solved to show how the procedure is applied for the purpose of analyses.

## 10.1 STEP BY STEP ALGORITHM TO ESTIMATE CAPACITY AND DELAY IN WORK ZONES

1. Find the speed reductions due to less-than-ideal lane width ($R_{LW}$) and lateral clearance.($R_{LC}$) from Table 10.1.

Table 10-1. Adjustment due to Lane Width and Lateral Clearance

| Adjustment for lane width | | | | |
|---|---|---|---|---|
| **Lane width (ft)** | **Reduction in speed (mph)** | | | |
| **12 ft or more** | 0.0 | | | |
| **11** | 1.9 | | | |
| **10** | 6.6 | | | |
| **9** | 15.0* | | | |
| **8** | 25.0* | | | |
| **Adjustment for left shoulder** | | | | |
| **Left shoulder (ft) width** | **Reduction in speed (mph)** | | | |
| **2 ft or more** | 0 | | | |
| **1** | 1 | | | |
| **0** | 2 | | | |
| **Adjustment for right shoulder** | | | | |
| **Right shoulder width (ft)** | **Reduction in speed (mph)** | | | |
| | **No of Lanes in one direction (without work zone)** | | | |
| | **2** | **3** | **4** | **>= 5** |
| **6 ft or more** | 0 | 0.0 | 0.0 | 0.0 |
| **5** | 0.6 | 0.4 | 0.2 | 0.1 |
| **4** | 1.2 | 0.8 | 0.4 | 0.2 |
| **3** | 1.8 | 1.2 | 0.6 | 0.3 |
| **2** | 2.4 | 1.6 | 0.8 | 0.4 |
| **1** | 3.6 | 2.0 | 1.0 | 0.5 |
| **0** | 3.9 | 2.4 | 1.2 | 0.6 |

**: Based on the authors' best estimate*

2. Determine the level of work intensity for short-term work zones by Table 10.2 and for long-term work zones by Table 10.3.

Table 10-2. Work Intensity Table for Short-Term Work Zones

| Lateral distance between the work activity area and the edge of the open lane (ft) | (# of workers) + (# of large construction equipment) in the work activity area | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **12** | **13** | **14** | **15** |
| **9** | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **8** | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **7** | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **6** | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **5** | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI |
| **4** | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI |
| **3** | LO | LO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI |
| **2** | LO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |
| **1** | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |

LO=Low work intensity MO=Moderate work intensity HI=High work intensity

Table 10-3. Work Intensity Table for Long-Term Work Zones

| Lateral distance between the work activity area and the edge of the concrete barrier closer to the activity area (ft) | (# of workers) + (# of large construction equipment) in the work activity area | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **12** | **13** | **14** | **15** |
| **9** | LO | LO | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO |
| **8** | LO | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO |
| **7** | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **6** | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **5** | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| **4** | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI |
| **3** | LO | LO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI |
| **2** | LO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI |
| **1** | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |

LO=Low work intensity MO=Moderate work intensity HI=High work intensity

3- Find the speed reduction corresponding to the work intensity determined in step 2. Use Table 10.4 for short-term work zones and Table 10.5 for long-term work zones.

Table 10-4. Speed Reduction due to Work Intensity in Short Term Work Zones

| **Work Intensity** | **Estimated Speed Reduction Range (mph)** | **Suggested Speed Reduction (mph)** |
|---|---|---|
| LOW | 6.04 - 10.80 | 8.00 |
| MODERATE | 10.81 - 14.40 | 12.00 |
| HIGH | 14.41 - 19.16 | 16.00 |

Table 10-5. Speed Reduction due to Work Intensity in Long-Term Work Zones

| **Work Intensity** | **Estimated Speed Reduction Range (mph)** | **Suggested Speed Reduction (mph)** |
|---|---|---|
| LOW | 0.01 - 2.65 | 2 |
| MODERATE | 2.66 - 3.80 | 3 |
| HIGH | 3.81 - 5.93 | 5 |

4- Find the speed reduction (mph) due to any treatment by using Table 10.6.

Table 10-6. Speed Reduction Reported in the Literature

| Speed Control Technique | Observed Range of Speed Reduction (mph) | Typical Speed Reduction (mph) |
|---|---|---|
| Changeable Message Signs[1] | 1.4 - 4.7 | 3.0 |
| Drone Radar[2] | 1.2 - 9.8 | 2.5 |
| Police Presence[3] | 4.3 - 5.0 | 4.5 |
| Speed Photo Enforcement[3] | 3.4 - 7.8 | 5.0 |
| MUTCD Flagging[4] | 3.0 - 12.0* | 7.0* |
| Innovative Flagging[4,**] | 4.0 - 16.0* | 10.0* |
| Changeable Message Signs with Radar[5] | 4.0 - 8.0 | 5.0 |
| Speed Monitoring Display[5] | 4.0 - 5.0 | 4.0 |

**: In the case of flagger, the authors recommend the use of the speed-flow curves for a speed limit of 45 mph and flagger in the work zone rather than the suggested speed reduction shown in the table.*

***: Innovative flagging is the combination of the flagging procedure described in MUTCD with the flagger using the other hand to motion the traffic to slow, and then to point to an adjacent speed limit sign (Garber and Srinivasan, 1998).*

1 Benekohal et al (1992)
2 Benekohal et al (1993)
3 Benekohal et al (2008)
4 Richards et al (1985)
5 Mattox et al (2007)

5-Compute the adjusted free flow speed by Equation 10.1:

$$AFFS = FFS - R_{WI} - R_{LW} - R_{LC} - R_T - R_o \quad (10.1)$$

Where
AFFS= Adjusted free flow speed (mph)
FFS =Free-flow speed (when there are no field data, FFS=62 mph for speed limit of 55 mph, FFS=55 mph for a site with speed limit of 45 mph and without flagger, and FFS=43 mph for a site with speed limit of 45 mph and presence of flagger)
$R_{LW}$ = Speed reduction due to lane width (mph),
$R_{LC}$ =Speed reduction due to lateral clearance (mph),
$R_{WI}$ = Speed reduction due to work intensity (mph),
$R_T$= Speed reduction due to treatment (mph),
$R_o$ = Speed reduction due to all other factors that may reduce speed (mph) (including those that may cause a flow breakdown).

6-Find the speed-flow curve corresponding to the adjusted free flow speed (see Appendix A and B or use the Excel worksheet provided) and read the maximum flow rate ($C_{Max}$) from the speed-flow curve. Use Figure 10.1 for a site with speed limit of 45mph and flagger, Figure 10.2 for speed limit of 45 mph without a flagger, Figure 10.3 for speed limit of 55 mph and flagger.

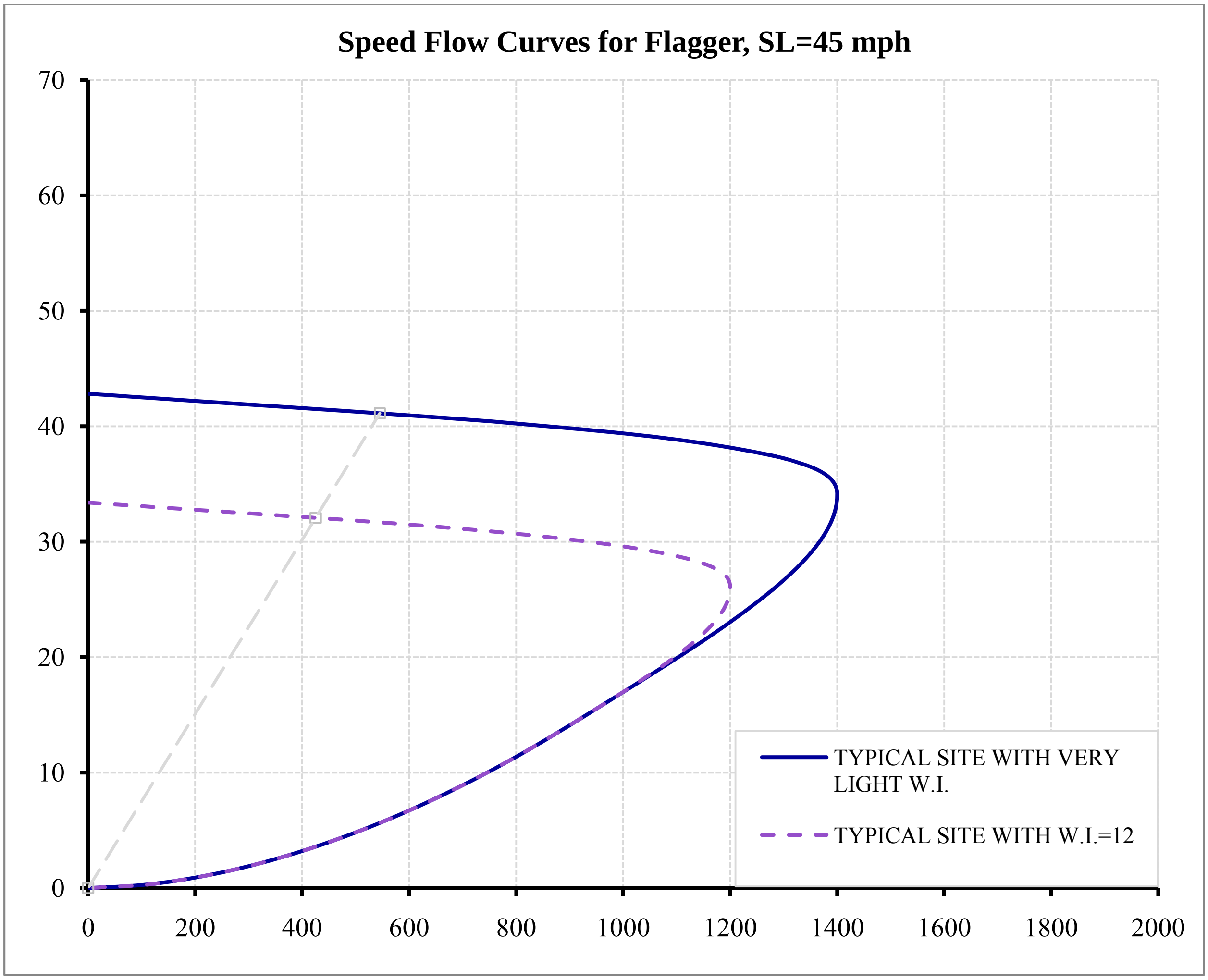


Figure 10-1. Speed-flow curve for a site with speed limit of 45 mph and flagger.

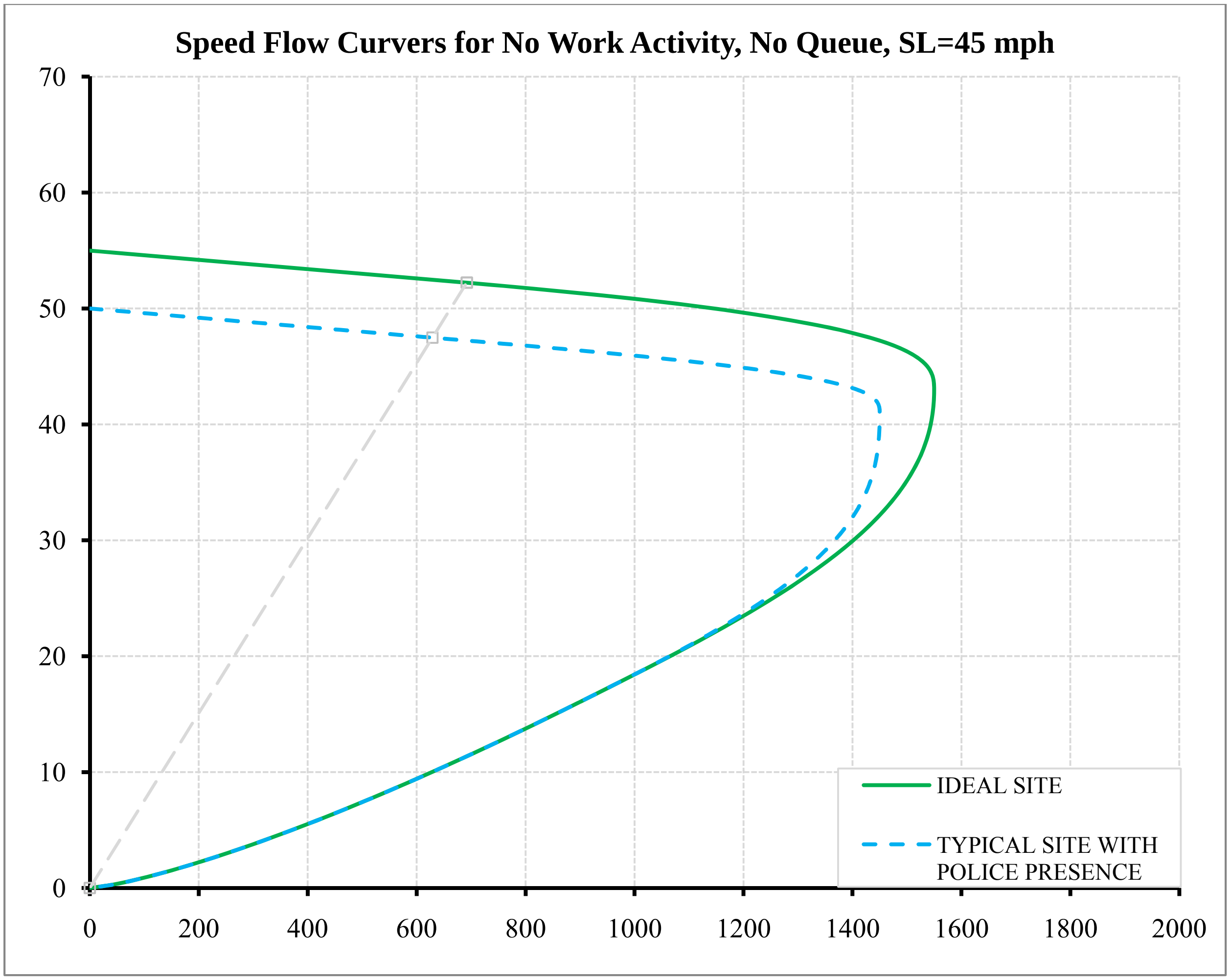


Figure 10-2. Speed-flow curve for a site with speed limit of 45 mph and no flagger.

