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Final Report

SAFETY AND CAPACITY EVALUATION OF THE INDIANA LANE MERGE SYSTEM

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16. Abstract

The entry to a work zone is critical for traffic smoothness and safety because of the discontinuation of one or more lanes. The Indiana Lane Merge System (ILMS) was designed to implement dynamic lane change control at work zone entries by encouraging drivers to switch lanes well upstream of the discontinuous lane taper, thereby reducing the number of aggressive lane changes at the taper.

At the time of this research, the ILMS had not been implemented on a large scale at rural freeway work zones. The Indiana Department of Transportation (INDOT) wanted to do a complete safety and capacity evaluation of the system before implementing it statewide. The safety of a system is usually evaluated by close examination prior to and following the installation of the system, i.e., *before and after studies*. Since crashes are rare and random occurrences, it would take a long time (usually 5-10 years) before the safety study can be completed. Another safety indicator that is used by a number of safety researchers is traffic conflicts. Unfortunately, safety benefits expressed in traffic conflicts (unlike crashes) cannot be converted to a monetary value.

In this study, a new method combining crashes and conflicts is proposed. The relative change in the number of conflicts with and without the system is first established, which is then assumed to be equivalent to the relative change in the number of crashes. This value is then multiplied by the expected number of crashes without the system to obtain the expected crash reduction using the new system. Crash prediction models without ILMS and conflict models with and without ILMS were developed. In addition, a capacity evaluation was conducted to estimate the capacity impacts of ILMS.

All the models were integrated and a detailed sensitivity analysis was performed by using a spreadsheet-based application that was developed using Visual Basic. The software program also serves as a tool for INDOT personnel to estimate the expected safety and monetary benefits for a given work zone. Sensitivity analysis results are summarized in the ILMS guidelines, which are based on daily vehicle profiles, directional distributions, and heavy vehicle profiles from rural freeways in Indiana. The estimated total benefits showed

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Safety and Capacity Evaluation of the Indiana Lane Merge System

Introduction

Construction and maintenance work zones have traditionally been hazardous locations within the highway environment. Studies show that accident rates at locations with construction zones were higher than in the periods preceding the presence of work zones. Most of the research on work zone safety has focused on segments inside work zones. Approaches to work zones have so far been neglected despite the problems with disturbed flows of traffic on approaches to work zones. This is especially visible on sections immediately before work zone locations, where one or more lanes are discontinued. Work zone entry points seem to be dangerous due to the aggressive behavior of certain drivers. Some drivers try to avoid the congested traffic conditions on the continuous lanes by approaching the work zone in the discontinued lane up to the taper where a lane change maneuver becomes difficult and risky. Such aggressive lane change maneuvers create turbulence in the traffic stream, which negatively affect performance. The effects are shock waves in the continuous lane and development of road rage, which culminate in a potentially dangerous situation, both at the merge point and within the work zone, continuing far beyond the point at which the aggressive behavior take place.

The Indiana Department of Transportation (INDOT) was aware of the contribution of work zone approaches to freeway safety and it has taken steps to improve safety at work zone locations. In addition to traditional traffic management, special traffic control devices are being installed on approaches to work zones. An important advance in this direction has been the development and installation of the Indiana Lane Merge System (ILMS), on approaches to work zones. The ILMS was developed by the Indiana Department of Transportation (INDOT) and is being evaluated by the Purdue University team as an advanced dynamic traffic control system to promote earlier merging based on the congestion levels on approaches to work zones.

Since the ILMS is a relatively new concept and has not yet been used in real construction zone environments, there is a need for evaluating the effectiveness of the new system. A simulation study was conducted for testing the effectiveness of the system by comparing average travel times and travel speeds.

The main objective of this research was to evaluate the safety and capacity effects of the ILMS in a real construction zone environment and different crash prediction models and capacity models were developed to attain this goal. The models were then integrated into a single evaluation procedure that helps INDOT engineers assess the efficacy of the ILMS for a given construction zone. The study also proposes simple guidelines for the use of the system on freeway work zones.

Findings

Contributions of the study include the following:

1. Prediction models were developed for crashes on work zone approaches for several crash types and severity levels. Two crash types were considered: rear- end crashes and merging crashes. Three severity levels were considered: property damage only (PDO) crashes, injury and fatal crashes, and total number of crashes.

2. A similar set of crash prediction models was developed for work zone segments.

3. A capacity model for predicting the capacity of rural two-lane freeway work zones with one lane closed was developed and then subsequently used to assess the impact of ILMS on capacity. In addition to ILMS, other capacity factors were investigated including rain, heavy vehicles, and law enforcement.

4. Traffic conflict frequency models were developed to predict the expected number of conflicts (per 15 minutes) as a function of several parameters: ILMS presence, congestion, traffic volume, etc.

5. A new safety evaluation method that combines the crash-based and conflict-based procedures was developed. Since conflicts are being used to determine the relative change in the number of crashes, this method is much faster than conventional before-and-after studies that utilize crashes.

6. A spreadsheet was developed using Visual Basic for automating the assessment of the economic and safety benefits from ILMS. The application accepts relevant work zone and traffic parameters and gives the expected safety and monetary benefits on the approach to work zones and on alternate routes with diverted freeway traffic.

7. After performing a detailed sensitivity analysis, a set of guidelines was developed for ILMS use. The guidelines include expected safety and total monetary benefits under various traffic volume ranges and queue constraints.

Implementation

A sensitivity analysis was performed so that a set of guidelines could be developed to help INDOT personnel decide whether or not to implement ILMS at a particular work zone. The entire analysis was based on the models developed and the assumptions made about the capacity impacts of the system. It was identified that capacity was the most crucial factor in deciding the final impacts of the system. To take care of this aspect the spreadsheet application gives the user the option to change any/all of the values in the capacity equation developed in the study.

The sensitivity analysis and the guidelines are based on the assumption of daily profile values for rural and urban weekdays and weekends. The rural profiles were built using hourly counts at telemetry stations along I-65, I-69, and I-74. Similarly, urban profiles were also constructed. However, daily profiles can change from one location to another. Even a limited change in the daily profiles can cause a significant difference in congestion levels if the rush hour volumes are close to the capacity values. Hence it is recommended that the developed evaluation tool be used with actual daily profiles for the investigated sites instead of approximate AADT values and typical daily profiles. The user has the option to input the actual daily profile values instead of the default profile values provided in the program. However, if the actual profiles are not available, then the default daily profile values may be used with caution.

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NOMENCLATURE

Symbols

$E(A_b)$	Expected number of crashes on a site before the improvement
$E(A_a)$	Expected number of crashes on a site after the improvement
$E(A_{without})$	Expected number of crashes without the new system
$E(A_{with})$	Expected number of crashes with the new system
$E(Con_{without})$	Expected number of conflicts without the new system
$E(Con_{with})$	Expected number of conflicts with the new system
$Crash_{reduction}$	Relative change in the number of crashes
$Conflict_{reduction}$	Relative change in the number of conflicts
SI	Safety impact due to the new system
K	Proportionality constant
Q(t)	Traffic volume at time t
$VMT_{without}$	Vehicles miles traveled without the system
VMT_{with}	Vehicles miles traveled when the system is in place
$Conflict_{total}$	Total number of conflicts over length L and duration D
$Conflict_{daily}$	Total number of conflicts over length L over a period of one day
L(t)	Length of the congested queue during time t
$L(t)_{max}$	Length of the maximum congested queue

$RC_{i,j,k}$	Crash reduction for the given crash category, severity level and zone number
L_c	Length of congested segment
Q_{of}	Maximum overflow queue, veh/hr
d_c	Average congested density (in-queue density), veh/km
d	Average unaffected density, veh/km
Q_{prev}	Flow in the hour before the start of congestion
A	Number of crashes on the approaches to or inside the work zone
Q	Average daily traffic, both on approaches and inside the work zone
C	Projected cost of the work/project
L	Length of the work zone
T	Duration of work, including only days when actual work was done
X_i	Explanatory variable (R_2,R_{10},R_{ON},R_{OFF},C/LT, W etc.)
β_i , δ_i	Coefficients of factors
$\frac{C}{LT}$	Proxy variable for representing the intensity of the work
W	Binary variable for work type, 1 for road works, 0 for non-road works
ρ^2	Goodness of fit measure for Negative Binomial models
α	Over dispersion parameter
Cap	Capacity of the work zone expressed in veh/hr
M	Indicator variable for ILMS
R	Indicator variable for rain
LD	Indicator variable for type of lane drop

P	Indicator variable for police presence,
H	Percentage of heavy vehicles in the traffic stream,
WS	Wind speed near the location, in km/hr.
Cap_{ideal}	Capacity under ideal conditions
f_M	Adjustment factor for Indiana Lane Merge System (ILMS)
f_R	Adjustment factor for rain
f_p	Adjustment factor for police
f_H	Adjustment factor for heavy vehicles in the traffic stream
P_H	Proportion of heavy vehicles in the traffic stream, expressed as decimal
E_H	Passenger car equivalent for heavy vehicles
CON	Expected number of conflicts (/15 min),
LS	Length of the segment under consideration for counting conflicts (m)
CG	Binary variable representing the status of congestion
R	Binary variable to indicate the presence of rain
DI	Proxy variable to represent the segment of the queue; It represents the distance of the center of the segment from the taper (m)

IMPLEMENTATION REPORT

The final report provides description of the research method, the performed field studies, and the analysis of the ILMS safety and capacity effects. Guidelines for effective use of ILMS are provided. The guidelines are supported with estimates of the expected safety and delay benefits for various traffic conditions and capacity scenarios. The report is supplemented with a spreadsheet that can be used by INDOT design personnel to estimate safety and delay benefits for individual work zones if their characteristics are considerably different from the studied ones.

The Study Advisory Committee recommends that ILMS units be used on rural freeways where congestion is expected during construction work (AADT > 40,000 veh/day for typical traffic pattern). The more elaborated guidelines developed by this research project will be introduced to the design practice in Indiana after INDOT units identified by the Study Advisory Committee complete other implementation tasks as described.

(1) Operations Support Division, ITS Program

INDOT should promote improvements of the ILMS hardware including replacement of the current traffic sensors and communication units by more reliable ones. In addition, Director of ITS Program should establish liaison between INDOT and State Police to ensure efficient ILMS enforcement.

(2) Design Division

Enforceable directives alternative to DO NOT PASS have to be considered to make lane change obligatory. The current directive does not require changing lanes. In addition, the current practice of setting the system should be changed to increase the freeway segment covered by the ILMS signage.

(3) Research Division

Capacity of selected Indiana work zones with and without ILMS should be monitored to observe changes in the ILMS impact on capacity as drivers become familiar with the new system. This task is crucial for proper implementation of the guidelines developed in this research project.

1. INTRODUCTION

The development of the automobile and the adaptations of it to move people and goods have made it a very important method of transportation. The last few decades saw a large increase in the demand for surface transport. Though this has led to a significant boom in the economy, the flip side to such a development has been the reduced safety and also the increased air and noise pollution problems. Safety on roadways has always been an important concern to both transportation engineers and road users alike. Departments of transportation (DOTs) and also private transportation agencies are engaging in active research to make the roads safer to their users. According to the National Highway Traffic Safety Administration (NHTSA) General Estimates System (GES), in 1991 there were an estimated 6.1 million police-reported crashes in the U.S. Traffic crashes impose an estimated $150 billion burden annually on the country's revenue.

Gunnarson (1996) states that roadway safety may be defined as the acceptability of risk, where risk is further described as the consequence of a crash or the probability that a crash will happen. A roadway is judged safe if its risks are judged to be acceptable. A crash is defined as an undesirable, suddenly occurring event that may result in human and material losses. Motor vehicle crashes create significant delays and also negatively impact road safety and often lead to secondary crashes as well. Several factors including degree of congestion, facility type, geometric characteristics and weather conditions may contribute to the number and severity of crashes. Freeways form a major segment in the road network system in the United States. Therefore, an attempt to increase safety on roads should include the freeways.

Construction and maintenance work zones have traditionally been hazardous locations within the highway environment. Studies show that accident rates at locations with construction zones were higher than similar periods before the work zones were set up. A number of factors have been cited as being responsible for the increase in the number of crashes at work zone locations. Pigman and Agent (1987) have cited the following factors as causes of the increase in accident rates: (a) inappropriate use of traffic control devices, (b) poor traffic management in work zones, (c) inadequate layout of work zones, and (d) a general misunderstanding of the unique problems associated with construction and maintenance work zones.

Most of the research on work zone safety has concentrated on the segment inside the work zone. Approaches to work zones have so far been neglected in spite of the problems with disturbed flows of traffic on approaches to work zones, especially on visible sections immediately before work zone locations, where one or more lanes are discontinued. Work zone entry points seem to be more dangerous due to the aggressive behavior of certain drivers. Some drivers try to avoid the congested traffic conditions on the continuous lanes by approaching the work zone in the discontinued lane up to the point where a lane change maneuver becomes difficult and risky. Such aggressive lane change maneuvers create turbulence in the traffic stream, which negatively affect performance. The effects are shock waves in the continuous lane and road rage, which culminate in potentially dangerous situations both at the merge point and within the work zone, and continue far beyond the point at which the aggressive behavior took place.

The Indiana Department of Transportation (INDOT) has realized these problems and has taken steps to improve safety in work zone locations. In addition to traditional traffic management, special traffic control devices are being installed on approaches to the work zones. The development and installation of the Indiana Lane Merge System (ILMS) has been an important advancement in this direction (Tarko et al., 1999a). The ILMS was developed by INDOT and is being evaluated by the Purdue University team as an advanced dynamic traffic control system to promote early merging based on congestion levels at approaches to work zones. The primary purposes of this research are a thorough evaluation of the ILMS and an objective measure for analyzing the efficacy of the system.

1.1 Indiana Lane Merge System

The ILMS (Figure 1.1) is believed to reduce the number of aggressive lane changes by encouraging drivers to switch lanes well upstream of the discontinuous lane taper. This allows drivers who are merging into the continuous lane to safely make the maneuver because of the increased headway between vehicles and the lower differential in speed between the two lanes. The system consists of a series of static and dynamic signs that create a variable no-passing zone in advance of the actual work zone segment.

The static sign has a white background and reads “DO NOT PASS.” The second type is a dynamic sign. The dynamic sign reads “DO NOT PASS WHEN FLASHING.”

There are three types of dynamic sign: (a) first, (b) middle and (c) last, which are placed in the order in which the traffic enters the work zone. The lights on the first dynamic sign are always on, thus creating a constant no-passing zone together with the static signs. There will usually be more than one middle sign. This arrangement guarantees that the no-passing zone will be sufficiently long (0.6-0.9 km). This minimum requirement provides law enforcement officials with a sufficient distance to stop violators before they enter the work zone.