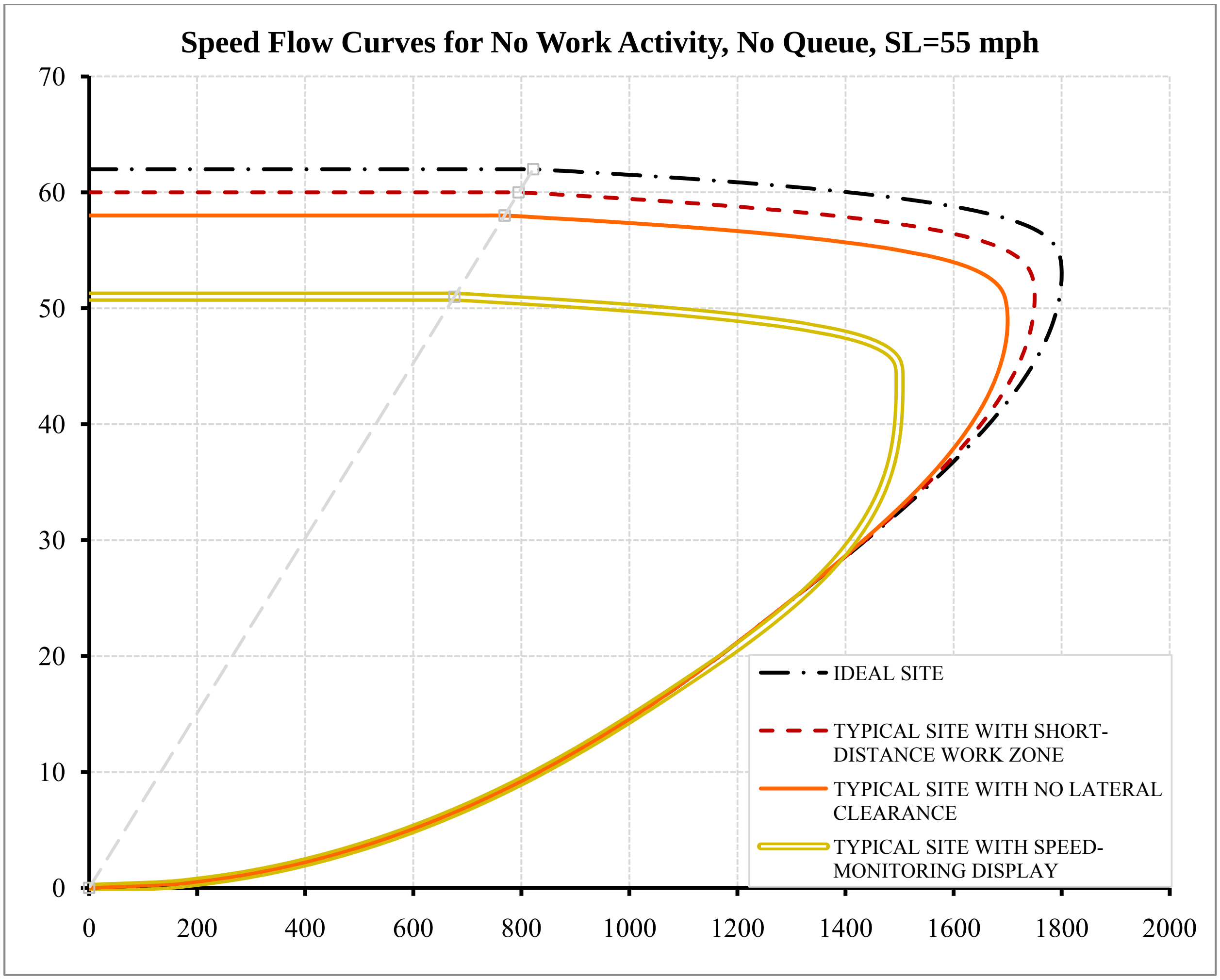


Figure 10-3. Speed-flow curve for a site with speed limit of 55 mph.

7- Find the capacity corresponding to the traffic condition in the work zone by using Equation 10.2:

$$\text{Capacity} = \begin{cases} C_{Max} & \text{if traffic is under normal condition} \\ \dfrac{(60 - t_1) * C_{Max}}{60}, & \text{When traffic is asked to stop} \end{cases} \tag{10.2}$$

Where
Capacity = Capacity (pcphpl) corresponding to the traffic condition in the work zone
$C_{Max}$= Maximum flow rate (pcphpl).
$t_1$ = Interval during which traffic is asked to stop (min)

8- Compute heavy vehicle adjustment factor as below:

$$f_{HV} = \frac{1}{1 + P_T(PCE - 1)} \tag{10.3}$$

Where
$f_{HV}$= Heavy vehicle adjustment factor
$P_T$ = Total percentage of trucks. (Entered as decimal)
PCE=Passenger car equivalence factor determined by using Table 10.7 when no grade is long enough or steep enough to cause a significant speed reduction on trucks (when no one grade of 3% or greater is longer than 0.25 mile or where no one grade of less than 3% is longer than 0.5 mile). Otherwise, PCE should be obtained from Exhibit 23-9 of the Highway Capacity Manual.

Table 10-7. Passenger Car Equivalence

| Passenger Car Equivalence | Type of Terrain | | |
|---|---|---|---|
| | Level | Rolling | Mountainous |
| Trucks and Buses | 1.5 | 2.5 | 4.5 |

9- Calculate the adjusted capacity:

$$C_{adj} = \text{Capacity} * f_{HV} \tag{10.4}$$

Where
$C_{adj}$ = Adjusted capacity (vphpl),
Capacity = Capacity corresponding to the traffic condition in the work zone (pcphpl),
$f_{HV}$= Heavy vehicle adjustment factor.

10-Find the traffic demand as follow:

$$D_i = n_{i-1} + V_i \tag{10.5}$$

Where
$D_i$=Traffic demand for the $i^{th}$ hour (vph)
$n_{i-1}$=The number of vehicles in queue at the end of the $(i-1)^{th}$ hour (vph)
$V_i$=Volume arriving during the $i^{th}$ hour (vph)

Determine the operating speed ($U_o$). If demand ($D_i$) is greater than the departure rate ($C_{adj}$*$N_{op}$), then read the operating speed at the maximum flow rate from the speed-flow curve of the site. Otherwise, read the speed from the corresponding speed-flow curve by entering the passenger-car equivalent hourly volume obtained by the Equation 10.6, on the horizontal axis.

$$V_{PCE} = \frac{Vi}{f_{HV} * N_{op}} \quad (10.6)$$

Where
$V_{PCE}$ = Passenger-car equivalent volume (pcphpl),
$V_i$ = Volume arriving during the $i^{th}$ hour (vph),
$f_{HV}$= The heavy vehicle adjustment factor.
$N_{op}$=Number of open lanes in the activity area,

11- If arriving volume ($V_i$) is less than or equal to the departure rate ($C_{adj}$ * $N_{op}$) and the number of the vehicles in queue at the end of the previous interval is zero ($n_{i-1}$=0), then $d_{qi}$=0, $n_i$=0, $n_{ci}$=0 and skip steps 11 and 12. Otherwise, estimate the number of vehicles in queue at the end of the hour as follows:

$$n_i = \max(0,\ D_i - C_{adj} * Nop) \quad (10.7)$$

Where
$n_i$ = Number of vehicles in queue at the end of the $i^{th}$ hour
$n_{i-1}$ =Number of vehicles in queue at the end of the $(i-1)^{th}$ hour
$D_i$ = Total demand in the $(i)^{th}$ hour(vph)
$C_{adj}$= Adjusted capacity of the work zone in vphpl
$N_{op}$= Number of open lanes in the activity area
$\bar{d}_{qi}$=Average queuing delay during the $i^{th}$ hour
$\acute{n}_i$=Number of queued vehicles on the closed lane at the end of the hour

If $n_i$>0, Compute I (average spacing between vehicles) as below:

$$I = \frac{U_q}{C_{adj}} * 5280 \quad (10.8)$$

I= Average spacing between vehicles (ft),
$P_T$=Percentage of heavy vehicles,
$U_{qO}$= Average speed (mph) of the traffic in queue. This speed is the speed at the capacity and is read from the corresponding speed-flow curve. When the interval is completely oversaturated,$U_q$is equal to Uo. Conversely, when the interval is completely in under-saturated condition or partially oversaturated, then $U_q$is notequal to Uo.
$C_{adj}$ = Adjusted capacity of the work zone (vphpl)

Calculate the stacked queue length at the end of the $i^{th}$ hour($Q_{Si}$):

$$Q_{Si} = n_i * I/5280 \quad (10.9)$$

Where
$Q_{Si}$= Stacked queue length (mile)
$n_i$ = Number of vehicles in queue at the end of the $i^{th}$ interval.
I = Average spacing between vehicles (ft)

Determine the distance (See Figure 10-4) from the beginning of the transition taper to the end of the activity area (C) in mile.

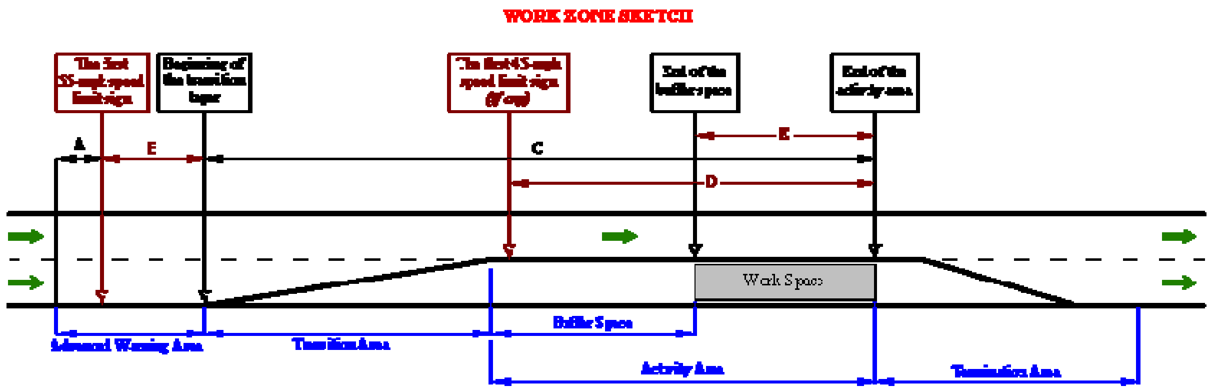


Figure 10-4. Work zone sketch.

If $C \geq Q_{si}/N_{op}$, the queue does not extend upstream of the transition area, and the number of queued vehicles on the closed lane is zero ($\acute{n}_i=0$).Also, the queue length (mile) at the end of this hour, $Q_i$, is computed by Equation 10.10.

$$Q_i = Q_{si}/N_{op} \tag{10.10}$$

Where
$Q_i$ = Queue length at the end of the $i^{th}$ hour (mile)
$Q_{si}$ = Stacked queue length at the end of the $i^{th}$ hour (mile)
$N_{op}$= Number of open lanes in the activity area,

If $C < Q_{si}/N_{op}$, the queue will extend upstream of the beginning of the transition area. The queue length (mile) at the end of this hour, $Q_i$, is found as below:

$$Q_i = C+(Q_{si} - C*N_{op})/N_{nr} \tag{10.11}$$

Where
$Q_i$ = Queue length at the end of the $i^{th}$ hour (mile),
C= The distance from the beginning of the transition taper to the end of the buffer space (mile),
$Q_{si}$ = Stacked queue length at the end of the $i^{th}$ hour (mile),
$N_{op}$= Number of open lanes in the activity area,
$N_{nr}$= Number of open lanes upstream of the beginning of the transition area.
Also, the number of queued vehicles on the closed lane is estimated by Equation 10.12:

$$n_{ci} = \frac{Q_i - C}{l} * 5280 \tag{10.12}$$

Where
$n_{ci}$ =The number of queued vehicles on the closed lane at the end of the $i^{th}$ interval,
$Q_i$ = Queue length at the end of the $i^{th}$ interval (mile),

C= The distance from the beginning of the transition taper to the end of the buffer space (mile),
I=The spacing between the vehicles in queue (ft).

Determine the average queue length during the i$^{th}$ interval (   ) by using Equation 10.13:

(10.13)

Where
= Average queue length during the i$^{th}$ interval (mile)
$Q_i$ = Queue length at the end of the i$^{th}$ interval (mile)
$Q_{i-1}$ = Queue length at the end of the (i-1)$^{th}$ interval(mile).
Determine the average number of queued vehicles on the closed lane:

(10.14)

=The average number of vehicles in queue and on the closed lane during the i$^{th}$ interval,
The number of queued vehicles on the closed lane at the end of the i$^{th}$ interval,
The number of queued vehicles on the closed lane at the end of the (i-1)$^{th}$ interval.

12- Determine the distances B, C, D, shown in Figure 10.4 and defined as below:
B = The distance from the first 55-mphspeed limit sign to the beginning of the transition taper (mile)
C=The distance from the beginning of the taper to the end of the activity area (mile)
D=The distance from the 45-mph speed limit sign to the end of the activity area (mile)

Estimate the delay in moving queue as below:
If the speed limit in the vicinity of the work space is 45 mph, then

(10.15)

If the speed limit in the vicinity of the work space is 55 mph, then

(10.16)

Where
=Average delay per vehicle during the i$^{th}$ hour due to queuing (hour/vehicle)
$\bar{Q}_i$ = Average queue length during the i$^{th}$ hour (mile)
= Average speed of the traffic in queue (mph).
=Average number of vehicles (obtained by Equation 10.14) in queue and on the closed lane

$\overline{\gamma}$=The additional time required for a vehicle to move from the closed lane to the open lane in the same queue position. Whenever $\acute{n}$ is zero, $\overline{\gamma}$=0.Otherwise, it is suggested to use $\frac{1}{C_{adj}}$for $\overline{\gamma}$ where $C_{adj}$ is the adjusted capacity of the work zone in vphpl.

13) If demand is more than departure rate ($D_i > C_{adj} * N_{op}$),$d_{ui}$=0, and skip step 13. Otherwise, estimate the average delay for each vehicle due to an operating speed less than the speed limit in the work zone:

$$d_{ui} = \begin{Bmatrix} \left(\frac{E}{U_o} - \frac{E}{SL}\right) & \text{if } U_o < SL \\ 0 & \text{Otherwise} \end{Bmatrix} \tag{10.17}$$

$d_{ui}$= Delay per vehicles in undersaturated condition during the i$^{th}$ hour (hour/vehicle)due to an operating speed less than speed limit
E= The distance from the end of the buffer space to the end of the activity area (mile)
$U_0$=The operating speed (mph)
SL=Speed limit in the vicinity of the work space
14-Estimate proportion of the hour in oversaturated condition, β, using Equation 10.18

$$\beta = \frac{n_{i-1}}{C_{adj} * N_{OP} - V_i} \tag{10.18}$$

Where
$n_{i-1}$=The number of vehicles in queue at the end of the (i-1)th hour
$C_{adj}$=Adjusted capacity of the work zone (vphpl)
$V_i$=Arriving volume during hour I (vph)
$N_{op}$=Number of lanes open in the activity area
Estimate average delay as below:

$$d_i = \begin{Bmatrix} \beta * d_{qi} + (1-\beta) * d_{ui} & \text{If } 0 < \beta < 1 \\ d_{qi} + d_{ui} & \text{Otherwise} \end{Bmatrix} \tag{10.19}$$

Where
$d_i$=Average delay experienced by users during the i$^{th}$ hour (hour/vehicle)
$d_{qi}$=Average delay per vehicle under queuing condition during the i$^{th}$ hour (hour/vehicle)
$d_{ui}$= Delay experienced by each vehicle in undersaturated conditions during the i$^{th}$ hour (hour/vehicle)due to an operating speed less than speed limit.

Estimate the total delay during the i$^{th}$ hour as follows:

$$D_{ti} = V_i * d_i \tag{10.20}$$

Where
$D_{ti}$= Total delay during the i$^{th}$ hour (hour)
$V_i$= Arriving volume during the i$^{th}$ hour (vehicle)
$d_i$=Average delay experienced by drivers during the i$^{th}$ hour (hour/vehicle)

Compute the users' cost for the i$^{th}$ hour using Equation 10.21:

$$\mathrm{UC_i} = \mathrm{D_{ti}} * (\mathrm{P_{SUTi}} * \mathrm{C_{SUTi}} + \mathrm{P_{MUTi}} * \mathrm{C_{MUTi}} + \mathrm{PC_i} * \mathrm{C_C} * \mathrm{N_{OCC}}) \quad (10.21)$$

$\mathrm{UC_i}$ = Total users' cost during the ith hour ($)
$\mathrm{D_{ti}}$ = Total delay during the ith hour (hr)
$\mathrm{C_{SUTi}}$=Hourly delay cost for single unit trucks ($/hr). The default value is 70$/hr in 2009.
$\mathrm{C_{MUTi}}$=Hourly delay cost for multiple unit trucks ($/hr). The default value is 90$/hr in 2009.
$\mathrm{C_C}$=Hourly delay cost for each passenger in a car ($/hr/passenger). The default value is 20$/hr in 2009.
$\mathrm{N_{OCC}}$=Average rate of occupancy (passengers/car). The default value is 1.25 passengers/car in 2009.
$\mathrm{P_{SUTi}}$=Percentage of single unit trucks during the $i^{th}$ hour. (Entered as decimal)
$\mathrm{P_{MUTi}}$=Percentage of multiple unit trucks during the $i^{th}$ hour. (Entered as decimal)
$\mathrm{P_{Ci}}$=Percentage of passenger cars during the $i^{th}$ hour and obtained using Equation 10.22:

$$\mathrm{P_{Ci}} = 1 - \mathrm{P_{SUTi}} - \mathrm{P_{MUTi}} \quad (10.22)$$

15) Do Step 1 to 14 for each hour of the study and compute total users' cost for the total period of the study as follows:

$$\mathrm{UC} = \sum_{i=1}^{t} \mathrm{UC_i} \quad (10.23)$$

UC=Total users' cost over all study hours
$\mathrm{UC_i}$=Total users' cost for the $i^{th}$ interval of the study
t= Total number of the study hours

**10.2 EXAMPLE PROBLEM: A SITE WITH FLAGGER AND SPEED LIMIT OF 45 MPH**

In this section, the proposed step-by-step procedure is followed to estimate the delay and queue length for a site where all conditions but volume are the same as the work zone on I39NB. This site is analyzed for three hours. For the purpose of illustration, hourly volume is chosen such that the traffic condition of the first hour is under completely undersaturated condition; the second one is under completely oversaturated conditions whereas the third one has partially oversaturated conditions.

The following information describes the prevailing conditions during a congested interval on I39NB. At this site, one lane was closed due to construction activity over a bridge. Just one 12-foot lane was open to traffic. No left shoulder was available in the lane closure section and the right shoulder was 8-foot wide. Concrete barriers separated the activity area from the right shoulder. The speed limit within the activity area was 45 mph and a flagger almost at the beginning of the bridge was showing the slow down sign. The traffic was composed of 2% single-unit trucks and 26% multiple-unit trucks, and the site was located on the level terrain. The following geometric data are also available:

The distance from the beginning of the advance warning area to the first 55-mphspeed limit sign, A, is 0.2 mile.
The distance from the first 55 mph-speed limit sign to the beginning of the transition taper, B, is 0.8 mile.
The distance from the beginning of the taper to the end of the activity area, C, is 2.5 miles.
The distance from the 45-mph speed limit sign to the end of the activity area, D, is 1.7 miles.
The distance from the end of the buffer space to the end of the activity area, E, is 1.5 miles.

Delay and users' cost are estimated for three consecutive hours in which the conditions are described as follows:

The first hour: The hourly volume is 800 vehicles. The interval before this hour was under undersaturated conditions and no queued vehicles are left from the previous interval. The workers and equipment are far from the travel lane such that it can be practically assumed that there is no work intensity effect on traffic within this hour.

The second hour: The hourly volume is 1100 vehicles and during this hour, six workers and also three large construction equipments were present on the right shoulder4 ft away from the travel lane. As a result, practically 4 ft is available as the right shoulder for traffic. Although the site is a long-term work zone, no concrete barrier is located between the travel lane and workers during this hour. Thus, the effect of work intensity is more similar to that in a short-term work zone rather than in a long term work zone.

The third hour: The conditions are the same as those in the second hour except that the hourly volume is 600 vehicles.

### 10.2.1 Solution for the First Hour

1. Find the speed reductions due to the less-than ideal lane width ($R_{LW}$) and lateral clearance
($R_{LC}$) from Table 10.1:
Ideal lane width: 12 ft. Then:
$R_{LW}$ =0

There is no left shoulder, so the speed reduction due to the lack of left shoulder=2 mph
There is more-than-6-ft right shoulder, so the speed reduction due to right shoulder =0mph
$R_{LC}$ = 2 +0= 2mph

2- Determine the level of work intensity.
As mentioned in the description of the prevailing conditions for the first hour, there are practically no work intensity effects on the traffic stream.

3- Find the speed reduction corresponding to the work intensity determined in step 2.
Use Table 10.4 for short term work zones and Table 10.5 for long term work zones.
Since there is no work intensity effect on the traffic, $R_{WI}$ =0;

4-Find the speed reduction (mph) due to any treatment
No treatment is implemented on site
$R_t$ = 0 mph

5- Compute the adjusted free flow speed (AFFS) by Equation 10.1:

AFFS= FFS-$R_{WI}$-$R_{LW}$-$R_{LC}$-$R_T$ -$R_o$ = 43– 0–0 – 2 – 0 – 0= 41 mph

6- Find the speed-flow relationship corresponding to a site with speed limit of 45 mph, with flagger and the adjusted free flow speed estimated in step 5. Read the maximum flow rate corresponding to the adjusted free flow speed of the site.

$C_{max}$=1362 pcphpl

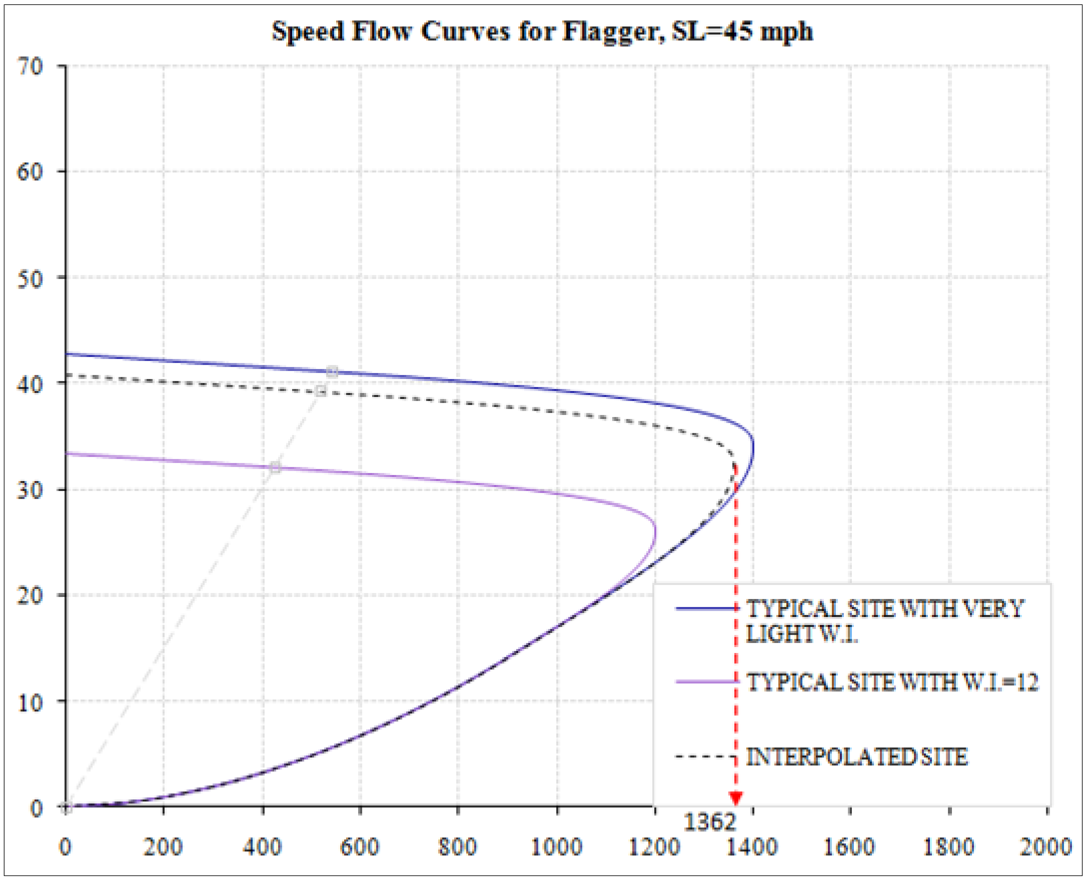

Figure 10-5.The capacity and corresponding operating speed for a site with speed limit of 45 mph, flagger, and AFFS of 40.88 mph

7- Since the traffic is not asked to stop, then the capacity is equal to the maximum flow rate.