Except for the last dynamic sign (the one farthest from the taper), all of the signs are provided with detectors, which are monitors of the traffic stream. The ILMS works on the following principle. When traffic backs up to the n^{th} sign it sends an activation signal to the $(n+1)^{st}$ sign which then gets activated (flashing lights are turned on). Thus, these signs create a *variable no-passing zone,* depending on the congestion levels on the approach. In other words, the ILMS induces drivers to merge behind the queue rather than merging into the queue. The benefits of ILMS are therefore three-fold: (a) increased safety due to a reduced number of drivers merging into the queue and (b) increased safety due to drivers merging behind the queue (fewer shockwaves). This reduces any sudden disruptions to the smooth flowing traffic. (c) less road rage caused by aggressive merging.

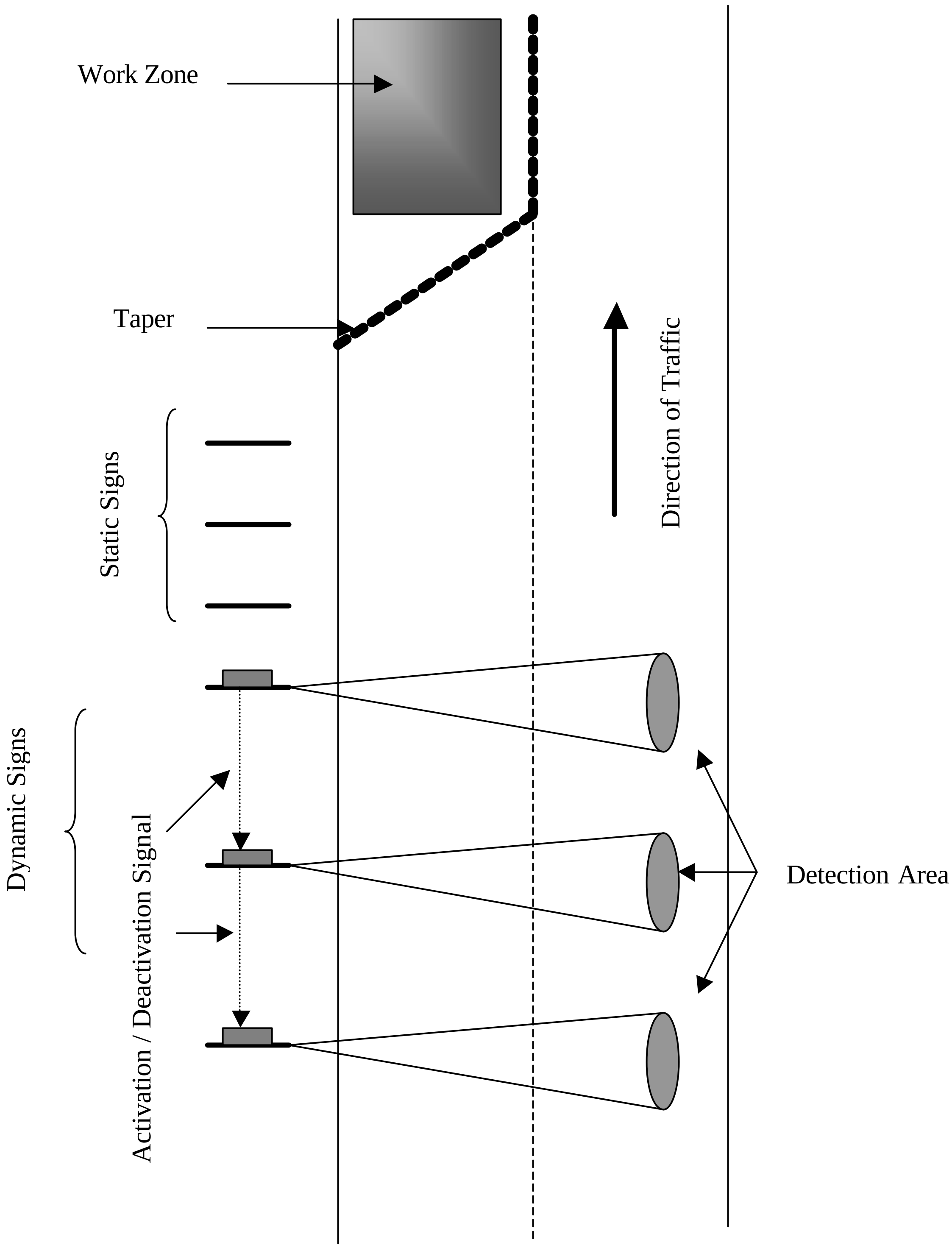

Figure 1.1: Layout of Indiana Lane Merge System

1.2 Objective of the Research

Since the ILMS is a relatively new concept and has not yet been used in a real construction zone environment, there is a need for evaluating the effectiveness of the new system. A simulation study was conducted for testing the effectiveness of the system by comparing average travel times and travel speeds (Tarko and Reddy, 1997). Although most of the analysis can be done using simulation, erroneous results may occur if the underlying phenomena are not well understood. Simulation usually works by building mathematical models or models of some of the factors involved and then analyzing their effect on the system. Simulation fails if exact, or at least, reasonable models cannot be built of the system. Safety related phenomena like crashes, belong in this category. In such cases, the researchers have to evaluate the system through direct observation in the field.

Therefore, the main objective of the research is to evaluate the safety and capacity effects of the ILMS in a real construction zone environment, which can be achieved by collecting data in real time and proposing a suitable methodology for analyzing the data. The subsequent chapters deal with the methodology proposed and discussions and analysis of results obtained.

2. METHODOLOGY

Intelligent Transportation Systems (ITS) have found their role in various areas of transportation engineering including traffic management, travel advisory systems, and traffic control. However, the effects of these systems have to be analyzed in detail before they can be implemented. Although most of the analysis can be done using simulation, erroneous results may occur if the underlying phenomenon is not well understood as was discussed in the last chapter. Usually, the safety impacts of a new system are evaluated through what is known as before-and-after studies. To evaluate safety impacts, this method uses the percent change in number of crashes before and after an improvement is made or a new system is implemented. The different methods of the before-and-after analysis are listed below.

According to the basic crash reduction method the reduction in crashes is given by:

$$\frac{E(A_b) - E(A_a)}{E(A_b)}.$$

This method just uses the absolute number of crashes at a site before and after an improvement. However, this method does not allow for temporal changes like traffic growth.

Then there is an adjusted method that allows for temporal adjustment in traffic growth. Also, a before-and-after analysis does not consider changes in other factors that can influence the number of crashes. Another problem with the before-and-after analysis is caused by the regression to mean effect (Hauer, 1997). The bias is caused by an erroneous assumption that the number of crashes in a location in the period before a new system was implemented is an unbiased estimate of what should be expected to occur in the location during an equivalent after period had the new system had been implemented. This error is basically due to the non-random sampling before improvement. There is usually a tendency to identify potentially dangerous sites and implement the improvement in these sites. However, the large number of crashes might be just due to a random fluctuation from the mean that causes over estimation of the crash rates at a location before improvement. As a result, the new system will show an increased effect whereas actually the effect is more due to fluctuation of the values around the mean. This clouds the actual impact of the investigated system (Hauer and Persaud, 1983).

The idea behind Bayesian statistics is to use a set of similar locations to improve the estimates of crash rates before improvement. The Bayesian method is used to combine the crash counts observed at the investigated location with the crash count estimate expected at this location.

The other popular method of safety analysis is the cross-sectional analysis. The safety impacts are estimated by considering similar locations with the improvement and contrasting them with locations without the improvement. Great care has to be taken to include all possible variables to get a true effect of the investigated system. In cross-sectional analysis, the assumption of distribution of crashes becomes a critical issue since regression models are being built. Since crashes are random discrete events, it becomes obvious that linear regression is not a suitable model. Linear regression is a good model when the dependent variable is continuous and normally distributed with a constant variance (homoskedasticity). But crashes being non-negative, random and discrete events, they do not satisfy the above criterion. Poisson and Negative Binomial distributions have been found to be appropriate for modeling crash data. However, Poisson models assume that mean equals variance, while the past studies have shown that crash counts display overdispersion. The overdispersion is defined as the extra Poisson variance due to variables that have not been included in the model. Therefore, the negative binomial model is a good choice since it allows the variance to differ from the mean. Venugopal and Tarko (2000), and Vogt and Bared (1998), have used Negative Binomial models for crash counts. In cross sectional analysis, thus regression models are built as a function of various traffic and roadway parameters. One of the major disadvantages of cross-sectional analysis with regards to ITS strategies is that it requires a number of sites with the system implemented. This limits the use of the method due to high costs and extended duration of analysis.

Another suggestion is to combine before-and-after analysis and cross-sectional analysis. As compared to cross-sectional analysis, all locations have accident data with and without the improvement. Crash prediction models are built for the year before and after the improvement on a number of locations. In all the studies discussed, crashes are used to measure safety. Unfortunately, crashes are rare and random and crash observations have to be taken from a sufficiently long period of time before and after the implementation of the system. Even for cross section and combined methods, only such long periods of data collection can ensure sufficient data for building statistically sound models. Such studies applied to intersections or small areas usually span over 6-10 years. Intelligent Transportation Systems (ITS), being relatively new technologies, age very fast. They might become obsolete before pilot safety studies would have been completed. Hence, it becomes imperative that safety studies for ITS systems be conducted faster, but by no means comprising their validity. This raises the need for faster, yet theoretically sound, safety impact evaluation models.

A traffic conflict occurs when one or more road users have to perform an evasive maneuver to avoid collision with another vehicle (Migletz and Glauz, 1981). An evasive action can be deceleration, weaving or any other maneuver that is useful and expedient. In such cases, evasive actions are observable. Traffic conflicts have been used as a means of analyzing traffic safety for the past two decades. Accidents are rare, while conflicts are more frequent. Also, conflicts unlike crashes are likely to be influenced more by external factors such as traffic volume and congestion (Cooper and Brown, 1986). Traffic conflicts can reduce the period of safety evaluation from years to days or weeks. A major disadvantage of traffic conflicts is that they cannot be converted to costs like crashes. Modifications to the traffic conflicts technique are proposed in this study to mitigate this weakness.

2.1 Proposed Method

The Indiana Lane Merge System (ILMS), like other ITS systems, was a new system which had not been tested for its impact on safety. The Indiana Department of

Transportation required the investigation of the system to be fast since it wanted to implement the system as soon as possible. Hence, there was a need for identifying new faster ways for evaluating the safety impact of this system faster. The basic idea is to combine the good features of both crashes and traffic conflicts in the new method. The objective of the method is to evaluate the number of crashes saved due to the new system.

$$\text{Number of crashes saved} = E(A_{\text{without}}) - E(A_{\text{with}}), \qquad (2.1)$$

$$\text{Number of crashes saved} = E(A_{\text{without}}) \cdot \text{Crash}_{\text{reduction}}, \qquad (2.2)$$

$E(A_{\text{without}})$ represents the number of crashes before the improvement is made and $\text{Crash}_{\text{reduction}}$ represents the proportionate reduction in crashes after the installation of the new system/ new improvement.

$$\text{Crash}_{\text{reduction}} = 1- [E(A_{\text{with}}) / E(A_{\text{without}})]. \qquad (2.3)$$

As mentioned before, crash data for 3-5 years before and after the improvement is needed to estimate crash reduction and this leads to a long study period extending for 6-10 years. A new proposed method combines the crash and conflict methods. The number of crashes before improvement can be converted to costs while conflicts counted before and after improvement can be used for estimating $\text{Crash}_{\text{reduction}}$. This would shorten the evaluation process. According to the new method, the number of crashes saved by the new system is estimated as

$$\text{Number of crashes saved} = E(A_{\text{without}}) \cdot \text{Conflict}_{\text{reduction}}, \qquad (2.4)$$

$$SI = E(A_{without}) \cdot Conflict_{reduction}. \tag{2.5}$$

The authors denote the number of crashes reduced by the new system as the safety impact (SI). Conflict $_{reduction}$ can be defined as proportionate reduction in the number of conflicts *before and after* the implementation of the system.

$$Conflict_{reduction} = 1- [E(Con_{with}) / E(Con_{without})]. \tag{2.6}$$

By doing so, we are making assumptions that a relationship between E(Crashes) and E(Conflicts) exists and that this relationship does not change with the implementation of the system. Also the relationship holds good only if it is of the linear form,

$$E(A) = K \cdot E(Con), \tag{2.7}$$

where E(A) is the expected number of crashes and E(Con) is the expected number of conflicts. Under such assumptions we can conclude that

$$1 – [E(A_{with}) / E(A_{without})] = 1 – [E(Con_{with}) / E(Con_{without})]. \tag{2.8}$$

2.2 Discussion of the Assumptions

Assumption 1: *The existence of a relationship between crashes and conflicts*

Before any assumptions and conclusions can be made about the existence of a linear relationship between crashes and conflicts, the author referred to some previous work done in this field. Brown (1994) evaluated the potential of traffic conflicts for road user safety studies. In this study, traffic conflicts were observed and recorded at intersections over three summer periods, and evaluated against 5-year accident records. The correlation between overall crashes and conflicts were not significant. However, when both the data was disaggregated, significance was concluded. The accidents were stratified into different categories: left-turn/opposing, left-turn/crossing, rear end, crossing, weaving and right turn. Conflicts' stratification followed the same categories. The stratification yields statistically more sound results. The correlation factors seemed to agree with results obtained by other authors (Glauz et al., 1985). The existence of a relationship between crashes and conflicts can be concluded.

Assumption 2: *A linear relationship between crashes and conflicts*

For equivalence between $\text{Crash}_{\text{reduction}}$ and $\text{Conflict}_{\text{reduction}}$ to hold, the relation between crashes and conflicts should be of type E(A) = K . E(Con) for each crash/conflict category. The zero-intercept assumption seems plausible because it is reasonable to assume that when zero conflicts are expected, then zero crashes will be expected too. This conclusion is valid under the assumption that the frequency of crashes cannot be higher than the frequency of conflicts.

Assumption 3: *The existence of the same relationship before and after improvements*

For using conflict reduction as a tool in estimating crash reduction we know that the equivalence between crash reduction and conflict reduction has to hold good. This will hold good only if the same linear relation between crashes and conflicts exists *before and after* the improvement in the system, i.e.,

$$E(A_{with}) = K \cdot [E(Con_{with})], \quad (2.8)$$

$$E(A_{without})] = K \cdot E(Con_{without})]. \quad (2.9)$$

Attempts were made to locate any studies that focused on this aspect, but attempts were not very successful.

2.3 Safety Impact Zone for ILMS

Indiana Lane Merge System, as any traffic control or management system, affects driver behavior in a bounded area. Thus, the estimation of the number of crashes and conflicts can be confined to such an area called here, *safety impact zone*. The safety impact zone of the ILMS (Figure 2.1) includes primarily the approaches to the work zone where the influence of ILMS is primarily observed. The other sections are mainly the work zone segment (inside the work area) and the alternative routes present in the network. The impacts on the driver behavior in all the zones are as follows.

Approach to the work zone: The approach to the work zone here is defined as the area where the direct influence of ILMS is observed. The approach considered in calculations is determined by the maximum permissible queue, beyond which, the users start using alternate routes and the same length is used though out the calculations. The deployment zone of ILMS (included in the approach) is determined based on the longest queue on the approach to the work zone. The approach thus includes the ILMS deployment length, a congested segment (with no ILMS) and perhaps, an uncongested segment. The safety impact for the approach is evaluated using the proposed crash-conflict procedure.

Open freeway sections: The safety impact is observed here due to a change in the traffic volumes since users opt for the alternate route. The first open freeway section starts at the off ramp used by the drivers to enter the alternate route and ends where the work zone approach starts. The second open freeway section starts at the end of the work zone segment and ends at the on-ramp where the alternative route merges with the freeway.

Work zone: Inside the work zone, the impact of the system is only observed through the changes in traffic volume (exposure level) passing through the work area with and without the system. Crash prediction models can be built using historical data of crashes inside work zone segments. The only parameter that will be affected is the traffic volume and the safety impact can be calculated using the available crash prediction equations.

Alternative routes: Here, again the safety impact is due to changes in traffic volume using the alternative routes. As in the case of freeway, the safety impact factor is calculated using the change in VMT and the number of crashes per million VMT for different highway categories. Since, ILMS was primarily going to be deployed in rural freeway work zones, the alternate routes were assumed to be rural arterials.

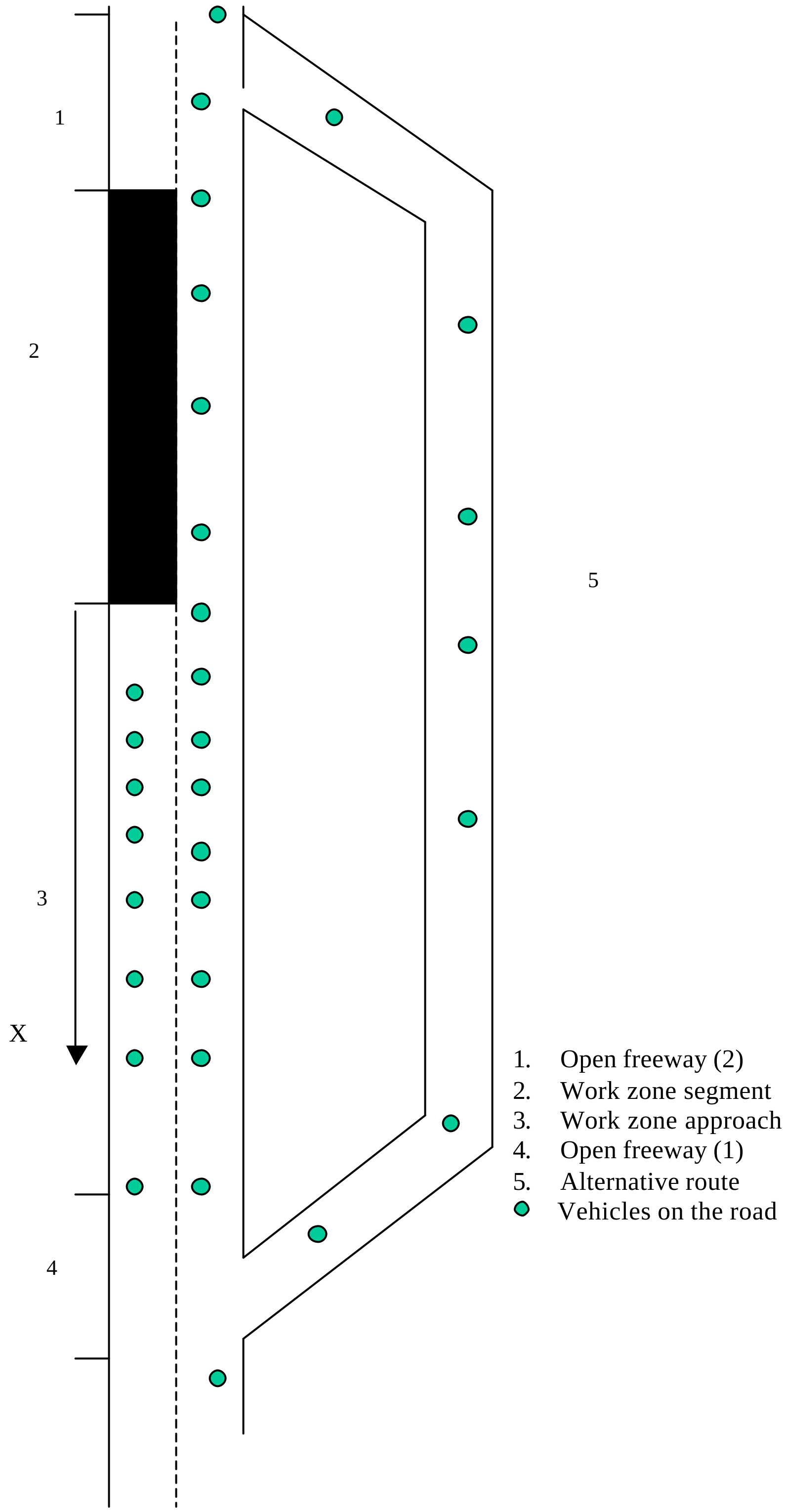

Figure 2.1: Simplified representation of impact zone of ILMS

2.4 Exact Formulation of the Safety Impact for the Different Impact Zones

2.4.1 Work Zone Approaches

The authors used the proposed crash – conflict method for estimating the safety impact factor. The safety impact using the combined method is given as:

$$SI = E(A_{\text{without}}) \cdot Con_{\text{reduction}} \, , \tag{2.10}$$

$$Con_{\text{reduction}} = 1 - [\, E(con_{\text{with}}) \,/\, E(con_{\text{without}}) \,]. \tag{2.11}$$

The expected number of crashes without ILMS will be computed using crash prediction models for approaches to work zones. Conflict models also have to be developed for approaches to work zones. Crash and conflict models will be developed for two categories: rear-end and merging crashes as they represent the predominant type of crashes on work zone approaches. Consequently, separate conflict models would be developed for braking and merging conflicts.

Both merging and braking conflicts have the same structure. The number of conflicts at a distance x can be given by

$$Conflict_{x} = f(vol, x, cong, ILMS), \tag{2.12}$$

when cong = 0, no congestion is present;

cong = 1, congestion is present;

ILMS = 0, ILMS not activated;

ILMS =1, ILMS activated;

The traffic volume on a rural freeway can be assumed to be a function of the daily profile and variations from the daily profile are minimal. Hence, volume can be assumed to be dependent at time, t. The expected number of conflicts at distance x (Fig. 2.1) at time t is given by f(x, t, cong, ILMS).

Let L stand for a total length over which the conflicts are aggregated. The length L needs to be fixed, since the aggregate count for conflicts, with and without ILMS needs to be taken over the same length L. For convenience, the authors assumed the length L to be greater than the length of the longest queue. The total duration T is taken as the duration of the work zone. Therefore, the total number of conflicts over a length L and duration D is given as

$$\text{Conflict}_{\text{total}} = \int_{o}^{D}\int_{o}^{L} \text{f (x, t, cong, ILMS) dx dt} . \qquad (2.13)$$

If we assume the same daily profile for the entire duration of work, then we can assume the total number of conflicts/day are the same and the total number of conflicts can be obtained as aggregating the number of conflicts/day. Therefore,

$$\text{Conflict}_{\text{total}} = \text{Duration} \cdot \text{Conflict}_{\text{daily}}, \qquad (2.14)$$

where Duration is expressed in days. When more than one daily profile has to be used the same procedure can still be used. The daily conflicts for each profile type can be calculated and then each can be multiplied by the corresponding duration and total number of conflicts can be estimated.

The number of conflicts per day is given by

$$\text{Conflict}_{\text{daily}} = \int_{o}^{T}\int_{o}^{L} \text{f (x, t, cong, ILMS) dx dt}. \qquad (2.15)$$

It is obvious that congestion is present only during some hours of the day. Assume congestion is present during the day between t_1 and t_2. L(t) gives the length of the congested queue, during congested period. It was assumed that $L(t)_{max}$ is less than L. Therefore, even during the congested period there will be a small length L – L(t) which is uncongested . Therefore, the number of conflicts per day is given by,

$$\mathrm{Con}_{\mathrm{without}} = \int_o^{t_1}\int_o^{L} f\ (x, t,0,0)\ dx\ dt + \int_{t_1}^{t_2}\int_o^{L(t)} f\ (x, t,1,0)\ dx\ dt + \int_{t_1}^{t_2}\int_{L(t)}^{L-L(t)} f\ (x, t,0,0)\ dx\ dt + \int_{t_2}^{T}\int_o^{L} f\ (x, t,0,0)\ dx\ dt. \qquad (2.16)$$

Assume that this represents the number of conflicts without ILMS. With ILMS, if we assume no change in capacity, then the number of conflicts/day with ILMS is given by

$$\mathrm{Con}_{\mathrm{with}} = \int_o^{t_1}\int_o^{L} f\ (x, t,0,1)\ dx\ dt + \int_{t_1}^{t'_2}\int_o^{L(t)} f\ (x, t,1,1)\ dx\ dt + \int_{t_1}^{t'_2}\int_{L(t)}^{L-L(t)} f\ (x, t,0,1)\ dx\ dt + \int_{t'_2}^{T}\int_o^{L} f\ (x, t,0,1)\ dx\ dt. \qquad (2.17)$$

It can be assumed that f(x,t,0,0) = f(x,t,0,1) since when congestion occurs, ILMS is turned off. The conflict reduction may be computed as:

$$\mathrm{Conflict}_{\mathrm{reduction}} = 1 - \left(\frac{\mathrm{Con}_{\mathrm{without}} - \mathrm{Con}_{\mathrm{with}}}{\mathrm{Con}_{\mathrm{without}}}\right), \qquad (2.18)$$

2.4.2 Open Freeway Sections and Alternate Routes

For alternate routes and highways, the safety impact is the effect of change in the exposure level (change in VMT)

$$SI = (VMT_{without} - VMT_{with}) \, . \, \text{Crash Rate.} \tag{2.19}$$

The crash rates and costs given in Table 2.1 have been taken from the course material "Estimating the Impacts of Transportation Alternatives" sponsored by USDOT and FHWA. The data has been derived from: the Fatal Accident Reporting System, the National Accident Sampling System, and the Highway Statistics. The willingness to pay methodology prescribed for valuing life-saving benefits by the U.S. Office of Management and Budget and by FHWA Technical Advisory T-7570.1 was used to compute the crash costs.

Table 2.1: Crash rates and 1999 unit costs

Area type	Road type	Crashes per million vehicle miles	Cost per crash
Urban	Interstate	1.06	$75,126
Urban	Other freeway	1.13	$80,268
Urban	Other principal arterial	5.83	$48,219
Urban	Minor arterial	5.74	$48,007
Urban	Collector	5.29	$46,595
Rural	Interstate	0.69	$129,607
Rural	Other freeway	1.48	$138,684
Rural	Other principal arterial	1.75	$120,923
Rural	Minor arterial	2.06	$116,595
Rural	Collector	3.57	$119,906

Vehicles miles traveled (VMT) is usually calculated as a function of traffic volume. The total VMT over a length L and duration D is for a flow Q is given by

$$VMT_{without} = \int_o^D \int_o^L Q_{without}(t)\, dx\, dt, \tag{2.20}$$

$$VMT_{with} = \int_o^D \int_o^L Q_{with}(t)\, dx\, dt. \tag{2.21}$$

2.4.3 Safety Impact for Work Zone Segments

The safety impact can be calculated using crash prediction models for work zone segments. The crash prediction models for different crash categories will be discussed in detail in Chapter 3. The safety impact for work zone segments is given by

$$SI_{workzonesegment} = [\int_o^T \int_o^L f_{without}(x, t)\, dx\, dt - \int_o^T \int_o^L f_{with}(x, t)\, dx\, dt\,], \tag{2.22}$$

where f(x,t) is the expected crash frequency for a given crash category.

2.5 Overall Safety Benefits

Since ILMS affects both safety and capacity, two measures of effectiveness (MOEs) are available for evaluating the final benefits.

1. Safety benefits in terms of number of crashes saved in each crash category and the total number of crashes saved.

2. The total monetary benefits due to the system that includes the delay benefits and safety benefits (in monetary terms).

If we estimate crash reduction for each crash category and for each severity level, then we can estimate the safety benefits of ILMS using the following equation.

$$\text{Safety Benefits of ILMS} = \sum_i \sum_j \sum_k (A_{i,j} RC_{i,j,k}), \qquad (2.23)$$

where:

j = crash category (Rear-end crash, merging crash, etc.),

i = severity level (PDO, fatal, injury, etc.),

k = zone number (inside the safety impact zone),

$A_{i,j}$ = Expected number of crashes for the given category and severity level,

$RC_{i,j,k}$ = crash reduction for the given crash category and severity level.