Capacity= $C_{Max}$ = 1362 pcphpl

8- Compute the heavy vehicle adjustment factor as below:
Since the site is located on level terrain, the PCE of 1.5 is selected from Table 10.7. Based on the given traffic information, 28% of the traffic is heavy vehicles ($P_T$=0.28) and $f_{HV}$ is computed as below:

$$f_{HV} = \frac{1}{1 + P_T\,(PCE - 1)} = \frac{1}{1 + 0.28\,(1.5 - 1)} = 0.88$$

9- Calculate the adjusted capacity: By using the capacity value obtained in Step 7, 1359 pcphpl, and $f_{HV}$ obtained in Step 8, 0.88:
$C_{adj} = C \times f_{HV}$ = 1362 * 0.88 ≈1199 vphpl

10-Since there is no queue left from the previous hour, $n_0$=0.Hence demand is computed as below

$D_1=n_0+V_1=0+800=800$ vph

Determine the operating speed ($U_o$): One lane is open in the activity area ($N_{op}$=1) and $C_{adj}$ , from Step 9, is 1199.Hence, the departure rate ($C_{adj}$*$N_{op}$) is 1199 vph. The demand (800 vph) is less than the departure rate, so it is needed to compute the adjusted volume to estimate the operating speed.

One lane is open within the activity area and the hourly volume is 800 vph ($V_1$=800). Also we know from Step 8 that $f_{HV}$=0.88.Therefore, the passenger-car equivalent hourly volume is calculated as below:

$$V_{PCE} = \frac{V_1}{f_{HV} * N_{op}} = \frac{800}{0.88*1} = 909 \text{ pcphpl}$$

The operating speed is determined by entering the passenger-car equivalent volume on the horizontal axis and reading the corresponding speed. Figure 10.6 shows that $U_o$= 37.95mph.

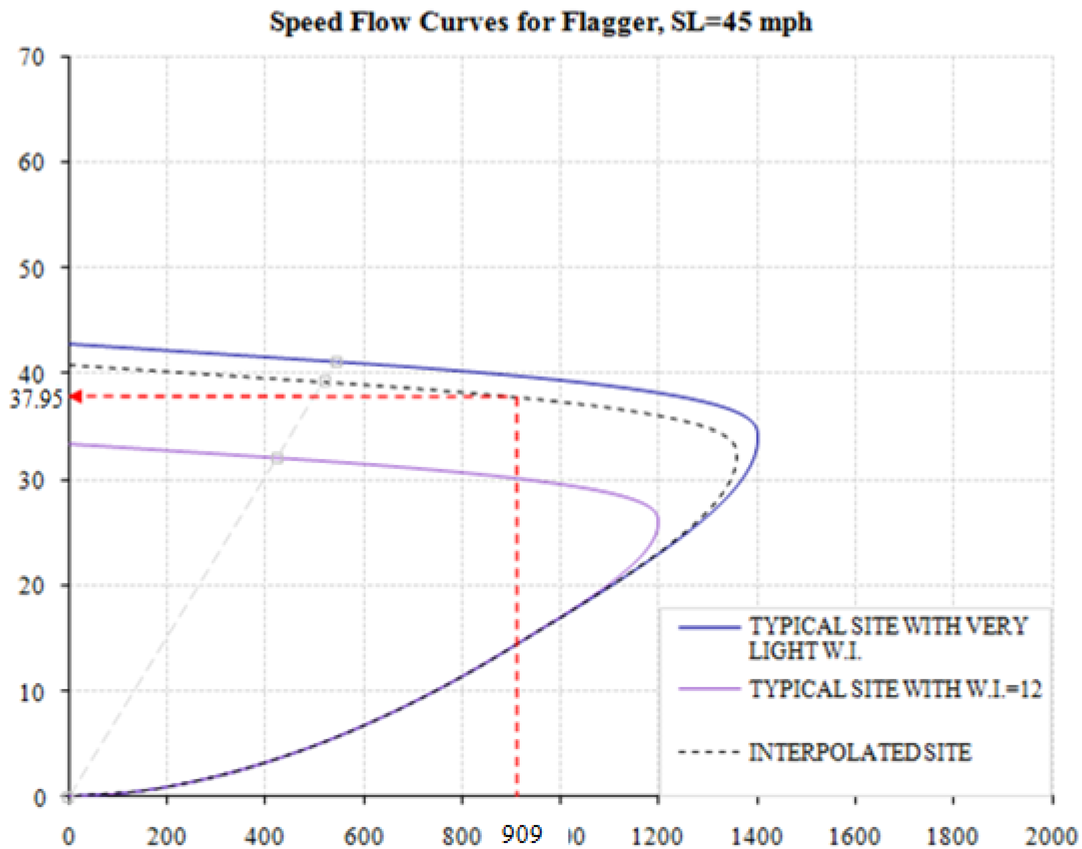


Figure 10-6. Operating speed corresponding to the volume of 909 pcphpl for a site with speed limit of 45 mph, flagger, and AFFS of 41 mph

11- Since the arrival volume ($V_1$=800 vph) is less than the departure rate ($C_{adj}$*$N_{op}$ =1199*1=1199 vph) and there is no vehicle left in queue from the previous interval ($n_0$=0), $d_{q1}$=0, $n_1$=0, $ń_1$=0 and Steps 11 and 12 will be skipped.

13- The demand ($D_1$=800 vph) is not greater than the departure rate (1199vph), so the delay due to an operating speed less than speed limit is computed as below:

The speed limit within the activity area is 45 mph and $U_0$<45 mph, then:

$d_{u1} = (\frac{E}{U_o} - \frac{E}{45}) = (\frac{1.5}{37.95} - \frac{1.5}{45})$=0.006 hr/veh

14) Since there is no queue in the beginning of the interval, $n_0$=0. The adjusted capacity, $C_{adj}$, (from Step 10) and hourly volume, $V_1$, are 1196 and 800, respectively. Hence,$\beta$ is estimated as below:

$$\beta = \frac{n_{i-1}}{C_{adj*N_{OP}} - V_i} = \frac{0}{1199 * 1 - 800} = 0$$

and

$d_1 = d_{q1} + d_{u1}$=0.006+0=0.006 hr/veh

The total delay is calculated by using the following equation:

$D_{t1} = V_1 * d_1 = 800 * 0.006 = 4.8\ \text{hr}$

Compute the users' cost for the 1st hour as below:

$UC_1 = D_{t1}*(P_{SUT1}*C_{SUT} + P_{MUT1}*C_{MUT}+P_{C1}*C_C*N_{OCC})$

The percentage of single-unit and multiple-unit trucks is 2 and 26, respectively. The dollar value of time for cars, single-unit trucks and multiple-unit trucks is 20, 70 and 90 $/hr, respectively. $N_{OCC}$ is 1.25.

$UC_1$=4.8*(0.02*70+0.26*90+0.72*20*1.25) =4.8*42.8=$205.44

15) Do Step 1 to 14 for each hour of the study. So in the next section, the site is analyzed for the second hour of the study.

### 10.2.3 Solution for the Second Hour

1. Find the speed reductions due to the less-than ideal lane width ($R_{LW}$) and lateral clearance
($R_{LC}$) from Table 10.1:
Ideal lane width: 12 ft. Then:
$R_{LW}$ =0

There is no left shoulder, so the speed reduction due to the lack of left shoulder=2 mph
There is 4-ft right shoulder, so the speed reduction due to right shoulder width=1.2 mph
$R_{LC}$ = 3.2mph

2- Determine the level of work intensity.
Since there is no concrete barrier between the workers and the construction equipment, then the effect of the work intensity is more similar to that in a short term work zone rather than in a long term work zone. Hence, Table 10.2 is used.
No. of workers+ No. of large construction equipment = 6+3=9
Lateral distance between the work activity area and the edge of the open lane= 4 ft
Based on Table 10.2, the work intensity is moderate

Table 10-8. Work Intensity for a Site with 6 Workers, 3 Equipments and 4 ft Lateral Distance

| Lateral distance between the work activity area and the edge of the open lane (ft) | | (# of workers) + (# of large construction equipment) in the work activity area | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **12** | **13** | **14** | **15** |
| | **9** | LO | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| | **8** | LO | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| | **7** | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| | **6** | LO | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO |
| | **5** | LO | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI |
| | **4** | LO | LO | MO | MO | MO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI |
| | **3** | LO | LO | MO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI |
| | **2** | LO | MO | MO | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |
| | **1** | MO | MO | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI | HI |

LO=Low work intensity MO=Moderate work intensity HI=High work intensity

3- Find the speed reduction corresponding to the work intensity determined in step 2. Use Table 10.4 for short term work zones and Table 10.5 for long term work zones.

By using Table 10.4, the speed reduction due to moderate work intensity:
$R_{WI}$ = 12 mph

4-Find the speed reduction (mph) due to any treatment
No treatment was implemented at the site, so:
$R_t$ = 0 mph

5- Compute the adjusted free flow speed (AFFS) by Equation 10.1:
$AFFS= FFS-R_{WI}-R_{LW}-R_{LC}-R_T -R_o = 43 – 12 –0 – 3.2 – 0 – 0= 27.8$ mph

6- Find the speed-flow relationship corresponding to a site with speed limit of 45 mph, with flagger and the adjusted free flow speed estimated in step 5. Read the maximum flow rate corresponding to the adjusted free flow speed of the site.
$C_{Max}$ =1082 pcphpl

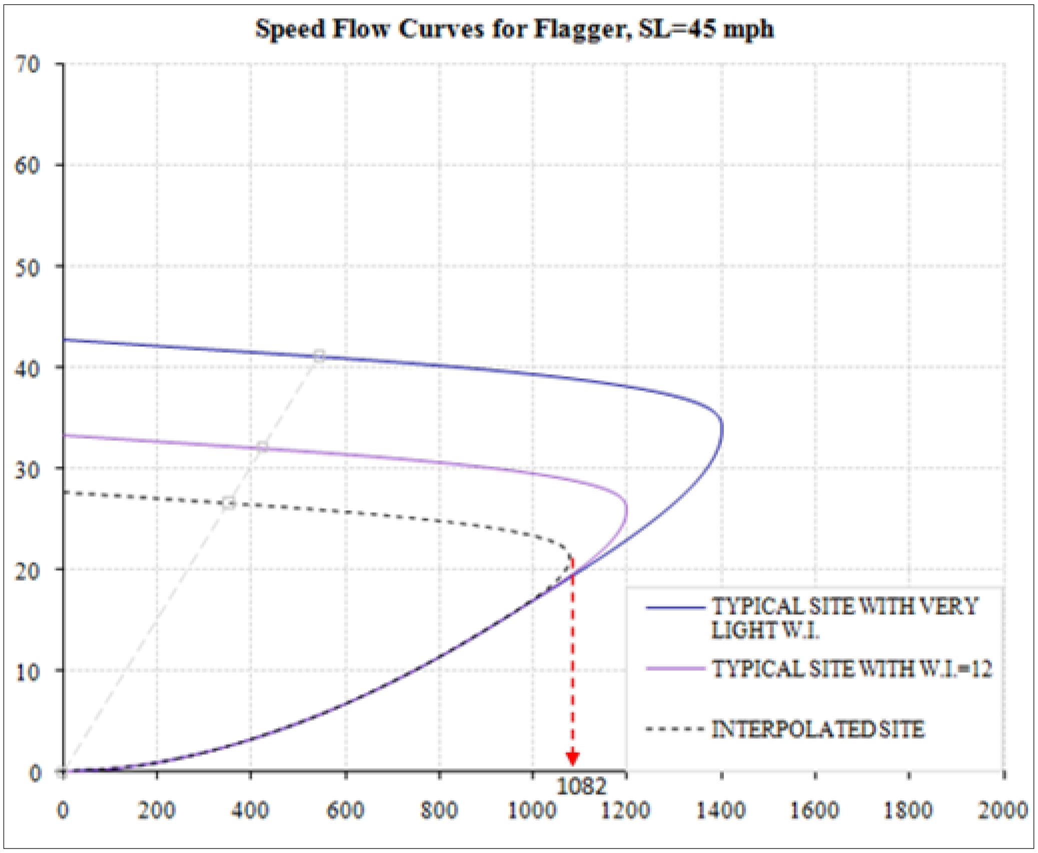


Figure 10-7. Capacity and corresponding speed for a site with speed limit of 45 mph, flagger, and AFFS of 27.8 mph

7- Since the traffic is not asked to stop, then the capacity is equal to the maximum flow rate.

$C=C_{Max}$ = 1082 pcphpl

8- Compute the heavy vehicle adjustment factor as below:

$$f_{HV} = \frac{1}{1 + P_T\,(PCE - 1)} = \frac{1}{1 + 0.28\,(1.5 - 1)} = 0.88$$

9- Calculate the adjusted capacity:

$C_{adj} = C \times f_{HV} * N_{op} = 1082 * 0.88 * 1 \approx 952$ vph

10-The number of vehicles in queue at the end of the first hour obtained from step 11 of the first hour is zero, so:

$D_2=n_1+V_2=0+1100=1100$ vph

Determine the operating speed ($U_o$):

Since the demand (=1100vph) is greater than the departure rate (952 pcph), then the operating speed is the speed at capacity. The speed at the maximum flow rate of 1082 pcphpl is found from Figure 10.7 as 21.27 mph. Thus,

$U_o$= 21.27 mph

11- The arriving volume ($V_2$=1100) is greater than the adjusted capacity ($C_{adj}$=952), so:

$n_2$=max (0, $D_1$ –$C_{adj}$*$N_{op}$) = max (0, 0+1100-952*1)=max(0,148)=148 vehicles

The number of vehicles at the end of the first hour, $n_1$, was determined in Step 11 of the analyses of the first hour (section 10.2.1). The volume during the second hour,$V_2$, is already given as 1100 vph. Adjusted capacity comes from the step 9.