2.5.2 Estimation of Monetary Benefits

Traffic delays have become a major transportation issue, especially in major cities where increasing levels of congestion and long delays often lead to road-rage. As a result, traffic delays have been given a lot of importance in transportation planning and planners usually assign a delay cost for evaluating any new transportation system. Since the second major objective of implementing ILMS in construction zone locations is improving the capacity on work zone approaches, the efficacy of ILMS in doing so can

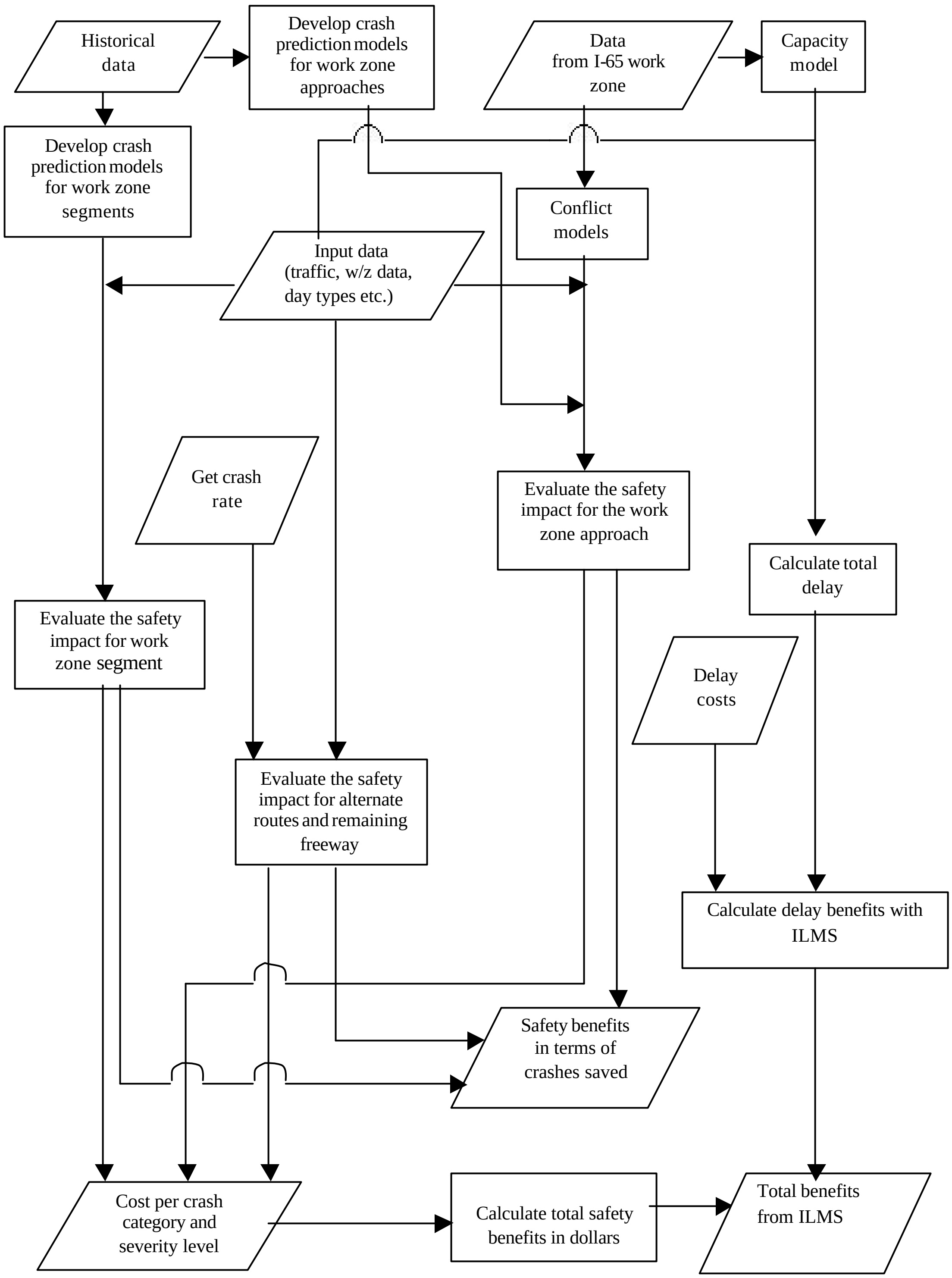

Figure 2.2: Flow chart for overall safety and delay impact evaluation of ILMS

be measured by the reduced delays after the systems have been installed. If the capacity with ILMS is more than that without ILMS, then we should see a significant reduction in delays. For estimating delays, we assume a delay cost of $ 8/hr and an average occupancy of 1.25 persons/vehicle.

Safety benefits in monetary terms can be obtained by multiplying the number of crashes saved by typical crash costs in that category. The overall benefits are obtained by putting together safety and delay benefits. The flow chart for the entire process is given in Figure 2.2.

3. CRASH PREDICTION MODELS

As explained in the previous chapter the crash prediction models form an important part of the safety study. In addition to using the models for predicting the expected crashes on construction zones without Indiana Lane Merge System, these models can be used to study safety in work zones and also analyze factors responsible for crashes at work zone locations. Since the purpose of the study was to analyze the effects of ILMS, it was imperative that crash models be developed for crashes occurring both on the approaches to work zones and inside the work zones.

Some literature was found on modeling of crashes on freeways. A study by Madanat et al. (1996) used binary logit models to predict the likelihood of vehicle crash incidents on the Borman expressway. Studies conducted by Zeeger et al. (1986) and Cleveland and Kitamura (1978) investigated the relationships between accident frequency, roadway geometry, and roadside conditions. Benekohal and Hashmi (1992) developed crash prediction models as a part of their attempt to estimate accident reduction factors on highways. Vogt and Bared (1998) developed and analyzed accident models for two lane rural roads: segments and intersections.

Even less research exists on safety relationships for work zones. Hence, separate crash prediction models were developed for both crashes on the approaches and inside the work zone segments. Also, as mentioned before, since the correlation between crashes and conflicts becomes significant only at the disaggregate level, the models were extended to each crash category and severity level. The subsequent sections deal with the various aspects of data collection, analysis, results and discussions.

3.1 Data Collection

The data used in this study is classified into three categories.

1. Work zone characteristics,
2. Crash characteristics,
3. Road and traffic characteristics.

3.1.1 Work Zone Data

The required data regarding work zones is as follows.

1. The freeway identifier (example, I-64, I-65 etc.),
2. The work zone location (mile markers at the starting and the ending points of the work zone),
3. The cost of the project,
4. The work code,
5. The duration of the work zone,

6. The length of the work zone.

The data obtained from the INDOT provided information about 393 construction projects that took place between 1993 and 1997.

3.1.2 Crash Data

Comprehensive data for work zone crashes was provided by INDOT. The crash database included not only crashes inside the work zone, but also crashes on approaches to work zones. Only crashes indicated by the investigating police officers as having occurred due to the construction activities were included in the analysis. The final comprehensive database had a total of 5025 crashes for the period from 1993 to 1997. The final database had the following data.

1. The interstate identifier,
2. The exact mile marker of the crash (distance in miles from the state line to the crash location),
3. The time of the day, the day of the week, the month and the year of the crash,
4. Number of injuries and fatalities,
5. Type of collision,
6. Geometry of collision.

Almost all the data obtained from INDOT were in a convenient format. Only the location of the crash required further processing to determine the distance between the crash location and the beginning of the work zone.

3.1.3 Road and Traffic Data

The authors retrieved relevant freeway data from a GIS database created by INDOT. Using TRANSCAD built-in filters, the authors obtained the following data.

1. The number of on and off ramps inside the work zone,
2. The number of ramps on the approach at less than 2 miles from the beginning of the work zone,
3. The number of ramps on the approach between 2 and 10 miles from the beginning of the work zone,
4. The Average Daily Traffic (ADT),
5. Percent of heavy vehicles.

The ADT values were used instead of Annual Average Daily Traffic (AADT) values to account for the seasonal and daily fluctuations in traffic volumes. The database provided by INDOT contained the AADT values and volume adjustment factors that were used to convert AADT into ADT. Percentage of heavy vehicles data was also available. Since, these are predictive models, the authors use the data available (average volume, ADT). It would be better to use actual volume during work zone conditions. But, this model was supposed to predict expected number of crashes before the work actually started. Therefore, keeping the practicality aspect in mind and also due to the unavailability of actual data, the volume during normal periods as a substitute for actual work zone volumes were used. This approximation is very appropriate with respect to this study since it is confined to rural freeway work zones. In rural freeways, an opportunity for diversion is rare and so the reduction in demand will be minimal. Even if some bias is present it is expected to be limited.

3.1.4 Final Database

An initial analysis was conducted to understand the nature of the problem that was being studied and if possible, to identify some responsible factors. As mentioned earlier, the number of rural freeway work projects available for the study numbered 393 for the period 1993-1997. Some of the minor work projects were done in conjunction with the major projects. So, the actual number of work zones was actually lesser than the number of the work projects commissioned for the period. The number of work zones available for study numbered 243. The first attempt was to check the completeness of data for the set of work zones. All work zones with missing critical data like the work zone location, duration and costs were identified and attempts were made to obtain this missing data. The work zones for which no data could be traced had to be discarded from the study.

Since the study was confined to rural freeway work zones, the next step was to remove all urban freeway work zones. It was assumed that rural freeway roads are always two lane divided highways and hence, any highways with 3 or more lanes were removed from the data base. The final database chosen for detailed analysis comprised of about 117 work zones.

It was also decided to classify the work zones based on the number of on / off ramps on the approaches to the work zone locations. Ramps were considered important to the study, because they might affect traffic patterns. For example, an off ramp close to the work zone may encourage some drivers to divert from the congested freeway to save time spent in the queue. Ramps beyond a distance of 16 kilometers (10 miles) were assumed to have no effect on the traffic. The study conducted by the authors indicated that traffic backup longer than 16 kilometers is very rare. Ramps on the approaches to work zones are classified into three categories:

1. Ramps at a distance of 16 kilometers or greater,
2. Ramps at a distance between 3.2 kilometers and 16 kilometers,
3. Ramps at a distance less than 3.2 kilometers.

The ramps at distances less than two miles and those between two and 10 miles are put in separate categories because it seems plausible that the closer the ramp the greater is its effect on the traffic.

3.2 Crash Assignment to Work Zones

Safety inside a work zone may differ from safety on approach to work zones. Barriers, reduced dimensions of cross sections, construction activities, and passing restrictions may influence traffic inside the work zone. Shock waves of congestion and aggressive lane changes influence traffic safety on the work zone approach. ILMS influences safety on the approaches to work zones and safety inside the work zones in different manners. On approaches to work zones the impact is felt at a more operational level and inside the work zone segment the impact is more due to a change in the exposure level. Therefore, the work zone-related crashes have to be separated between work zone segments and work zone approaches.

It must be pointed out that the crashes included in the analysis had been classified by the investigating police officers as work zone related. The task that the authors faced was to distribute the crashes between the work zone segments and work zone approaches and not to determine which crashes had been caused by the construction activities and which ones were not. Work zone crashes are defined henceforth as crashes that take place inside the work zone. A certain crash was assigned to a work zone if (a) the crash location fell between the beginning and end of a particular work zone and (b) the time of the crash coincided with the work zone presence.

Approach crashes are crashes that take place upstream of the work zone and are caused by the work zone presence. A certain crash was assigned to a work zone approach if (a) the crash location fell within the estimated congested segment upstream of the work zone beginning and (b) the time of the crash coincided with the work zone presence.

The length of the congested segment can be calculated using the following equation, (Tarko et al., 1998):

$$L_c = 0.92 \, . \, Q_{of} \, (d_c - d), \qquad (3.1)$$

where:

L_c = length of congested segment, km,

Q_{of} = maximum overflow queue, veh,

d_c = average congested density (in-queue density), veh/km,

d = average unaffected density, veh/km.

The maximum overflow queue is estimated using:

$$Q_{of} = \sum_{1}^{n} (Q_i - C), \tag{3.2}$$

where (Q_i) is the demand for interval *i* and *C* is the capacity of the road segment under construction. This is summed over all intervals with overflows.

An approximate estimate of density for two lane rural freeways in Indiana (Tarko et al., 1998) is used for the purpose,

$$d = 46.3 - \sqrt{2150 - 0.742\, Q_{prev}}, \tag{3.3}$$

where Q_{prev} is the flow in the hour before the start of congestion. Average congestion density on two lane approaches to work zones observed at two sites in Indiana is 71.6 veh/km. Once the approach crashes have been identified, the database is again split into two categories, depending on the type of crash. The 2 main categories, which accounted for about 85 % of the approach crashes were: (a) merging crash and (b) rear end crash.

For work zone crashes, we do not use the combined conflict-crash procedure and hence disaggregate models for crash types need not be made. However, since crash costs are different at different severity levels, models for both approach crashes and work zone crashes have to be built for different severity levels, viz. PDO, injury and fatality.

3.3 Initial Safety Study

Before the model was developed, the authors decided to do a preliminary study of crashes in work zone locations for all the interstates in Indiana. As explained earlier, the total number of work zones selected for study was 117. The total number of crashes associated with these work zones numbered 2035. This included 696 injury crashes and 33 fatal crashes. Separate analyses were conducted for approach crashes and for work zone crashes. The crashes were classified by severity level to show the gravity of the problem. The results have been shown in Table 3.1. It reveals the magnitude of the safety problem on work zone approaches. On an average, a freeway work zone in Indiana experiences about 18 crashes, 9 on the approach to the work zone and 9 inside the work zone. While the frequency of crashes on approaches to work zones is very similar to that inside work zones, the severity of crashes on approaches seems to be greater than that inside the work zone. Percentage of injury plus fatal crashes on approaches represents about 40 % of crashes on approaches to the work zone while inside the work zone it is about 30 %. These results indicate a very serious problem on approaches to work zones. This situation can be caused by typically higher speeds on work zone approaches and sudden lane change maneuvers from the discontinued lane into the continuous lane. A cursory look at the average number of crashes per work zone reveals the effect of traffic volumes on crashes. The highest average number of crashes is for interstates I-65 and I-94, which also have the highest volumes of traffic in the state. The present study hopes to identify factors that are responsible for these crashes, to construct crash prediction models and also, to suggest some measures by which safety on work zones can be enhanced.

Table 3.1: Work zone crash statistics for Indiana interstates, 1993-1997

Interstate	All crashes	Injury crashes	Fatal crashes	% of injury and fatal crashes	Number of work zones	Average per work zone
Crashes on approaches to work zones						
I-64	43	19	1	46.51	19	2.26
I-65	311	103	7	35.37	33	9.42
I-69	343	156	2	46.06	28	12.25
I-70	186	63	7	37.63	14	13.29
I-74	70	31	4	50	18	3.89
I-94	89	20	0	22.47	4	22.25
All	1042	392	21	39.64	116	8.98
Crashes inside the work zone						
Interstate	All crashes	Injury crashes	Fatal crashes	% of injury and fatal crashes	Number of work zones	Average per work zone
I-64	62	20	2	35.48	19	3.26
I-65	313	114	1	36.74	33	9.48
I-69	212	71	1	33.96	28	7.57
I-70	192	55	6	31.71	14	13.7
I-74	114	32	1	28.95	18	6.33
I-94	100	12	0	12	4	25
All	993	314	11	31.72	116	8.56

3.4 Modeling

It was decided to develop a regression model for predicting the number of crashes at a work zone location. The models predict the expected number of crashes for a single work zone, given the work zone characteristics such as ADT and duration of work. Predictive models were deemed suitable for the purpose, since our aim was to determine "crash potential" of work zones even before the work starts. Therefore, the aggregate crash prediction model was considered as the best choice available. The following variables were identified as potential factors for crashes at work zone locations.