Compute l (average spacing between vehicles) as below:

l= $\frac{U_o}{Cap}$=$\frac{21.27}{952} * 5280$=118.0ft

Calculate the stacked queue length ($Q_{S2}$):
$Q_{S2}$ = $n_2$* l/5280 = 148 118/5280= 3.3 mile

The distance from the beginning of the transition area to the end of the activity area, C, is 2.5 miles and it is less than stacked queue length, $Q_{S2}$. So the queue length at the end of the second hour is estimated as below:

$Q_2$= C+ ($Q_{s2}$ – C*$N_{op}$)/$N_{nr}$=2.5 + (3.3 -2.5*1)/2= 2.9miles
And
$n_{c2} = \frac{Q_2 - C}{l} * 5280 = \frac{2.9-2.5}{118} * 5280 \approx$18 vehicles

Determine the average queue length:

$$\overline{Q}_2 = \frac{Q_1 + Q_2}{2} = \frac{0 + 2.9}{2} = 1.45 \text{ mile}$$

Determine the average number of vehicles on the closed lane:
$\overline{n}_{c2} = \frac{(n_{c2}+n_{c1})}{2} = \frac{(18+0)}{2} \approx$9 vehicles
$n_{c1}$ comes from Step 11 of the analyses for the first hour.

12- Determine the distances (ft) B, C, D shown in Figure 10.4. The distances B, C, and D are 0.8, 2.5, and 1.7 miles, respectively. Since the speed limit in the vicinity of the work space is 45 mph and $\overline{Q}_2 < D$,
$d_{q2} = \left(\frac{\overline{Q}_2}{U} - \frac{\overline{Q}_2}{45}\right) + \overline{n}_{c2}\overline{\gamma}$= $(\frac{1.45}{21.27} - \frac{1.45}{45})$ +9$\overline{\gamma}$=0.036+9$\overline{\gamma}$
Also $\overline{\gamma}$= $\frac{1}{C_{adj}}$=$\frac{1}{950}$ =1.053*$10^{-3}$
As a result:
$d_{q2}$ =0.036+9$\overline{\gamma}$ =0.036+9*1.053*$10^{-3}$≈0.045hr/veh

13- The demand (1100) is more than the adjusted capacity (948), then $d_{u2}$=0 and this step will be skipped.

14-Estimate β using Equation 10.19:

$\beta = \frac{n_1}{C_{adj}*N_{OP}-V_2} = \frac{0}{950*1-1100} = 0$

Estimate the average delay as below:
$d_2 = d_{q2} + d_{u2}$=0.045+0=0.045hr/veh
Estimate the total delay as follow:
$D_{t2} = V_2 * d_2$=1100*0.045= 49.5 hr

Compute the users' cost for the 2nd hour:
$UC_2 = (D_{t2})*(P_{SUT2}*C_{SUT} + P_{MUT2}*C_{MUT}+P_{C2}*C_C*N_{OCC})$
The parameters of the above equation are the same as Step 14 of the first hour except $D_{t2}$ is different, hence:
$UC_2$ =49.5*42.8=$2118.6

15 - Do Step 1 to 14 for each hour of the study. So in the next section, the site is analyzed for the third hour of the study.

### 10.2.4 Solution for the Third Hour

All the conditions except demand are the same as the second hour. So the results obtained from Steps 1 to 9 are the same as those of the second hour.

10- The number of vehicles in \queue at the end of the second hour, $n_2$, is 148 and the arriving volume of the third hour, $V_3$, is 600, so
$D_3=n_2+V_3$=148+600=748 vehicles

The demand (748 vehicles) is less than the adjusted capacity (952 vehicles).
$V_{PCE} = \frac{V_1}{f_{HV}*N_{op}} = \frac{600}{0.88*1}$=681 pcphpl
As shown in Figure 10.8, the operating speed corresponding to 681 pcphpl is 25.52 mph.

11- Since there are some vehicles left in queue from the previous hour,
$n_3$=max (0, $D_i$-$C_{adj}$*$N_{op}$) =max (0,748-952*1) =max (0,-204) =0
Hence, the stacked queue length, $Q_{s3}$, and queue length, $Q_3$, at the end of this hour is zero. Also $\acute{n}_3$=0.

Determine the average queue length:

$$\overline{Q}_3 = \frac{Q_2 + Q_3}{2} = \frac{2.9 + 0}{2} = 1.45 \text{ mile}$$

Determine the average number of queued vehicles on the open lane:
$\bar{n}_{c3} = \frac{(n_{c3}+n_{c2})}{2} = \frac{(0+18)}{2} \approx 9$ vehicles

12-This step is the same as Step 12 of the second hour, so $d_{q3}$= 0.045 hr

13-The demand (748 vehicles) is less than the adjusted capacity (952 vehicles). The speed limit within the activity area is 45 mph and $U_0$<45 mph, then:
$d_{u3} = (\frac{E}{U_o} - \frac{E}{45}) = (\frac{1.5}{25.52} - \frac{1.5}{45})$=0.025 hr/veh

14- β is estimated as below:

$\beta = \frac{n_2}{C_{adj} - V_3} = \frac{148}{952-600} \approx 0.42$

$\beta$ is between 0 and 1, so the average delay is computed as follows:

$d_3 = \beta * d_{q3} + (1-\beta) * d_{u3}$=0.42*0.045+ (1-0.42)*0.025=0.0334 hr/veh

Estimate the total delay during the 3rd hour:

$D_{t3} = V_3 * d_3$=600*0.0334=20.04 hr

Compute the users' cost for the 3rd hour:

$UC_3 = (D_{t3})*(P_{SUT3}*C_{SUT} + P_{MUT3}*C_{MUT} + P_{C3}*C_C*N_{OCC})$

The parameters of the above equation, except $d_{t3}$, are the same as the one for the first hour hence:

$UC_3$ =20.04*42.8=$857.712

15- Step 1 to 14 was followed for each hour. Then the total users' cost is:

$UC = \sum_{i=1}^{3} UC_i$ =UC1+UC2+UC3=205.44+2118.6+ 857.712=$3181.752

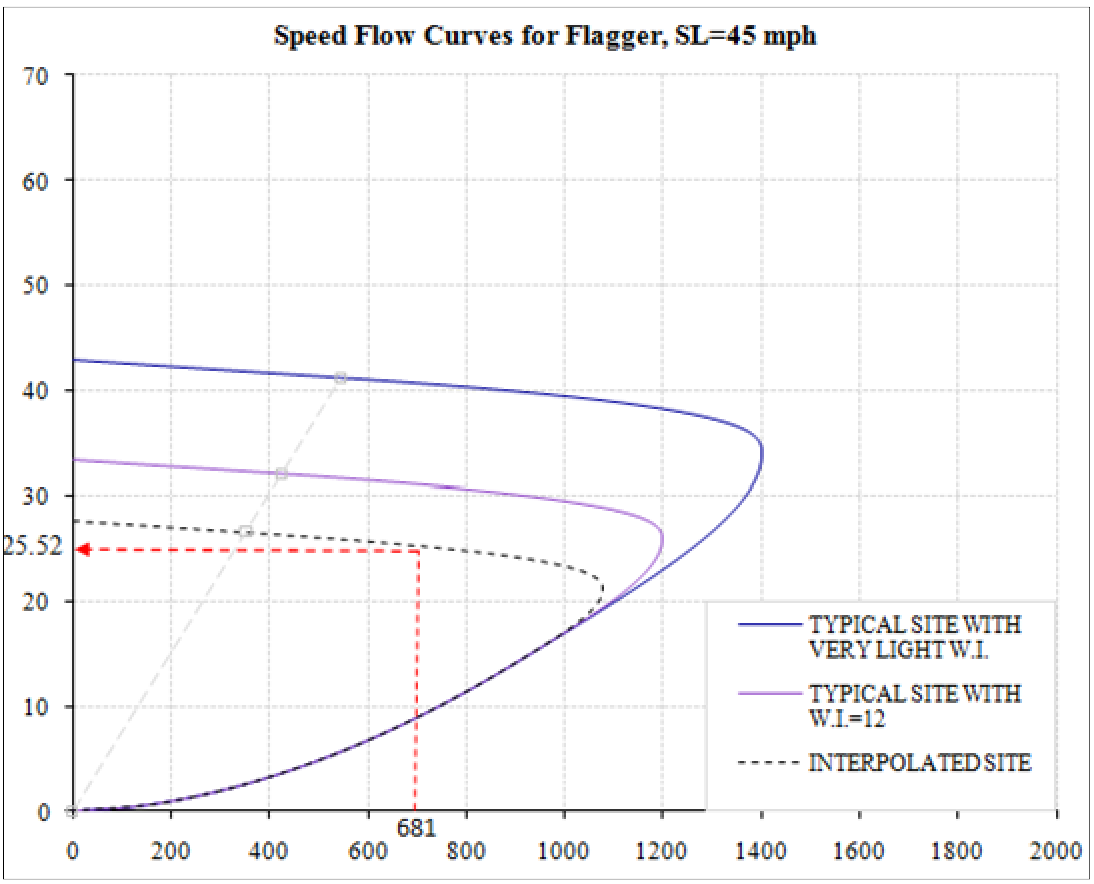


Figure 10-8. Operating speed corresponding to the volume of 681 pcphpl for a site with speed limit of 45 mph, flagger, and AFFS of 27.8 mph

# CHAPTER 11 CONCLUSIONS AND RECOMMENDATIONS

Field data were collected from six work zones sites in Illinois. Thirteen data sets were extracted from these sites (five with queuing condition and eight with non-queuing condition). The sites were 2-to-1 work zones with different prevailing conditions. Capacity values were suggested based on the field data for different work zone conditions. A model was also proposed to estimate moving queue length and corresponding delay.QuickZone2, which is usually used for delay and users' costs estimation in work zones, was also evaluated.

The suggested capacity for the sites with speed limit of 45 mph ranges from 1200 pcphpl to 1550 pcphpl. The lowest capacity was suggested a traffic condition with flagger and queue. The highest belongs to a traffic condition with no work activity, no treatment and no queue. A capacity of 1700 pcphpl was suggested for a site with speed limit of 55 mph, no work activity, no queue, and long-distance work zone. For the same traffic condition but short-distance work zone, 1750 pcphpl was recommended.

Three speed-flow curves were proposed for different types of work zones: work zones with speed limit of 45 mph and with a flagger, work zones with speed limit of 45 mph and without a flagger, and work zones with speed limit of 55 mph. Each of these models can be adjusted to non-ideal conditions. The speed-flow curves are used to estimate operating speed and capacity which are then used to compute delay and queue length. The proposed models were based on the data collected from 2-lane-to-1-lane work zones.

Methods to estimate the length of moving queue and delay were developed to handle cases where a higher demand than capacity causes queue.

Comparison of field data from work zones that had moving or intermittent queues to QuickZone2 results, indicated that QuickZone2 returned zero delay and queue for all five data sets that actually had queues and delays. For undersaturated work zones (no slow moving or intermittent queues), Quickzone2 did not estimate any delay for any of the eight data sets while six of the data sets actually had delay which ranges from 0.1 to 4.8 hr, due to travel speeds below the speed limit. To avoid significant error, QuickZone should not be used when there is queue or slow moving vehicles in the work zones.

The following recommendations are made:

- Use the proposed method to estimate capacity, operating speed, delay, queue length, and users' costs in work zones with queuing is expected.
- The proposed method should be further be evaluated by getting input from IDOT engineers after they are trained on how to use the methods.
- Develop a "computer programs" so IDOT engineers can easily use the proposed methods.
- The proposed speed-flow curves are based on the data collected from 2-to-1 work zones. A similar study is recommended for other lane configurations.
- The proposed method suggested a very approximate way of accounting the effect of some of ITS technologies/techniques on work zone capacity, however, detailed study on these issues is needed.
- Effect of flow breakdown on capacity and delay needs to be investigated and preventative measures needs be identified.
- Effects of truck in delay and queue length need further studies to properly determine the trucks impacts in work zones.

- Work intensity and corresponding speed reduction are based on the survey of drivers in a previous study. It is recommended to conduct a field study to refine the validity of that relationship.

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# APPENDIX A: LOOK-UP TABLES FOR KEY POINTS ON THE SPEED-FLOW CURVE

In this appendix look-up tables are provided to determine key points on four-regime speed-flow curves for a given intercept (operating speed). The schematic definitions of the key-points are shown in Figure A-1.

Table A-1: Look-Up Table for Key Points On The Speed-Flow Curve For Sites With Flagger & 45-Mph Speed Limit

| | Bending Point | | Peak Point | | Spline-to-Congestion Model Connection Point | |
|---|---|---|---|---|---|---|
| **Intercept Speed** | Flow (pcphpl) | Speed (mph) | Capacity (pcphpl) | Optimum Speed (mph) | Flow (pcphpl) | Speed (mph) |
| **55.00** | 700 | 52.83 | 1659 | 44.34 | 900 | 14.10 |
| **53.00** | 675 | 50.91 | 1616 | 42.64 | 900 | 14.10 |
| **51.00** | 649 | 48.99 | 1574 | 40.95 | 900 | 14.10 |
| **49.00** | 624 | 47.07 | 1531 | 39.25 | 900 | 14.10 |
| **47.00** | 598 | 45.15 | 1489 | 37.55 | 900 | 14.10 |
| **45.00** | 573 | 43.22 | 1446 | 35.86 | 900 | 14.10 |
| **43.00** | 547 | 41.30 | 1404 | 34.16 | 900 | 14.10 |
| **41.00** | 522 | 39.38 | 1362 | 32.46 | 900 | 14.10 |
| **39.00** | 497 | 37.46 | 1319 | 30.77 | 900 | 14.10 |
| **37.00** | 471 | 35.54 | 1277 | 29.07 | 900 | 14.10 |
| **35.00** | 446 | 33.62 | 1234 | 27.37 | 900 | 14.10 |
| **33.00** | 420 | 31.70 | 1192 | 25.68 | 900 | 14.10 |
| **31.00** | 395 | 29.78 | 1150 | 23.98 | 900 | 14.10 |
| **29.00** | 369 | 27.86 | 1107 | 22.28 | 900 | 14.10 |
| **27.00** | 344 | 25.93 | 1065 | 20.59 | 900 | 14.10 |
| **25.00** | 318 | 24.01 | 1022 | 18.89 | 900 | 14.10 |
| **23.00** | 293 | 22.09 | 980 | 17.19 | 900 | 14.10 |

Table A-2: Look-Up Table for Key Points On The Speed-Flow Curve For Sites With 45-Mph Speed Limit & No Flagger

| | Bending Point | | Peak Point | | Spline-to-Congestion Model Connection Point | |
|---|---|---|---|---|---|---|
| **Intercept Speed** | Flow (pcphpl) | Speed (mph) | Capacity (pcphpl) | Optimum Speed (mph) | Flow (pcphpl) | Speed (mph) |
| **60.00** | 755 | 56.98 | 1650 | 45.00 | 900 | 16.06 |
| **58.00** | 730 | 55.08 | 1610 | 44.20 | 900 | 16.06 |
| **56.00** | 705 | 53.18 | 1570 | 43.40 | 900 | 16.06 |
| **54.00** | 679 | 51.28 | 1530 | 42.60 | 900 | 16.06 |
| **52.00** | 654 | 49.38 | 1490 | 41.80 | 900 | 16.06 |
| **50.00** | 629 | 47.48 | 1450 | 41.00 | 900 | 16.06 |
| **48.00** | 604 | 45.58 | 1410 | 39.80 | 900 | 16.06 |
| **46.00** | 579 | 43.69 | 1370 | 38.45 | 900 | 16.06 |
| **44.00** | 553 | 41.79 | 1330 | 36.97 | 900 | 16.06 |
| **42.00** | 528 | 39.89 | 1290 | 35.38 | 900 | 16.06 |
| **40.00** | 503 | 37.99 | 1250 | 33.69 | 900 | 16.06 |
| **38.00** | 478 | 36.09 | 1210 | 31.91 | 900 | 16.06 |
| **36.00** | 453 | 34.19 | 1170 | 30.04 | 900 | 16.06 |
| **34.00** | 427 | 32.29 | 1130 | 28.09 | 900 | 16.06 |
| **32.00** | 402 | 30.39 | 1090 | 26.06 | 900 | 16.06 |
| **30.00** | 377 | 28.49 | 1050 | 23.96 | 900 | 16.06 |
| **28.00** | 352 | 26.59 | 1010 | 21.80 | 900 | 16.06 |
| **26.00** | 327 | 24.69 | 970 | 19.57 | 900 | 16.06 |

Table A-3: Look-Up Table for Key Points On The Speed-Flow Curve For Sites With 55-Mph Speed Limit

| | Bending Point | | Peak Point | | Spline-to-Congestion Model Connection Point | |
|---|---|---|---|---|---|---|
| **Intercept Speed** | Flow (pcphpl) | Speed (mph) | Capacity (pcphpl) | Optimum Speed (mph) | Flow (pcphpl) | Speed (mph) |
| **70.00** | 930 | 70.00 | 2000 | 61.00 | 1200 | 21.19 |
| **68.00** | 903 | 68.00 | 1950 | 59.00 | 1200 | 21.19 |
| **66.00** | 876 | 66.00 | 1900 | 57.00 | 1200 | 21.19 |
| **64.00** | 849 | 64.00 | 1850 | 55.00 | 1200 | 21.19 |
| **62.00** | 822 | 62.00 | 1800 | 53.00 | 1200 | 21.19 |
| **60.00** | 795 | 60.00 | 1750 | 51.00 | 1200 | 21.19 |
| **58.00** | 769 | 58.00 | 1700 | 49.00 | 1200 | 21.19 |
| **56.00** | 742 | 56.00 | 1643 | 47.57 | 1143 | 19.17 |
| **54.00** | 716 | 54.00 | 1586 | 46.14 | 1086 | 17.25 |
| **52.00** | 689 | 52.00 | 1529 | 44.71 | 1029 | 15.44 |
| **50.00** | 663 | 50.00 | 1471 | 43.21 | 971 | 13.73 |
| **48.00** | 636 | 48.00 | 1414 | 41.60 | 914 | 12.12 |
| **46.00** | 610 | 46.00 | 1357 | 39.95 | 857 | 10.61 |
| **44.00** | 583 | 44.00 | 1300 | 38.28 | 800 | 9.21 |
| **42.00** | 556 | 42.00 | 1243 | 36.59 | 743 | 7.91 |
| **40.00** | 530 | 40.00 | 1186 | 34.87 | 686 | 6.71 |
| **38.00** | 503 | 38.00 | 1129 | 33.13 | 629 | 5.61 |
| **36.00** | 477 | 36.00 | 1071 | 31.38 | 571 | 4.61 |
| **34.00** | 450 | 34.00 | 1014 | 29.61 | 514 | 3.72 |
| **32.00** | 424 | 32.00 | 957 | 27.82 | 457 | 2.92 |

**EXAMPLE:** Draw the speed-flow curve for a site with flagger and 45-mph speed limit that has an intercept speed (i.e. adjusted free-flow speed) of 36 mph.

**SOLUTION:** Table **A-1** gives the coordinates of the key points for sites with flagger and 45-mph speed limit for several intercept speeds. By using Table, the key points for a speed-flow curve with intercept speed of 36-mph is found by linear interpolation as follows:

| | Bending Point | | Peak Point | | Spline-to-Congestion Model Connection Point | |
|---|---|---|---|---|---|---|
| Intercept Speed | Flow (pcphpl) | Speed (mph) | Capacity (pcphpl) | Optimum Speed (mph) | Flow (pcphpl) | Speed (mph) |
| 37.00 | 471 | 35.54 | 1277 | 29.07 | 900 | 14.10 |
| **36.00** | **458** | **34.58** | **1256** | **28.22** | **900** | **14.10** |
| 35.00 | 446 | 33.62 | 1234 | 27.37 | 900 | 14.10 |

Once the key points of the speed-flow curve are determined, the new speed flow curve with the intercept speed of 36 mph can be approximately drawn parallel to the existing speed-flow curves. The new speed-flow curve is shown in Figure A-1:. A detailed procedure to obtain the exact-speed flow curve for a given intercept speed is given in an extended appendix, which is available on request.

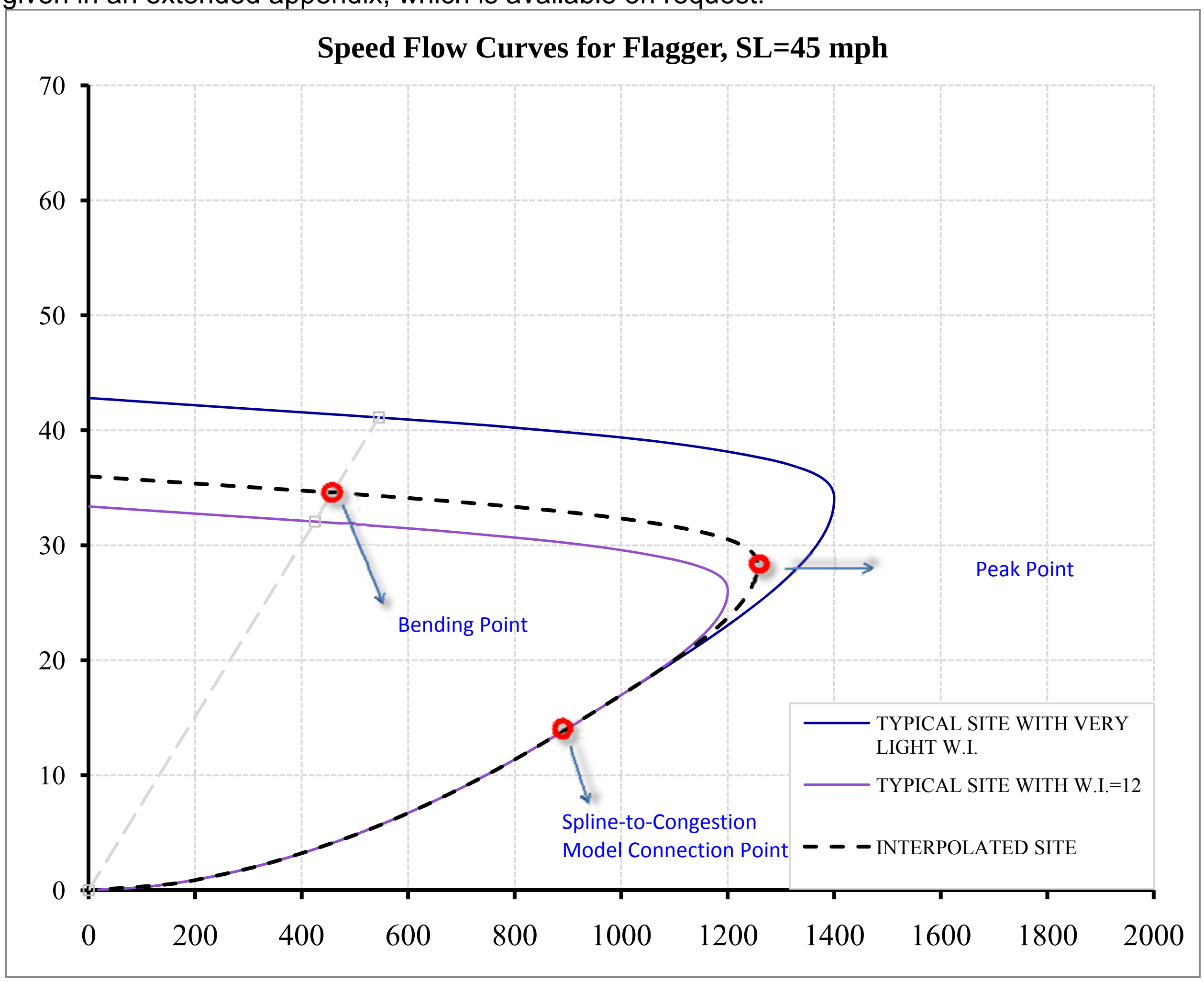


Figure A-1: Speed-flow curve for a site with flagger and 45-mph speed limit that has an intercept speed of 36 mph.

# APPENDIX B: FLOW RATE LOOK-UP TABLES

In this appendix, look-up tables are provided to read the flow rate, corresponding to a given intercept and operating speed. These tables are established based on the four-regime speed-flow curves.