1. *Cost of work (C),* is the total contract amount paid in $ 1000s by INDOT to the contractors for the project. It is expected to be correlated with duration and magnitude of work. This variable is used to calculate the intensity of work. The intensity of work is estimated as cost */ (duration * length of work zone)*. The intensity is believed to have a positive correlation with crashes, i.e., the crashes are supposed to increase as the intensity of work increases
2. *Average daily traffic volume (Q),* The ADT volumes are used instead of AADT volumes to take into account the daily and seasonal variability of traffic. The ADT volumes were obtained for both the approaches and inside the work zone. These values can differ due to the presence of entry or exit ramps. The volume is assumed to be an exposure-to-risk variable; any increase in volume with other parameters remaining the same will increase the number of crashes.

3. Ramps on work zone segments and on work zone approaches are represented by the following variables:

The number of ramps at a distance of 3.2 kilometers or shorter to the work zone, R_2;

The number of ramps at distances between 3.2 and 16 kilometers on approaches to the work zone, R_{10};

The number of on and off ramps on approaches or inside the work zone, R_{ON} and R_{OFF};

4. *Work zone length (L),* This measure of exposure-to-risk can be used to estimate the vehicle miles traveled. The work zone length is measured in kilometers. For all bridge work only the starting points have been given in the data, indicating very short work areas. To rectify this problem, we assume a fixed value of 0.008 kilometers (0.005 miles) for all bridge work zones and also for all work zones for which only the starting points have been specified. The length of a work zone is an exposure-to-risk variable for crashes inside work zones. However for approach crashes, the effect of length is not very apparent. The length may be used to substitute for some other variables not included in the model.

5. *Duration of work (T),* This is the number of days when a construction zone was present. The duration of work should also have an almost linear influence on crashes since other parameters remain constant. The objective of the study was to develop practical crash prediction models. The data that was available to the authors was the starting date and the ending date of the construction. But, no specific information was available about periods with actual work force in place. Since the idea was to develop

practical models for predictive purpose, the authors decided to use the total duration of construction as it is easier to predict.

6. *Type of work (W),* This indicates the type of work using letter codes as follows:

Road rehabilitation, resurfacing, other road works, J;

Bridge Rehabilitation and repair, C ;

Bridge replacement works, E ;

Roadside maintenance, landscaping etc., N;

Sign painting, signal installation etc., V ;

Interchange work, R ;

Other type, X ;

Works differ from each other by the visual and physical distraction to the traffic, by the construction equipment present in the work site, and by the number of people involved. For example, a road rehabilitation work/ road re-surfacing may be expected to have a greater effect on traffic than painting signs. The one reason for this is that resurfacing is very frequently associated with lane closures, whereas the other works usually are not. Because lane closure is believed to be a significant safety factor, especially on approaches, the road works have been classified into two categories, those with lane closure (type J) and those without (other types). If a mathematical model has to be developed, it is necessary that the independent variables have numerical values. All J works were given the value W=1 and the rest of the works were given the value W=0.

The models are being developed to predict expected number of crashes on typical rural freeway work zones. The past database used by the authors is assumed to be representative of typical rural work zones and hence they represent the typical past police presence in such locations. Of course, unusual police activity in some work zones could cause biases in results. But, unfortunately we did not have the data to incorporate these factors. The dependent variables are as follows:

Number of crashes (all types included), A;

Number of PDO crashes, PD;

Number of fatal and injury crashes, I.

3.4.1 Negative Binomial Model

It is reasonable to assume that crashes occurring on a particular roadway segment are independent of one another and that a certain mean number of crashes is characteristic of the given location and of other locations with the same properties. This particular property makes Poisson or Negative Binomial models a reasonable choice. Poisson and Negative Binomial models seem to be a better method of modeling discrete rare events such as roadway accidents (Miaou and Lum, 1993). The Negative Binomial model is superior to Poisson since the Negative Binomial model allows for extra variation caused by other variables not included in the model. This variation is represented by the overdispersion parameter. The form of the model is:

$$A = K(Q)^{ß_1}(T)^{ß_2}(L)^{ß_3}\exp\left(\sum_i d_i X_i\right), \tag{3.4}$$

where:

A = number of crashes on the approaches to or inside the work zone,

Q = average daily traffic, both on approaches and inside the work zone,

L = length of the work zone,

T = duration of work, including only days when actual work was done

K = slope parameter,

X_i = explanatory variable (R_2, R_{10}, R_{ON}, R_{OFF}, C/LT, W etc.),

β_i , δ_i = coefficients of factors.

Variables Q, T and L and their products represent the exposure to risk. Since it seems plausible that for crashes on approaches to the work zone, the length L is not an exposure-to-risk factor, it was moved inside the exponential function for approach crashes. Several models were tested changing the positions of variables, but the following model forms were found to be the most suitable.

- Approach crashes :

$$A = K(Q)^{\beta_1}(T)^{\beta_2}\exp(?_1 L + ?_2(\frac{C}{LT}) + ?_3 W + ?_4 R_2 + ?_5 R_{10} + ?_6 R_{ON} + ?_7 R_{OFF}), \tag{3.5}$$

where:

$\frac{C}{LT}$ = explanatory variable for representing the intensity of the work,

W = binary variable for work type, 1 for road works, 0 for non-road works.

It is to be noted here that, the *approach crash* models discussed here are for approach crashes that have been aggregated. Since, we need separate models for rear-end and merging crashes, the current models have been extended to include models for different crash types.

- Work zone crashes:

$$A = K(Q)^{\beta_1} (T)^{\beta_2} (L)^{\beta_3} \exp(?_1(\frac{C}{LT}) + ?_2 W + ?_3 R_{ON} + ?_4 R_{OFF}). \quad (3.6)$$

3.4.2 Modeling Process

The software used for regression analysis, in this study, was LIMDEP. As can be seen, Negative Binomial models were used for the purpose of developing both aggregate and disaggregate crash prediction models. Three important tests for an acceptable model are listed below.

1. The estimated regression for each covariate should be statistically significant, i.e., one should be able to reject the null hypothesis that the co-efficient is zero. The significance of explanatory variables was tested using the hypothesis tests using the t-tests and p-values. Only those variables, which were statistically significant, were included in the model. In this particular research, a significance level of 95 % is used for testing the statistical significance of the various covariates.
2. Engineering and intuitive judgments should be able to confirm the validity and practicality of the sign and rough magnitude of each estimated coefficient.

3. Goodness of fit was tested using the ρ^2 statistic. It is calculated in the following equation, ρ^2 = *(1 – (log likelihood/restricted log likelihood))*. Value ρ^2 grows with increasing amount of information that the model variables carry about the dependent variable (number of crashes). Value of ρ^2 equal to 1 indicates that the entire variability in number of crashes is explained by the variables included in the model. In reality, it is often impossible to include all the explanatory variables. Hence, for a Negative Binomial to be considered appropriate for all practical purposes, the ρ^2 values of 0.4 and higher are acceptable.

As explained earlier, the extra Poisson variation is indicated by the overdispersion parameter. The variance of the expected count estimate Y is the variance of the crash counts at Y reduced by the Poisson variance. In Negative Binomial model this variance is = a $\mathbf{Y^2}$**,** where α is the overdispersion parameter. Thus the estimation error is $\alpha^{1/2}$ **Y** and therefore, the relative error of estimation is $100\alpha^{1/2}$. The changes in the standard errors were checked after bringing in and taking out independent variables. The final variables that were included in the model had a significant impact on the model. Exclusion of these variables caused significant increase in the overdispersion parameter. On the other hand, inclusion of non-significant variables in the model caused very insignificant increase in the over dispersion parameter hinting that these variables might not be very representative of crashes. The slopes of independent variables were fairly stable under covariate inclusion and exclusion.

3.5 Results and Discussion: Approach Crashes

As expected, the exposure-to-risk variables, traffic volume and the duration of the work zone turned out to be significant variables. In addition to the exposure-to-risk variables, the other variables that were significant in all of the models were the intensity of work and the work type. As postulated, length of the work zone did not turn out to be a significant variable in most of the models.

The overdispersion parameter α is highly significant for all the cases (Tables 3.2 and 3.3) indicating that the selection of Negative Binomial model for regression was a good choice. The goodness of the models is measured with the ρ^2 value and the standard error of estimation of the expected count Y.

The ρ^2 value for all the models ranged from 0.28 to about 0.39 (Tables 3.2 and 3.3) indicating reasonable results. The relative error of estimation is $100\alpha^{1/2}$. Therefore, the relative error of estimation for the models developed varies between 62 % and 70 %. Such errors might be due to variables that have not been included in the model. However, the estimation error is comparable to the estimation errors obtained in other research. The standard errors, the p-values and the t-statistics for all the significant variables in all the models can be found in Tables 3.5 to 3.9.

The Figures (3.1 through 3.6) indicate that the results are unbiased. At lower values, the points are close together and as the observed value increases, the dispersion seems to be increasing. This is in complete agreement with the overdispersion values obtained from the regression analysis. Since overdispersion is greater than zero for all cases, dispersion will be small for low values of the observed dependent variable (in this case, the crashes) and it will increase with the increase in observed values. For injury and fatal crashes for both rear end and merging crashes, dispersion even at low values of observed crashes seems to be more than for other categories. This probably might be due to the fact that less factors responsible for injury crashes have been included into the model compared to the other categories.

Table 3.2: Negative Binomial models for rear end crashes

Rear End Crashes		
Model	ρ^2	α
Brake_tot = $0.00032\,(Q)^{0.803}\,(D)^{1.064}\exp(0.0017\,(C/LT) + 2.848\,(W) - 0.0346L)$	0.35	0.461
Brake_ PDO = $0.00055\,(Q)^{0.726}\,(D)^{0.979}\exp(0.0014\,(C/LT) + 2.644\,W - 0.0364\,L)$	0.30	0.388
Brake_inj = $0.00007\,(Q)^{1.121}\,(D)^{0.954}\exp(0.0022\,(C/LT) + 2.756\,W)$	0.36	0.427

Table 3.3: Negative Binomial models for merging crashes

Merging Crashes		
Model	ρ^2	α
Merge_tot = $0.000087\,(Q)^{0.731}\,(D)^{1.39}\exp(0.00095\,(C/LT) + 1.724\,W$	0.39	0.492
Merge_PDO = $0.00038\,(Q)^{0.607}\,(D)^{1.098}\exp(0.001\,(C/LT) + 1.794\,W)$	0.28	0.452
Merge_inj = $0.0000098\,(Q)^{0.966}\,(D)^{1.969}\exp(0.0002\,(C/LT) + 1.17\,W)$	0.31	0.475

Q = average Daily Traffic (in 10000 vehicles/hr),

D = duration of the construction work (in days),

L = length of the work zone segment (in km),

C = cost of the work zone (in ‘000s of dollars),

W = work type (Traveled way work =1; other = 0).

Table 3.4: Calibration parameters of crash prediction models for rear end crashes

Total Rear End Crashes				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-8.0330	1.6294	-4.9300	0.0000
Q	0.8034	0.2266	3.5446	0.0004
D	1.0640	0.2777	3.8306	0.0001
C/LT	0.0017	0.0003	6.1049	0.0000
W	2.8478	0.4208	6.7676	0.0000
L	-0.0560	0.0284	-1.9766	0.0481
α	0.4608	0.1204	3.8263	0.0001

Table 3.5: Calibration parameters of crash prediction models for merging crashes

Total Merging Crashes				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-13.8281	1.9523	-7.0828	0.0000
Q	0.9661	0.2018	4.7870	0.0000
D	1.9687	0.3337	5.8999	0.0000
C/(LT)	0.0002	0.0001	2.68047	0.0153
W	1.1686	0.5234	2.2328	0.0256
α	0.4748	0.1924	2.4674	0.02580

Table 3.6: Calibration parameters of crash prediction models for rear end injury + fatal crashes

Rear End Injury + Fatal Crashes				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-9.5806	2.8008	-3.4207	0.0000
Q	1.1279	0.3822	2.9512	0.0032
D	0.9538	0.4574	2.0851	0.0371
C/(LT)	0.0022	0.0006	3.4189	0.0006
W	2.7564	0.8191	3.3649	0.0008
α	0.4275	0.1665	2.5672	0.0102

Table 3.7: Calibration parameters of crash prediction models for merging injury + fatal crashes

Merging Injury + Fatal Crashes				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-7.8660	1.0234	-7.6860	0.0000
Q	0.6070	0.1323	4.5894	0.0000
D	1.0984	0.1785	6.1528	0.0000
C/(LT)	0.0010	0.0002	4.5543	0.0000
W	1.7944	0.3526	5.0883	0.0000
α	0.4520	0.2393	1.8888	0.05965

Table 3.8: Calibration parameters of crash prediction models for rear end PDO crashes

Rear End PDO Crashes				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-7.5036	1.6925	-4.4333	0.0000
Q	0.7261	0.2223	3.2657	0.0011
D	0.9797	0.2884	3.3974	0.0007
C/(LT)	0.0014	0.0003	5.2655	0.0000
W	2.6444	0.4203	6.2909	0.0000
L	-0.0590	0.0282	-2.0941	0.0362
α	0.3877	0.1219	3.1797	0.0015

Table 3.9: Calibration parameters of crash prediction models for merging PDO crashes

Merging PDO Crashes				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-9.5806	2.8008	-3.4207	0.0000
Q	1.1279	0.3822	2.9512	0.0032
D	0.9538	0.4574	2.0851	0.0371
C/(LT)	0.0022	0.0006	3.4189	0.0006
W	2.7564	0.8191	3.3649	0.0008
α	0.4275	0.1665	2.5672	0.0102