Table B-1: Flow Rate Look-Up Table for Sites With Flagger & 45-Mph Speed Limit

| | **INTERCEPT SPEED** | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **SPEED ON THE SPEED-FLOW CURVE** | **55** | **53** | **51** | **49** | **47** | **45** | **43** | **41** | **39** | **37** | **35** | **33** | **31** | **29** | **27** | **25** | **23** |
| **0-14** | Compute the flow rate from the equation $\mathbf{Flow} = \mathbf{211.56} * (\mathbf{Speed})^{\mathbf{0.5472}}$ for $0 \le \text{Speed} \le 14.10$ | | | | | | | | | | | | | | | | |
| **15** | 932 | 932 | 932 | 932 | 932 | 932 | 932 | 932 | 932 | 932 | 932 | 932 | 931 | 931 | 931 | 931 | 931 |
| **17** | 1001 | 1001 | 1001 | 1001 | 1001 | 1001 | 1001 | 1001 | 1001 | 1001 | 1001 | 1000 | 1000 | 999 | 998 | 994 | 979 |
| **19** | 1071 | 1070 | 1070 | 1070 | 1070 | 1070 | 1070 | 1069 | 1069 | 1068 | 1067 | 1066 | 1063 | 1059 | 1049 | 1022 | 909 |
| **21** | 1139 | 1139 | 1138 | 1138 | 1138 | 1137 | 1136 | 1135 | 1133 | 1131 | 1128 | 1123 | 1114 | 1098 | 1063 | 939 | 596 |
| **23** | 1206 | 1206 | 1205 | 1204 | 1203 | 1201 | 1199 | 1196 | 1193 | 1187 | 1179 | 1167 | 1145 | 1103 | 964 | 605 | 0 |
| **25** | 1272 | 1271 | 1269 | 1267 | 1265 | 1261 | 1257 | 1252 | 1244 | 1233 | 1217 | 1190 | 1140 | 986 | 612 | 0 | - |
| **27** | 1335 | 1333 | 1330 | 1326 | 1322 | 1316 | 1309 | 1299 | 1285 | 1265 | 1234 | 1176 | 1006 | 618 | 0 | - | - |
| **29** | 1395 | 1391 | 1386 | 1381 | 1373 | 1364 | 1352 | 1335 | 1311 | 1277 | 1211 | 1025 | 623 | 0 | - | - | - |
| **31** | 1451 | 1445 | 1438 | 1429 | 1418 | 1403 | 1383 | 1356 | 1319 | 1244 | 1043 | 627 | 0 | - | - | - | - |
| **33** | 1502 | 1494 | 1483 | 1470 | 1453 | 1430 | 1401 | 1360 | 1275 | 1060 | 631 | 0 | - | - | - | - | - |
| **35** | 1548 | 1536 | 1521 | 1501 | 1477 | 1445 | 1401 | 1305 | 1076 | 634 | 0 | - | - | - | - | - | - |
| **37** | 1588 | 1571 | 1549 | 1523 | 1488 | 1440 | 1333 | 1091 | 637 | 0 | - | - | - | - | - | - | - |
| **39** | 1620 | 1596 | 1567 | 1531 | 1479 | 1361 | 1105 | 640 | 0 | - | - | - | - | - | - | - | - |
| **41** | 1643 | 1612 | 1574 | 1516 | 1387 | 1119 | 642 | 0 | - | - | - | - | - | - | - | - | - |
| **43** | 1656 | 1616 | 1552 | 1413 | 1132 | 643 | 0 | - | - | - | - | - | - | - | - | - | - |
| **45** | 1657 | 1587 | 1437 | 1145 | 644 | 0 | - | - | - | - | - | - | - | - | - | - | - |
| **47** | 1621 | 1461 | 1157 | 645 | 0 | - | - | - | - | - | - | - | - | - | - | - | - |
| **49** | 1483 | 1168 | 645 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **51** | 1179 | 645 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **53** | 645 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **55** | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

Table B-2: Flow Rate Look-Up Table for Sites with 45-Mph Speed Limit & No Flagger

| SPEED ON THE SPEED-FLOW CURVE | INTERCEPT SPEED | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **60** | **58** | **56** | **54** | **52** | **50** | **48** | **46** | **44** | **42** | **40** | **38** | **36** | **34** | **32** | **30** | **28** | **26** |
| **0-16** | Compute the flow rate from the equation $\mathbf{Flow} = \mathbf{109.30} * (\mathbf{Speed})^{\mathbf{0.7594}}$ for $0 \leq \text{Speed} \leq 16.06$ | | | | | | | | | | | | | | | | | |
| **18** | 982 | 982 | 982 | 982 | 982 | 982 | 982 | 982 | 982 | 982 | 982 | 981 | 981 | 980 | 979 | 977 | 973 | 962 |
| **20** | 1066 | 1066 | 1065 | 1065 | 1065 | 1064 | 1064 | 1063 | 1062 | 1061 | 1059 | 1057 | 1054 | 1050 | 1043 | 1031 | 1008 | 970 |
| **22** | 1147 | 1146 | 1145 | 1144 | 1143 | 1141 | 1139 | 1137 | 1134 | 1131 | 1126 | 1120 | 1111 | 1099 | 1080 | 1050 | 1010 | 869 |
| **24** | 1225 | 1223 | 1220 | 1218 | 1215 | 1211 | 1207 | 1202 | 1196 | 1189 | 1179 | 1166 | 1148 | 1124 | 1091 | 1050 | 909 | 498 |
| **26** | 1298 | 1294 | 1290 | 1285 | 1279 | 1273 | 1265 | 1257 | 1246 | 1233 | 1216 | 1195 | 1167 | 1131 | 1090 | 947 | 503 | 0 |
| **28** | 1366 | 1359 | 1352 | 1344 | 1335 | 1325 | 1313 | 1300 | 1283 | 1263 | 1239 | 1208 | 1171 | 1130 | 981 | 505 | 0 | - |
| **30** | 1427 | 1418 | 1407 | 1396 | 1382 | 1368 | 1351 | 1332 | 1309 | 1281 | 1249 | 1211 | 1170 | 1011 | 506 | 0 | - | - |
| **32** | 1482 | 1469 | 1454 | 1438 | 1420 | 1400 | 1378 | 1353 | 1323 | 1289 | 1251 | 1210 | 1033 | 504 | 0 | - | - | - |
| **34** | 1529 | 1512 | 1493 | 1472 | 1449 | 1424 | 1396 | 1364 | 1329 | 1290 | 1249 | 1047 | 502 | 0 | - | - | - | - |
| **36** | 1569 | 1548 | 1524 | 1498 | 1470 | 1439 | 1406 | 1369 | 1330 | 1286 | 1051 | 501 | 0 | - | - | - | - | - |
| **38** | 1601 | 1575 | 1546 | 1515 | 1482 | 1447 | 1409 | 1370 | 1314 | 1045 | 500 | 0 | - | - | - | - | - | - |
| **40** | 1625 | 1594 | 1561 | 1526 | 1489 | 1450 | 1410 | 1330 | 1033 | 500 | 0 | - | - | - | - | - | - | - |
| **42** | 1641 | 1606 | 1569 | 1530 | 1490 | 1444 | 1332 | 1016 | 500 | 0 | - | - | - | - | - | - | - | - |
| **44** | 1649 | 1610 | 1570 | 1524 | 1458 | 1323 | 999 | 500 | 0 | - | - | - | - | - | - | - | - | - |
| **46** | 1649 | 1604 | 1549 | 1463 | 1300 | 985 | 500 | 0 | - | - | - | - | - | - | - | - | - | - |
| **48** | 1635 | 1570 | 1467 | 1288 | 972 | 500 | 0 | - | - | - | - | - | - | - | - | - | - | - |
| **50** | 1590 | 1476 | 1286 | 968 | 500 | 0 | - | - | - | - | - | - | - | - | - | - | - | - |
| **52** | 1489 | 1290 | 969 | 500 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **54** | 1298 | 972 | 500 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **56** | 976 | 500 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **58** | 500 | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **60** | 0 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

Table B-3: Flow Rate Look-Up Table for Sites With 55-Mph Speed Limit

| | INTERCEPT SPEED | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **70** | **68** | **66** | **64** | **62** | **60** | **58** | **56** | **54** | **52** | **50** | **48** | **46** | **44** | **42** | **40** | **38** | **36** | **34** | **32** |
| **1** | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 | 271 |
| **2** | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 | 380 |
| **3** | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 | 463 |
| **4** | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 533 | 539 |
| **5** | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 594 | 595 | 600 | 614 |
| **6** | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 649 | 650 | 655 | 666 | 686 |
| **7** | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 700 | 704 | 713 | 729 | 754 |
| **8** | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 747 | 750 | 757 | 770 | 790 | 817 |
| **9** | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 791 | 793 | 798 | 809 | 825 | 846 | 875 |
| **10** | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 833 | 838 | 846 | 859 | 876 | 898 | 927 |
| **11** | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 872 | 875 | 882 | 892 | 906 | 923 | 945 | 973 |
| **12** | 910 | 910 | 910 | 910 | 910 | 910 | 910 | 910 | 910 | 910 | 910 | 910 | 911 | 916 | 925 | 936 | 950 | 967 | 987 | 1011 |
| **13** | 946 | 946 | 946 | 946 | 946 | 946 | 946 | 946 | 946 | 946 | 946 | 947 | 950 | 957 | 966 | 977 | 991 | 1006 | 1024 | 1042 |
| **14** | 981 | 981 | 981 | 981 | 981 | 981 | 981 | 981 | 981 | 981 | 981 | 983 | 988 | 996 | 1005 | 1016 | 1028 | 1041 | 1054 | 1067 |
| **15** | 1014 | 1014 | 1014 | 1014 | 1014 | 1014 | 1014 | 1014 | 1014 | 1014 | 1015 | 1019 | 1025 | 1033 | 1042 | 1052 | 1061 | 1071 | 1079 | 1084 |
| **16** | 1047 | 1047 | 1047 | 1047 | 1047 | 1047 | 1047 | 1047 | 1047 | 1047 | 1049 | 1054 | 1061 | 1068 | 1076 | 1084 | 1091 | 1096 | 1097 | 1094 |
| **17** | 1078 | 1078 | 1078 | 1078 | 1078 | 1078 | 1078 | 1078 | 1078 | 1079 | 1083 | 1088 | 1095 | 1102 | 1108 | 1113 | 1116 | 1116 | 1110 | 1098 |
| **18** | 1108 | 1108 | 1108 | 1108 | 1108 | 1108 | 1108 | 1108 | 1109 | 1111 | 1116 | 1121 | 1127 | 1133 | 1137 | 1139 | 1137 | 1131 | 1118 | 1095 |
| **19** | 1138 | 1138 | 1138 | 1138 | 1138 | 1138 | 1138 | 1138 | 1139 | 1143 | 1148 | 1153 | 1158 | 1162 | 1163 | 1161 | 1155 | 1142 | 1120 | 1088 |
| **20** | 1167 | 1167 | 1167 | 1167 | 1167 | 1167 | 1167 | 1167 | 1169 | 1174 | 1179 | 1183 | 1187 | 1188 | 1186 | 1180 | 1168 | 1148 | 1118 | 1075 |
| **21** | 1195 | 1195 | 1195 | 1195 | 1195 | 1195 | 1195 | 1196 | 1199 | 1204 | 1208 | 1212 | 1213 | 1212 | 1206 | 1195 | 1177 | 1150 | 1112 | 1060 |
| **22** | 1222 | 1222 | 1222 | 1222 | 1222 | 1222 | 1222 | 1225 | 1229 | 1233 | 1237 | 1239 | 1238 | 1233 | 1223 | 1207 | 1183 | 1148 | 1102 | 1041 |
| **23** | 1250 | 1250 | 1250 | 1250 | 1250 | 1250 | 1250 | 1253 | 1257 | 1261 | 1264 | 1264 | 1260 | 1251 | 1237 | 1215 | 1185 | 1144 | 1090 | 1022 |
| **24** | 1278 | 1278 | 1277 | 1277 | 1277 | 1277 | 1277 | 1281 | 1285 | 1288 | 1289 | 1286 | 1280 | 1267 | 1248 | 1221 | 1184 | 1136 | 1075 | 1002 |
| **25** | 1305 | 1305 | 1305 | 1305 | 1305 | 1305 | 1304 | 1308 | 1312 | 1314 | 1313 | 1308 | 1297 | 1281 | 1257 | 1224 | 1181 | 1126 | 1060 | 984 |
| **26** | 1333 | 1332 | 1332 | 1332 | 1332 | 1332 | 1331 | 1335 | 1338 | 1338 | 1335 | 1327 | 1312 | 1291 | 1262 | 1224 | 1175 | 1116 | 1046 | 970 |
| **27** | 1360 | 1360 | 1360 | 1359 | 1359 | 1358 | 1358 | 1361 | 1363 | 1361 | 1355 | 1343 | 1325 | 1300 | 1266 | 1222 | 1168 | 1104 | 1032 | 960 |
| **28** | 1387 | 1387 | 1387 | 1386 | 1386 | 1385 | 1384 | 1386 | 1387 | 1383 | 1374 | 1358 | 1336 | 1306 | 1267 | 1218 | 1160 | 1093 | 1022 | 957 |
| **29** | 1414 | 1414 | 1413 | 1413 | 1412 | 1411 | 1409 | 1411 | 1409 | 1403 | 1390 | 1371 | 1345 | 1310 | 1266 | 1213 | 1151 | 1083 | 1015 | 949 |
| **30** | 1442 | 1441 | 1440 | 1439 | 1438 | 1436 | 1434 | 1434 | 1430 | 1421 | 1405 | 1383 | 1352 | 1312 | 1264 | 1207 | 1143 | 1076 | 1014 | 891 |
| **31** | 1468 | 1468 | 1467 | 1465 | 1463 | 1461 | 1458 | 1457 | 1450 | 1438 | 1419 | 1392 | 1357 | 1313 | 1261 | 1201 | 1136 | 1072 | 1001 | 732 |
| **32** | 1495 | 1494 | 1493 | 1491 | 1488 | 1485 | 1481 | 1478 | 1469 | 1453 | 1430 | 1399 | 1360 | 1313 | 1257 | 1195 | 1131 | 1071 | 932 | - |
| **33** | 1522 | 1520 | 1518 | 1516 | 1513 | 1509 | 1504 | 1498 | 1486 | 1467 | 1440 | 1405 | 1362 | 1311 | 1253 | 1190 | 1129 | 1050 | 763 | - |
| **34** | 1548 | 1546 | 1544 | 1541 | 1537 | 1532 | 1525 | 1517 | 1502 | 1479 | 1448 | 1409 | 1363 | 1309 | 1249 | 1187 | 1126 | 971 | - | - |