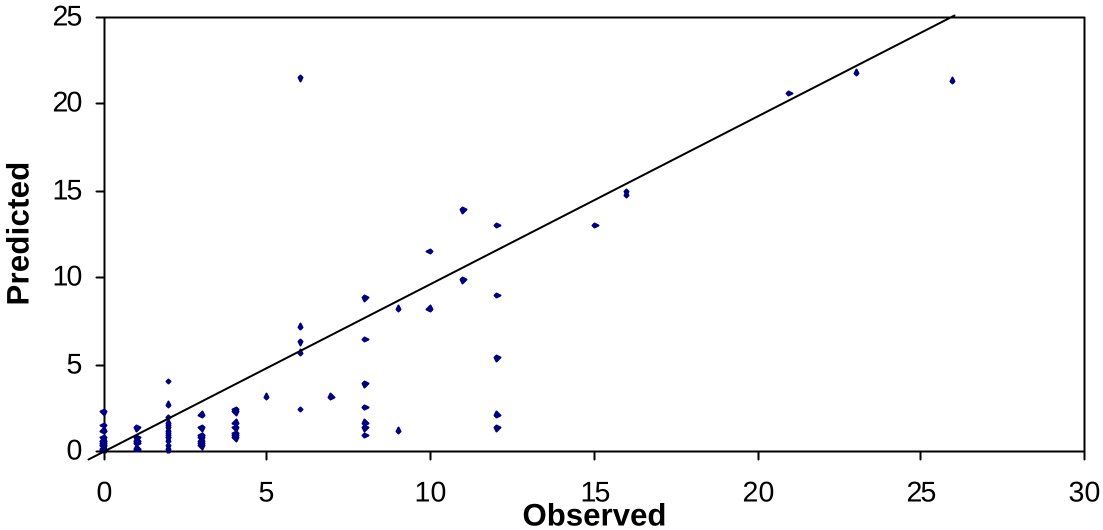

Figure 3.1: Predicted vs. observed values for rear end crashes

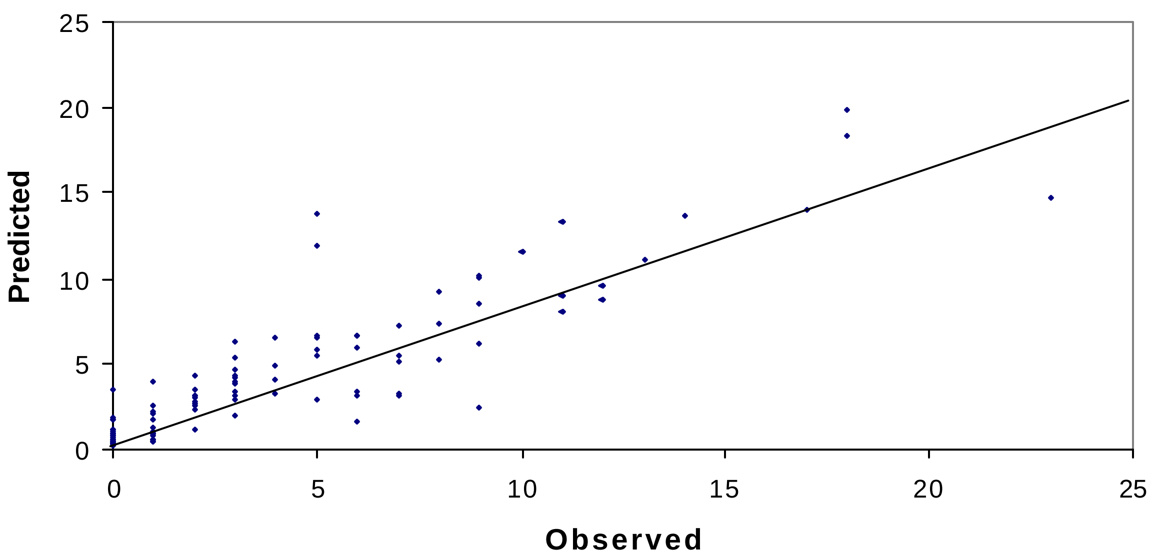

Figure 3.2: Predicted vs. observed values for merging crashes

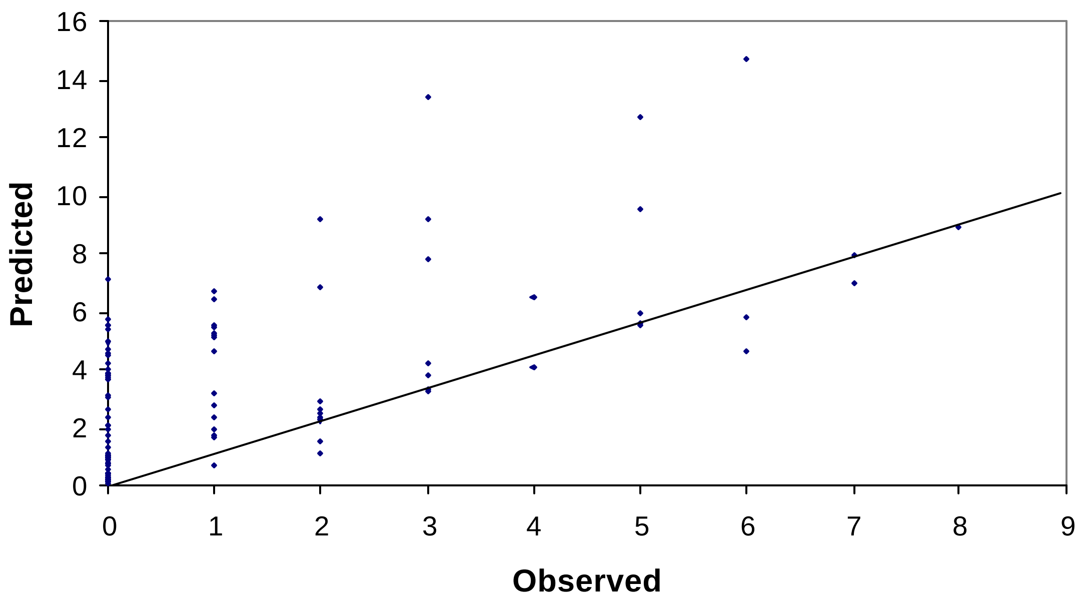

Figure 3.3: Predicted vs. observed values for rear end injury + fatal crashes

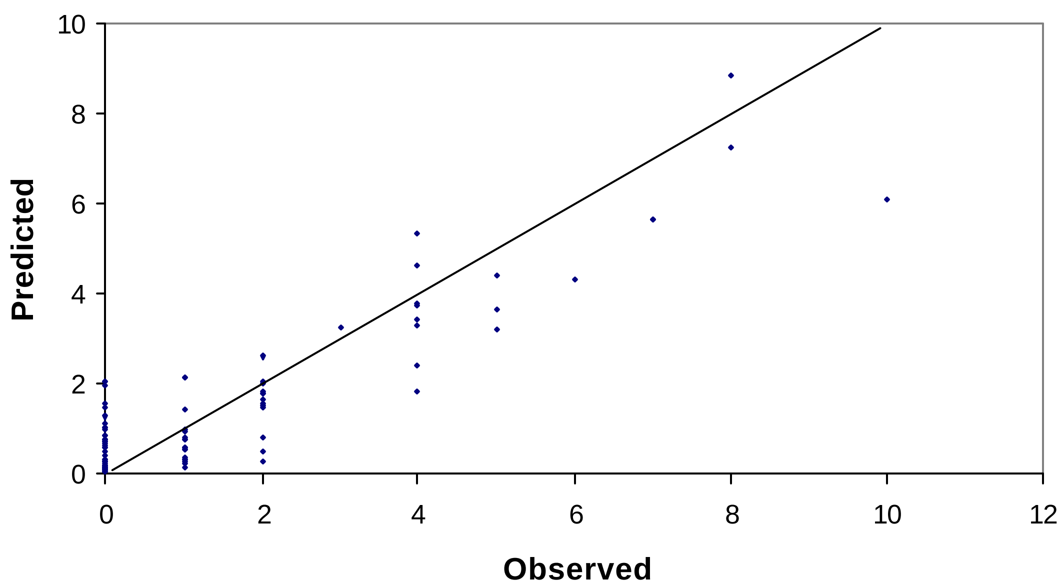

Figure 3.4: Predicted vs. observed values for merging injury + fatal crashes

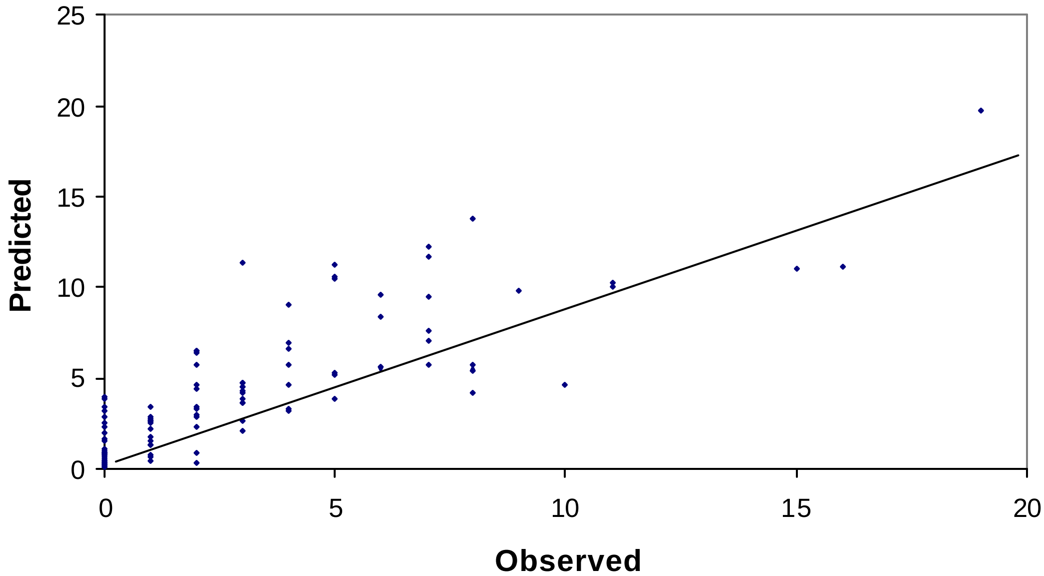

Figure 3.5: Predicted vs. observed values for rear end PDO crashes

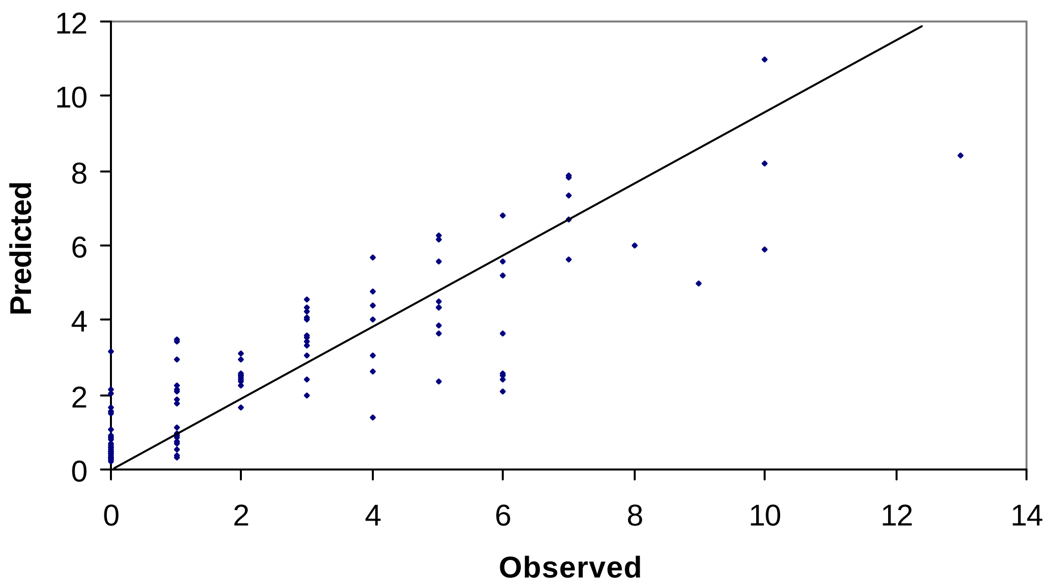

Figure 3.6: Predicted vs. observed values for merging PDO crashes

3.5.1 Discussion of Factors

As could be expected, ***traffic volumes*** turned out to be a primary factor affecting crashes during road construction. The regression parameter associated with volume is positive and very similar for both merging and rear end crashes. The value of the regression parameter ranged from a low value of 0.607 to an approximately linear value of 1.12. The relationship between crashes and conflicts seem to be approximately linear in most of the cases.

The ***length*** of the work zone was not expected to be a factor for crashes on approaches to work zones. In contradiction to our expectations, the variable turned out to be statistically very significant for two cases, total rear end crashes and rear end PDO crashes. Even more surprising and counter-intuitive was the sign of the length variable in the models. The length regression parameter was negative indicating that shorter work zones had a larger number of merging crashes than longer work zones given that other factors remain the same. The authors suspect that the work zone length may represent the effect some factors omitted in the model. One such omitted factor may be traffic management on work zones and that length can substitute for traffic management. For long work zones, usually because of their long time periods, the traffic management is more intensive than for short work zones. Hence, people tend to be more cautious.

The *duration* of work turned out to be a significant factor in all cases. For almost all the cases, the factor was approximately one. This shows that the number of crashes increases almost linearly with the duration.

The effect of *Ramps* was found to be insignificant for both the work zone segments and the work zone approaches. The exact reason for the insignificance is not known.

The *type of work* turned out to be highly significant for both merging and rear end crashes. This is an interesting and encouraging result. As mentioned earlier, work type is a binary variable with traveled way works (=1) and other works (=0). The intention was to split the entire construction database into two categories, the ones with lane drop and ones without. Usually, traveled way works are accompanied by lane drops. Indiana Lane Merge System is designed for rural freeway work zones with lane drops and when working effectively is supposed to smoothen out the traffic flow on the approaches to the work zone. Thus problems with lane drops can be solved by the use of ILMS. Since lane drop is a crucial factor influencing the expected number of crashes, the use of ILMS can reduce the crash numbers by improving traffic movement near the taper.

The *intensity of work* represented by (Cost/(Duration · Length)) turned out to be a significant factor for both rear end and merging crashes. The effect of Intensity can be attributed to the distraction presented to the road user by the construction equipment and personnel. The intensity effect also implies that short, costly work zones are more dangerous than long ones, a result very similar to the effect of length of work zone.

3.6 Results and Discussions: Work Zone Crashes

Here as expected, all the exposure to risk variables, traffic volume, duration and length turned out to be statistically significant variables. In addition to the "exposure-to-risk" variables, the other variables that were significant in all of the models were the intensity of work and the work type. The t-statistics, p-values and standard errors for all the variables are given in Tables 3.11 to 3.13.

The overdispersion parameter α is highly significant for all the cases (Table 3.10) indicating that the selection of Negative Binomial model for regression was a good choice. The goodness of the models is measured with the ρ^2 value and the standard error of estimation of the expected count Y. The ρ^2 value for all the models ranged from 0.28 to about 0.39 (Table 3.10) indicating reasonable results. The relative error of estimation for the models developed varies between 74 % and 89 %. Errors are not very insignificant; however, such errors might be due to unknown variables that could not been included in the model. The Figures (3.7 through 3.9) indicate that the results are more or less unbiased. At lower values, the points are close together and as the observed value increases, the dispersion seems to be increasing which is in complete agreement with the overdispersion values obtained from the regression analysis. For injury and fatal crashes for work zone crashes, dispersion seems to be more than for other categories.

This probably might be due to the fact that less factors responsible for injury crashes have been included into the model compared to the other categories.