Table B-3. Flow Rate Look-Up Table for Sites With 55-Mph Speed Limit *(Cont'd)*

| | INTERCEPT SPEED | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **SPEED ON THE SPEED-FLOW CURVE** | **70** | **68** | **66** | **64** | **62** | **60** | **58** | **56** | **54** | **52** | **50** | **48** | **46** | **44** | **42** | **40** | **38** | **36** | **34** | **32** |
| **35** | 1574 | 1572 | 1569 | 1565 | 1560 | 1554 | 1546 | 1535 | 1516 | 1489 | 1455 | 1413 | 1362 | 1306 | 1245 | 1186 | 1098 | 794 | - | - |
| **36** | 1600 | 1597 | 1593 | 1589 | 1583 | 1575 | 1566 | 1551 | 1529 | 1499 | 1460 | 1414 | 1361 | 1303 | 1243 | 1181 | 1009 | - | - | - |
| **37** | 1625 | 1622 | 1617 | 1612 | 1605 | 1596 | 1584 | 1566 | 1540 | 1506 | 1464 | 1415 | 1360 | 1301 | 1243 | 1145 | 824 | - | - | - |
| **38** | 1650 | 1646 | 1641 | 1634 | 1626 | 1615 | 1602 | 1580 | 1551 | 1513 | 1467 | 1416 | 1359 | 1300 | 1233 | 1046 | - | - | - | - |
| **39** | 1675 | 1670 | 1664 | 1656 | 1646 | 1634 | 1618 | 1593 | 1559 | 1518 | 1469 | 1415 | 1358 | 1299 | 1189 | 853 | - | - | - | - |
| **40** | 1699 | 1693 | 1686 | 1677 | 1665 | 1651 | 1633 | 1604 | 1567 | 1522 | 1471 | 1415 | 1357 | 1284 | 1082 | - | - | - | - | - |
| **41** | 1722 | 1716 | 1707 | 1697 | 1684 | 1667 | 1646 | 1614 | 1573 | 1525 | 1471 | 1414 | 1354 | 1232 | 882 | - | - | - | - | - |
| **42** | 1745 | 1737 | 1728 | 1716 | 1701 | 1682 | 1658 | 1622 | 1577 | 1527 | 1471 | 1414 | 1334 | 1117 | - | - | - | - | - | - |
| **43** | 1768 | 1759 | 1748 | 1734 | 1717 | 1696 | 1669 | 1629 | 1581 | 1528 | 1471 | 1408 | 1274 | 911 | - | - | - | - | - | - |
| **44** | 1789 | 1779 | 1766 | 1751 | 1732 | 1708 | 1678 | 1634 | 1584 | 1528 | 1471 | 1381 | 1151 | - | - | - | - | - | - | - |
| **45** | 1810 | 1799 | 1784 | 1767 | 1745 | 1719 | 1686 | 1638 | 1585 | 1529 | 1461 | 1314 | 940 | - | - | - | - | - | - | - |
| **46** | 1831 | 1817 | 1801 | 1782 | 1757 | 1728 | 1692 | 1641 | 1586 | 1526 | 1427 | 1185 | - | - | - | - | - | - | - | - |
| **47** | 1850 | 1835 | 1817 | 1795 | 1768 | 1736 | 1696 | 1643 | 1585 | 1511 | 1353 | 968 | - | - | - | - | - | - | - | - |
| **48** | 1869 | 1852 | 1832 | 1807 | 1778 | 1742 | 1699 | 1643 | 1579 | 1471 | 1218 | - | - | - | - | - | - | - | - | - |
| **49** | 1886 | 1868 | 1845 | 1818 | 1786 | 1746 | 1700 | 1640 | 1559 | 1390 | 996 | - | - | - | - | - | - | - | - | - |
| **50** | 1903 | 1882 | 1857 | 1828 | 1792 | 1749 | 1699 | 1631 | 1513 | 1249 | - | - | - | - | - | - | - | - | - | - |
| **51** | 1919 | 1896 | 1868 | 1835 | 1796 | 1750 | 1694 | 1606 | 1427 | 1024 | - | - | - | - | - | - | - | - | - | - |
| **52** | 1933 | 1908 | 1878 | 1842 | 1799 | 1749 | 1682 | 1555 | 1281 | - | - | - | - | - | - | - | - | - | - | - |
| **53** | 1946 | 1918 | 1885 | 1846 | 1800 | 1744 | 1653 | 1464 | 1052 | - | - | - | - | - | - | - | - | - | - | - |
| **54** | 1958 | 1928 | 1892 | 1849 | 1799 | 1729 | 1597 | 1312 | - | - | - | - | - | - | - | - | - | - | - | - |
| **55** | 1969 | 1936 | 1896 | 1850 | 1793 | 1697 | 1500 | 1080 | - | - | - | - | - | - | - | - | - | - | - | - |
| **56** | 1978 | 1942 | 1899 | 1849 | 1776 | 1638 | 1344 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **57** | 1986 | 1946 | 1900 | 1842 | 1742 | 1536 | 1108 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **58** | 1992 | 1949 | 1898 | 1824 | 1678 | 1375 | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **59** | 1996 | 1950 | 1891 | 1786 | 1572 | 1136 | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **60** | 1999 | 1948 | 1871 | 1719 | 1407 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

Table B-3. Flow Rate Look-Up Table for Sites With 55-Mph Speed Limit *(Cont'd)*

| | 70 | 68 | 66 | 64 | 62 | 60 | 58 | 56 | 54 | 52 | 50 | 48 | 46 | 44 | 42 | 40 | 38 | 36 | 34 | 32 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **61** | 2000 | 1940 | 1831 | 1609 | 1164 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **62** | 1998 | 1919 | 1760 | 1439 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **63** | 1989 | 1876 | 1645 | 1193 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **64** | 1966 | 1801 | 1471 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **65** | 1920 | 1681 | 1221 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **66** | 1842 | 1503 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **67** | 1718 | 1250 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **68** | 1535 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **69** | 1278 | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

# APPENDIX C: CREATING QUEUE BY DATA MANIPULATION IN QUICKZONE

It was discussed in Section 9.4.1 that straight forward use of QuickZone 2 may not return queuing condition for the sites that actually have queuing condition. In this appendix, a queuing condition is created for each data set by changing the actual traffic data.

## C.1 CHANGING DEMAND FOR THE SITES WITH INTERMITTENT QUEUE

Figure C-1 shows the pattern of queue for I80WB-AM which had intermittent queue. The pattern for I80WB-PM data, I80EB-AM data, and I80EB-PM data are similar to the Figure 2.1.

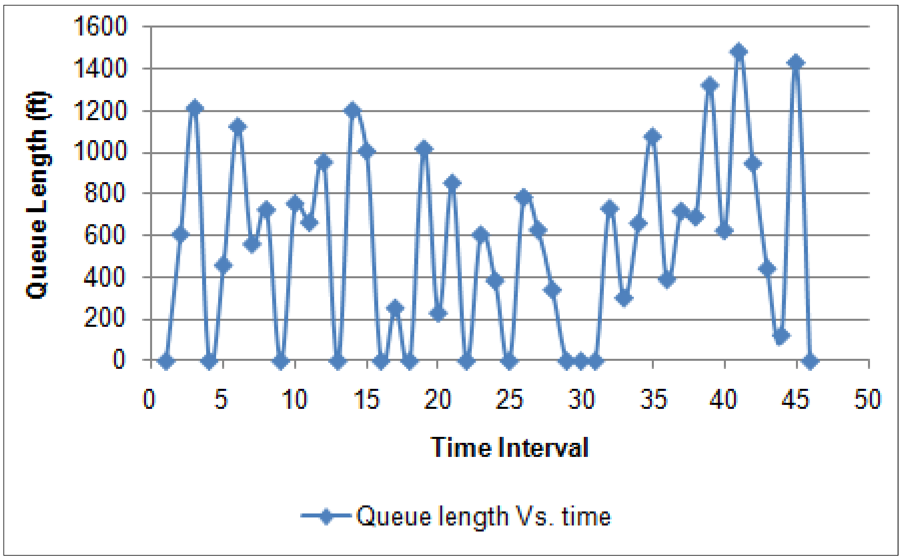


Figure C-1: Queue length variation for I80WB-AM data

For these sites, the intervals with no-queuing conditions are removed from the data so that we have a virtual congestion data. Virtual congestion period was divided into two equal subintervals. The departing volume for each subinterval is equal to the actual departing volume. In the first subinterval, queue builds up whereas during the second subinterval, it reduces and completely vanishes at the end of the second subinterval. Demand was set such that the number of vehicles in queue at the end of the first interval is equal to the twice of the average number of vehicles in queue during the entire congestion period. By this set up, the total number of vehicles in the modeled queue is expected to be equal to the one in the actual queue. Since the duration of each subinterval is less than an hour, it is scaled up to be one hour. The corresponding hourly volume and departure rate were input to the QuickZone2. Then the QuickZone2 outputs are scaled down to make the results comparable with the field data. The average queue length and total delay after scaling down are shown in Table C-1. The results show that QuickZone2 underestimated the average queue length and overestimated the total delay. Even if QuickZone2 returned reasonable results, the authors would not recommend this type of analysis for the practical purposes. Because no-queuing intervals were removed from the middle of the congestion condition, this analysis is based on some fictitious congested data. Besides, as already mentioned, the demand was manipulated to get the same number of vehicles in queue as the actual. Moreover, expanding each subinterval to one hour and scaling down the corresponding QuickZone2 outputs can cause some error. It should also be noted that the difference between the Quickzone2 outputs and the field data does not reflect the accuracy of QuickZone2 because the analyses are based on some pseudo conditions like manipulated demand and scaling up and down.

Table C-1: Comparison of QuickZone2 Outputs Based on the Fictitious Demand for the Sites with Intermittent Queue

| Data Set | Average Queue Length (ft) | | Total Delay (hr) | |
|---|---|---|---|---|
| | Field Data | QuickZone2 | Field Data | QuickZone2 |
| I80 EB, AM | 590.96 | 264.00 | 0.54 | 3.3 |
| I80 EB, PM | 1244.31 | 422.40 | 1.93 | 6 |
| I80 WB, AM | 535.20 | 264.00 | 0.30 | 2.3 |
| I80 WB, PM | 584.61 | 211.20 | 0.19 | 1.9 |

## C.2 EXPANDING QUEUING DURATION FOR THE SITE WITH MOVING QUEUE

The variation of the queue length in I39NB is shown in Figure C-2. Contrary to the sites with intermittent queue, we have a continuous congestion period.

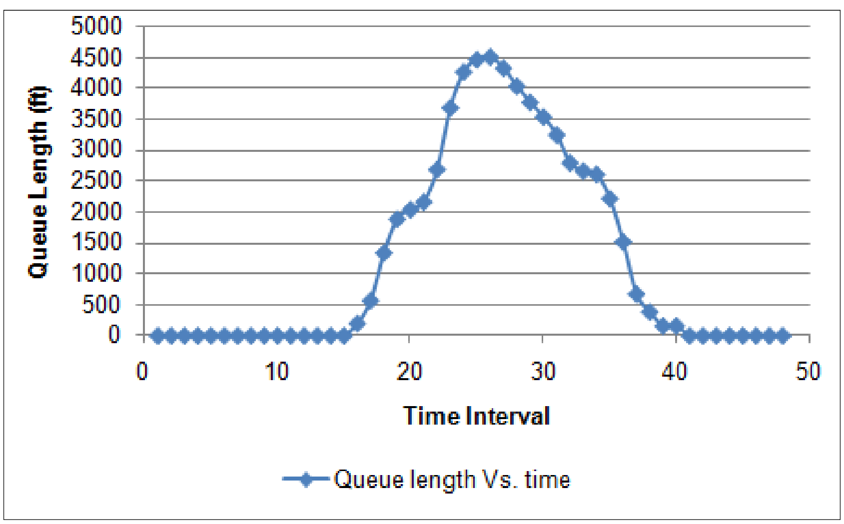


Figure C-2: Variation of the queue length for I39NB data

For I39NB, congestion period was divided into two equal subintervals. Since these subintervals are less than an hour, they were expanded. No data were deleted from the middle of the congested condition, so the hourly volume and departure rate based on the field data were input as the demand and capacity of each subinterval, respectively. The results from QuickZone2 after scaling down are shown in Table C-2.

In this case, QuickZone2also underestimated the average queue length and overestimated the total delay. Again, this method of analysis is not recommended since it includes some scaling up and down which could be a source of error. Similar to the previous case, the difference between the estimated MOE and the MOE based on the field data is not a measure of accuracy of QuickZone2 since QuickZone2 input and outputs were scaled up and down, respectively.

Table C-2: Comparison of QuickZone2 Outputs Based on the Expansion of the Congested Interval for the Site with Moving Queue

| Data Set | Average Queue Length (ft) | | Total Delay (hr) | |
|---|---|---|---|---|
| | Field Data | QuickZone2 | Field Data | QuickZone2 |
| I39NB | 2647 | 898 | 7.52 | 8.6 |

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