Table 3.10: Negative Binomial models for work zone crashes

Work Zone Segments		
Model	ρ^2	α
$A = 0.00217\,(Q)^{1.1588}\,(T)^{0.5126}\,(L)^{0.760}\,\exp(0.1615(C/LT) + 2.308(W))$	0.33	0.5593
$I = 0.00812\,(Q)^{1.0497}\,(T)^{0.5263}\,(L)^{0.8531}\,(C/LT)^{0.3743}$	0.20	0.7940
$D = 0.0008\,(Q)^{1.1901}\,(T)^{0.4952}\,(L)^{0.9956}\,\exp(0.1851(C/LT) + 2.3279(W))$	0.33	0.7003

where:

A = total number of crashes,

I = number of injury and fatal crashes,

D = number of PDO crashes,

Q = average daily traffic, in 10000's

L = length of the work zone segment in km,

T = duration of the project in days,

C = cost of the construction project in \$ 1000's,

C/LT = proxy variable for intensity of work,

W = work type; $W = 1$ for j type work, $W = 0$ for other types,

ρ^2 = (1- (log likelihood/restricted log likelihood)).

Table 3.11: Calibration parameters of crash prediction models for all work zone crashes

Total Crashes Inside the Work Zone				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-6.1332	1.3514	-4.5384	0.0000
Q	1.1588	0.2207	5.2501	0.0000
T	0.5126	0.2425	2.1138	0.0345
L	0.7601	0.1573	4.8331	0.0000
C/(LT)	0.1615	0.0558	2.8969	0.0038
W	2.3080	0.3128	7.3776	0.0000
α	0.5593	0.1437	3.8926	0.0001

Table 3.12: Calibration parameters of crash prediction models for work zone injury + fatal crashes

Injury and Fatal Crashes Inside the Work Zone				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-4.8139	1.8217	-2.6425	0.0082
Q	1.0497	0.3996	2.627	0.0086
T	0.5263	0.298	1.7664	0.0773
L	0.8531	0.2358	3.6177	0.0003
C/(LT)	0.3743	0.1832	2.0427	0.0411
α	0.7940	0.2311	3.4366	0.0006

Table 3.13: Calibration parameters of crash prediction models for work zone PDO crashes

PDO Crashes Inside the Work Zone				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-7.131	1.5828	-4.5053	0.0000
Q	1.1901	0.2211	5.3825	0.0000
T	0.4952	0.2872	1.7241	0.0847
L	0.9956	0.2363	4.213	0.0000
C/(LT)	0.1851	0.0833	2.223	0.0262
W	2.3279	0.5064	4.597	0.0000
α	0.7003	0.1936	3.617	0.0003

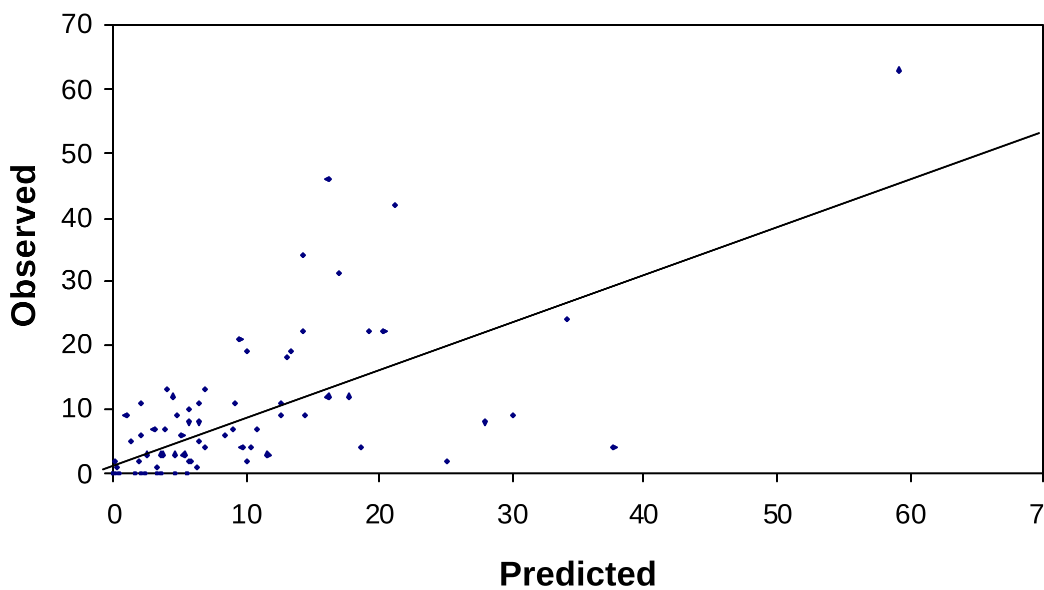

Figure 3.7: Predicted vs. observed values for all work zone crashes

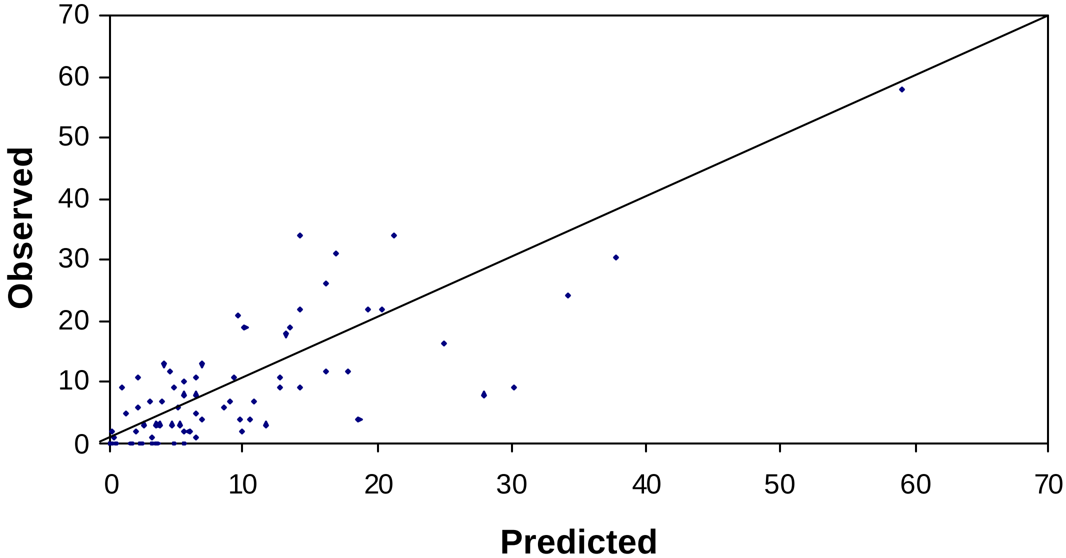

Figure 3.8: Predicted vs. observed values for work zone PDO crashes

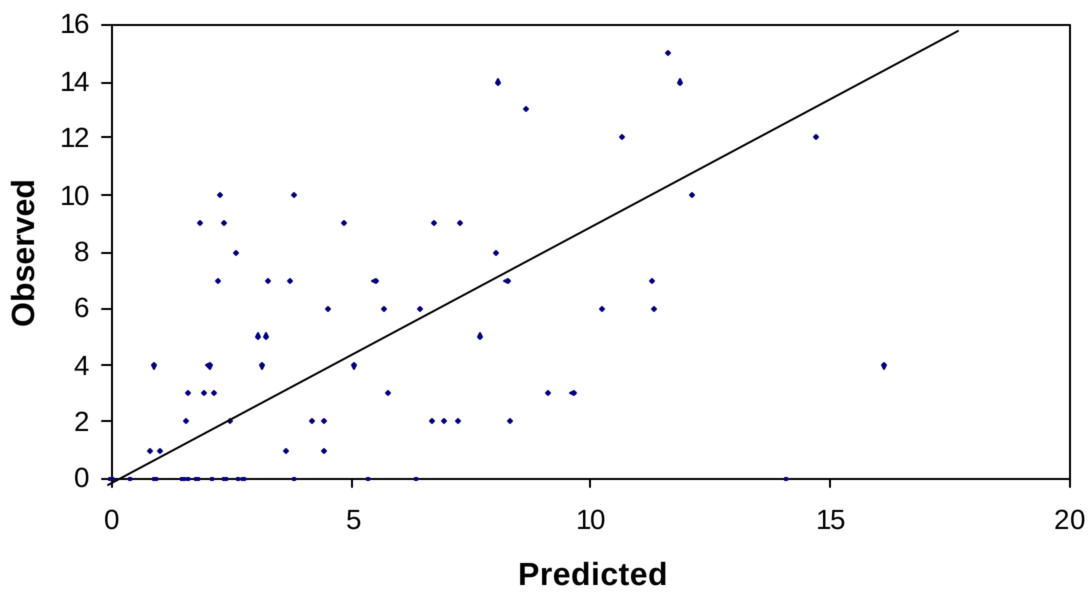

Figure 3.9: Predicted vs. observed values for work zone injury + fatal crashes

3.6.1 Discussion of Factors

As was with approach crashes, *traffic volume* was a statistically significant factor affecting work zone crashes during road construction. The regression parameter associated with volume is positive and very similar for all severity levels of work zone crashes. The value of the regression parameter ranged from a 1.05 to a 1.19 (tables 3.11 to tables 3.13). Since all the values are close to a perfectly linear value of one, the relationship between crashes and conflicts seem to be linear in all the cases. This is not a very surprising result, since volume was assumed to be an exposure-to-risk variable for work zone crashes.

The *length* of the work zone was also another exposure to risk variable for work zone crashes. Hence, as expected, the variable turned out to be statistically very significant for all categories of work zone crashes. The regression parameter associated with length varied from 0.76 to 0.99.

The *duration* of work turned out to be a significant factor in all cases. But contrary to expectations, for all categories, the regression parameter associated with duration was close to 0.5. In other words, the numbers of crashes do not increase linearly with the duration, but tend to taper off after some time. In other words, the marginal increment in crashes with the increment in the duration of the work zone, tends to decrease as the duration of work zone keeps increasing. The result was a bit surprising. Two reasons could be hypothesized for this non-linear behavior. One reason could be that volume is highly correlated to some variables that have not been included in the model. Another reason might be that if the duration of a work zone is large, the familiarity of the drivers using the highway keeps increasing with time. More familiarity usually leads to safer driving and hence fewer accidents.

The *type of work* turned out to be highly significant all types of work zone crashes. The type of work determines the construction equipment and personnel presence inside the work zone and the temporary lane closures. Usually, long road work zones have often one of the roadways closed and both traffic directions use the remaining roadway equipped with a separation. This causes discomfort to some drivers and may increase the risk of accidents.

The *intensity of work* represented by (cost/(duration x length)) turned out to be a significant factor for both rear end and merging crashes. The effect of intensity can be attributed to the distraction presented to the road user by the construction equipment and personnel.

3.7 Closure Remarks

Mathematical models were developed in this study to predict the expected number of crashes in freeway work zones, both inside the work zone segments and on approaches to the work zone. As expected, the traffic volume, length and duration of work turned out to be significant factors. In addition, the cost of work zone (as a measure of intensity of work) and the work type were also critical factors of safety inside work zones. Since all the models are built for cases without ILMS, they can be used effectively in the safety evaluation of ILMS. The crash prediction models for approach crashes can be used in the combined "crash conflict" method for estimating the number of crashes saved in the immediate zone.

$$SI = E(A_b) \cdot Con_{reduction}. \qquad (3.7)$$

As mentioned in chapter 2, the crash prediction models for work zone crashes can be utilized in evaluating the safety impact of ILMS on work zone segments

$$SI = [\int_o^D \int_o^L f(Q_{without}(t), x, t)\, dx\, dt - \int_o^D \int_o^L f(Q_{with}(t), x, t)\, dx\, dt]. \qquad (3.8)$$

In addition to evaluating the safety effects of ILMS the models can be used for analyzing safety on work zone locations. These models are also very useful in optimizing work zone schedules and for deciding better pavement management strategies. The study shows that a long single work zone is better than two short work zones. Thus, if there are two similar work zones close to each other, it is safer to combine the two work zones than do them individually. The frequency of pavement rehabilitation is often decided after taking into account the benefits and costs of the project. The cost of crashes can be included as a cost in the *benefit-cost* analysis for more efficient pavement management strategies.

The crash prediction models can be utilized for purposes other than evaluation of Indiana Lane Merge System. With the advent of new ITS technologies and user information systems like the Advanced Traffic Information Systems (ATIS), the user can be given information about the crash potential of a particular work zone, so that, a safer route may be adopted. The present study can be extended to predict the probability of a crash occurring in a work zone during a particular hour given the work zone and temporal traffic characteristics. Such models will be useful in giving dynamic real-time information to the user about the *crash potential* of the work zone during that hour and thus will make re-routing strategies more fluent. Such models would give a more comprehensive evaluation of work zone safety to address safety issue for intelligent or smart work zones.

4. INVESTIGATION OF CAPACITY EFFECT OF ILMS

The 1997 Highway Capacity Manual (HCM) defines capacity as “the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions.” The Highway Capacity Manual considers various conditions that influence capacity. They include (a) roadway conditions such as lane width, shoulder width and clearance, design speed and the type of facility and its development environment, (b) traffic conditions such as percentage of heavy vehicles and also directional distributions of traffic, (c) control conditions such as traffic signal phasing, metering and filtering. Other adverse factors of capacity are poor weather and also the occurrence of incidents.

Apart from the main investigated factor -- ILMS, one of the other major factors affecting the capacity on freeway is the presence of heavy vehicles (trucks) in the traffic stream. Trucks usually travel slower and keep longer gaps as compared to other vehicles. As a result, the capacity of freeway drops as the proportion of trucks in the traffic mix increases. Although the effect has been investigated on freeways, there have been no attempts to quantify this effect in work zones.

The other factors that were investigated include the effects of harsh weather conditions like heavy rain and strong winds on work zone capacity. These factors generally have a negative effect on capacity. Researchers have investigated the effect of rains, but the effects have not been well quantified (HCM, 1997). It was concluded that during rainy conditions the capacity goes down by about 10 to 20 %. The authors wanted to quantify the effect of rain on capacity. Another, major weather factor which has not been given serious consideration is the wind effect. During heavy winds, the drivers often have a hard time keeping their automobiles on the road. This effect is more pronounced in the case of truck drivers. Therefore, during heavy winds people tend to be more safe and drive slower. This must have a negative impact on capacity.

Traffic in major rural freeway work zones is managed through crossovers. One roadway is closed to enable construction activities while traffic is re-directed onto the open roadway. Re-directed traffic has to merge into one lane before crossing the median. The authors investigated the capacity of work zone approaches with left-lane closures and right lane closures to check if there is any difference.

Indiana Department of Transportation (INDOT) is currently investigating the potential benefits of police presence in work zones. Since the presence of police can have a significant impact on driver behavior, it was decided to analyze the effect of police presence on capacity.

4.1 Background

The 1997 Highway Capacity Manual discusses the effect of work zones on freeway capacity. HCM notes that the work zone capacities depend on the nature of work, the number and size equipment at site, and the location of equipment and crews with respect to moving lanes of traffic. The manual provides a summary of observed capacities for some typical construction and maintenance operations (Table 4.1).

Hall and Duah (1991) attempted to re-define capacity in their paper. Since the 1987 HCM defines capacity as the maximum hourly rate of vehicles, it sometimes gives a wrong connotation to capacity as the absolute maximum flow observed. The authors argue that capacity should be a rate of flow that can be repeatedly achieved under unchanged conditions. Capacity is often measured as a rate of vehicles discharging from the queue. Hall and Duah (1991) argue that capacity should be measured at the bottleneck downstream of the queue. They also observed a drop in capacity once the queue has formed. They concluded that there is a higher capacity prior to queue formation and that more emphasis should be given to the pre-queue capacity.

Several authors have measured and analyzed capacities of work zones. Richards and Dudek (1979) measured capacity of urban freeway work zones. They measured traffic volumes when queues were formed upstream from the lane closures and thus essentially represented either the capacities of the bottlenecks created by the lane closures or the effects of the driver distraction presented by the work zone machinery. The authors compared the capacities of a work location with and without the work crew.

One of the early works on the effect of lane closures on freeway capacity was done by Nemeth and Rouphail (1982) using a microscopic computer simulation model. A car following rule controlled vehicle movement. The merging behavior was controlled by traffic control devices and driver's preference for early or delayed merge.

There were a number of works that analyzed the effect of trucks on highway capacity. Cunagin and Chang (1982) analyzed the effect of trucks on freeway vehicle headways under off-peak flow conditions. They concluded that the presence of trucks in the traffic stream is accompanied by an increase in the mean headway and this would significantly reduce capacity. Truck drivers seem to keep more space in the front than do automobile drivers.

Studies have shown that bad weather conditions adversely affect capacity. Even though quantitative information is sparse, Jones and Goolsby (1970) found that presence of rain reduced capacity by about 14 percent. HCM (1997) states that it is typical to find about 10 to 20 percent reductions in capacity due to rain and even higher reductions are possible. HCM also recommends that these effects be considered in any facility analysis, particularly when such conditions are common.

Table 4.1: Measured average work zone capacities

Number of lanes normal	Number of lanes open	Average capacity (veh/hr)
3	1	1,170
2	1	1,340
5	2	1,370
4	2	1,480
3	2	1,490
4	3	1,520

Source: 1997 Highway Capacity Manual

4.2 Data Collection

The experimental test bed for the study was chosen to be an I-65 work zone near West Lafayette, Indiana. The work zone segment extended approximately from MP 178 near US-43 interchange to MP 193 near US-231 interchange. It was a typical 4-lane rural freeway work zone. The work started in the end of March 1999 and continued for 5 months till July 1999. The work involved rehabilitation and resurfacing of the pavement and hence, it involved intensive activity and presence of construction equipment and personnel. The daily traffic ranged from about 24,000 veh/day in the weekdays (Monday through Thursday) to about 42,000 veh/day during the weekends.

Congestion was observed only during the weekends, on Fridays on the I-65 southbound approach and Sundays on the I-65 northbound approach. Congestion in both the directions typically started around 3:00 PM and ended between 6:00 and 7:00 PM. The I-65 work zone could be considered a quite typical long-term construction site.

The main data required for the analysis was traffic volume and speed data. Traffic speed and volume data was collected for a three-month period from April 1999 to July 1999. The data was collected using a series of taped down loop detectors installed on both the entrances to the work zone (Figure 4.1). Volume and speed detectors were installed in the open lane, close to the tapered sections. In addition, a set of speed detectors was installed at the beginning of the tapered section. The distance between the first and the second set of detectors was approximately 0.40 kilometers (400 m).

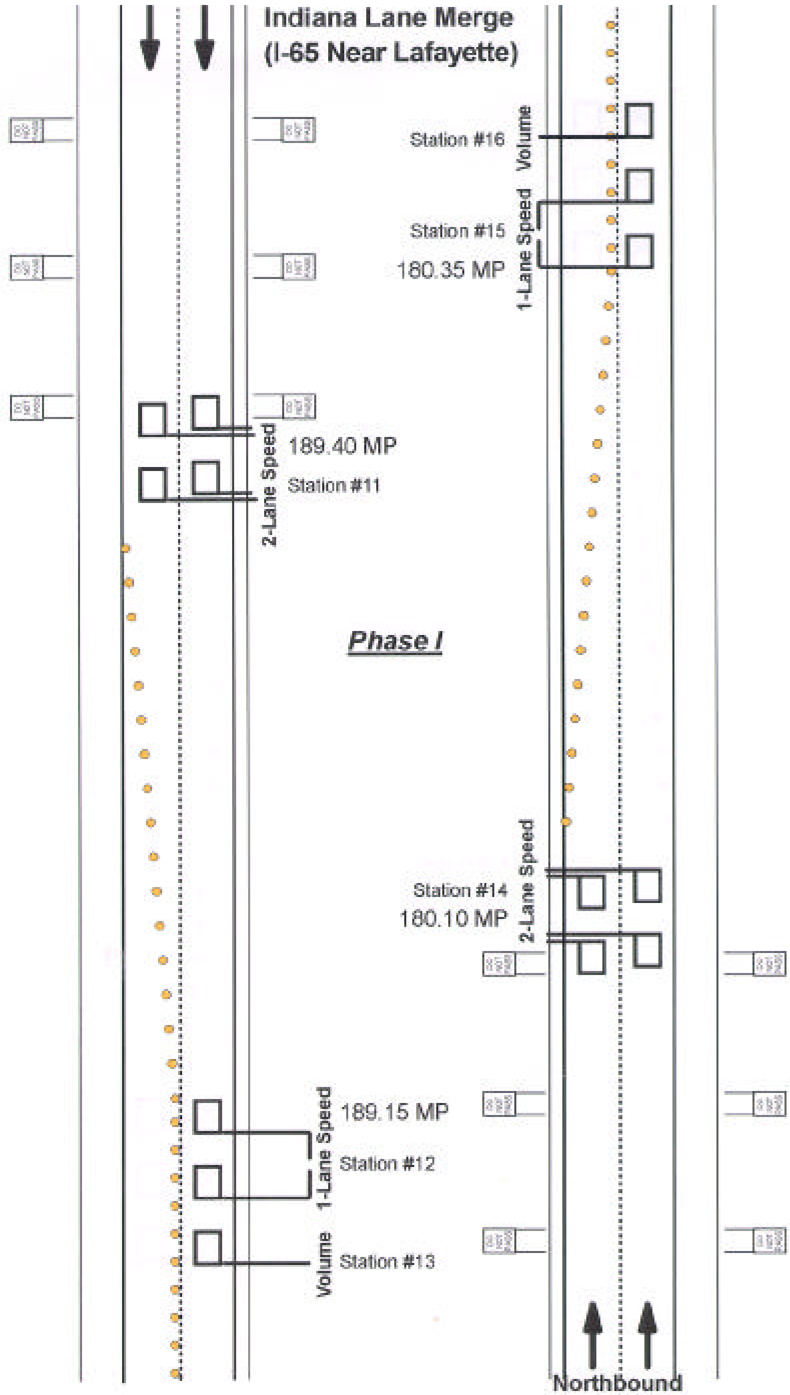

Figure 4.1: The layout of loop detectors for volume and speed measurements

The observation for the study is defined as a vector of traffic, roadway and weather characteristics associated with an interval when traffic was congested on the given work zone approach. The following characteristics are included:

1. Traffic volume (veh/hr),
2. Percentage of vehicles,
3. Presence of ILMS,
4. Presence of rain,
5. Wind speed (km/hr),
6. Type of lane drop (left / right),
7. Presence of police.

The traffic data collected are for a three-month period and therefore it is critical to identify capacity values among the entire set of observations. Identifying capacity conditions from the data sample is one of the most crucial steps in the analysis process. Hall and Duah (1991) had observed the presence of two capacity regimes, a higher capacity prior to the formation of queue and a capacity drop just after the queue is formed. It was decided to use the latter, since this capacity lasts for a longer time as compared to the higher pre-queue capacity. We used speeds measured at the spots where congested traffic was expected after the queue had formed. A sudden drop of speed indicated the capacity conditions. Capacity was calculated as the average value of vehicle volumes observed during capacity conditions. The determination of capacity is illustrated with the volume-speed graph obtained as a part of the analysis (Figure 4.2). The volumes were measured at the taper (expected bottleneck). The graph shows a rapid drop of speed at this location, then an extended period with lower speeds and then a sudden rise of speeds that marks the end of capacity conditions. The presented speeds during capacity conditions do not reflect the actual speeds properly since, the lowest speed bin was set at 0-30 mi/h.

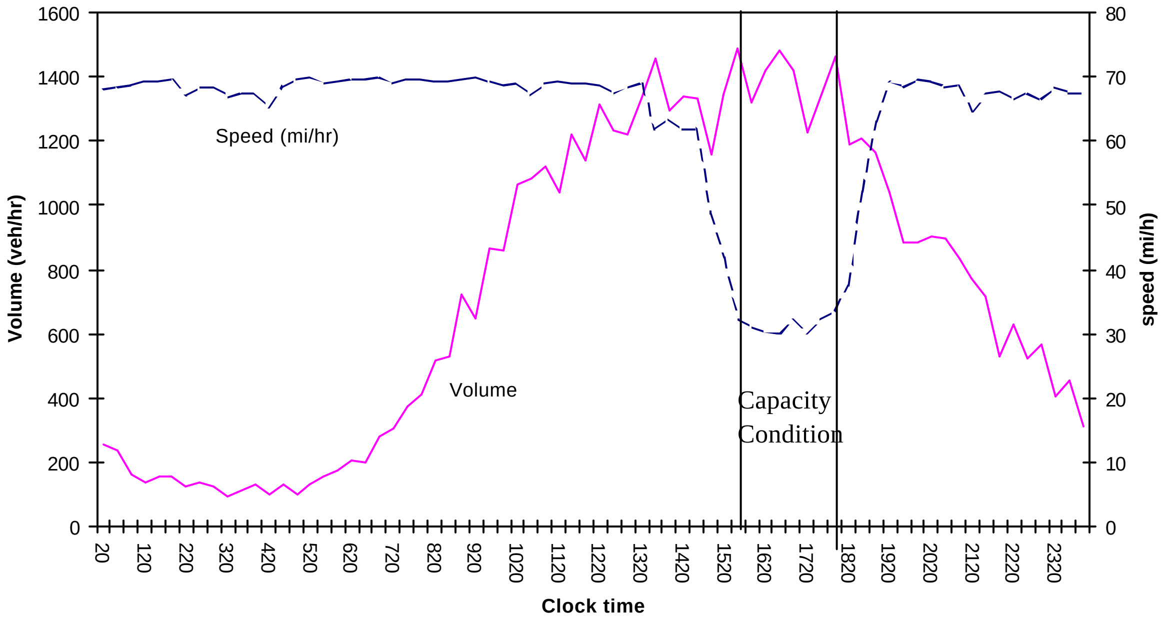

Figure 4.2: Volume – speed graph for identifying capacity

As has been mentioned, an observation includes traffic volume measurements in 20- minute intervals. Once all the capacity volumes had been identified, additional data including weather, heavy vehicle percentages, wind speed, and the status of ILMS were collected to extend the number of investigated characteristics. The loop detectors installed for volume and speed measurements also had the capability of classifying vehicles according to their length. This feature was used to obtain the percentage of heavy vehicles. The weather information about rain and wind speed during the study period were obtained from the Earth and Atmospheric Sciences department at Purdue University, West Lafayette. The weather station was located 5 miles from the I-65 work zone. The traffic data was collected at the selected site without ILMS. Then, ILMS was turned on and about two weeks were given for people to adjust to the new system. The data was then compared. After removing incomplete observations, the final sample had 182 observations.

The analysis carried out in this study was two-fold. A preliminary capacity investigation was conducted in an attempt to confirm Hall and Duah's (1991) findings and to study the various aspects of the volume-speed profiles in capacity conditions. Then capacity prediction models were built to study the effects of ILMS and other characteristics.

4.3 Preliminary Capacity Analysis

Table 4.2 shows results of a preliminary analysis. The average value of capacity obtained for the I-65 work zone, 1320 veh/hr, is very close to the capacity value obtained for a one lane dropped highway, 1340 (Table 4.1). This confirms that the I-65 work zone was a typical work zone and that the results obtained there should represent general trends. Table 4.2 indicates that all variables except police are well represented in the sample. For the police variable, there are only 13 observations with police presence and 169 observations without police presence. This might result in an insignificant estimation of the police effect if it was weak.

From Figures 4.3 and 4.4, it is evident that capacity values are not stable. The reasons for the capacity instability are not fully clear. However, from the field observations and from the video data collected, the authors hypothesized the following reasons. The valleys in the capacity curves corresponded to the stop-and-go conditions. The sequence of breakdowns may be caused by aggressive merging at the taper that leads to backward congestion waves. The authors have noticed a particular truck behavior that often coincides with breakdowns. Two trucks move side by side and block both the lanes to prevent being passed by other vehicles. The trucks continue in the same fashion until the taper, where the truck in the discontinued lane merges into the open lane. The vehicles behind the trucks form two lanes. Those in the closed lane are left with no choice but to aggressively merge at the taper. This leads to a very dense platoon of traffic entering the work zone. When inside the work zone, vehicles spread along to regain safe distances. This creates a backward shock wave eventually leading to stop-and-go conditions on the approach.

Table 4.2. Descriptive statistics of the investigated characteristics

Continuous variables				
Variable	Maximum	Minimum	Average	Standard deviation
Capacity , veh/hr	1591	1000	1320.57	118.99
Percentage of heavy vehicles	24.32	0.65	8.85	3.97
Wind speed, km/hr	24	3.2	7.05	3.70

Discrete variables				
		Counts from the sample		
Value	Rain	ILMS	Type of lane drop	Police
1	59	62	71	13
0	123	120	111	169

In the introduction to this paper, the authors had mentioned findings by Hall and Duah (1991). In their paper, Hall and Duah had discussed the capacity drop observed at the formation of the queue. In the current study, queuing conditions were observed on 13 days. Out of the 13 cases, only in two cases (Figures 4.3 and 4.4) could any capacity drop can be concluded. In both the figures, a small capacity drop just after the formation of the queue is noticeable. A rather fast buildup of volume before the breakdown conditions does not create conditions for an extended pre-queue capacity. Extended periods of pre-queue capacity could help reduce the capacity drop if such occurs.

Although, it was not possible to convincingly substantiate results obtained by Hall and Duah (1991), the identification of the existence of a higher capacity before queue formation lends more weight to the ideas proposed by Hall and Duah and calls for further research to be done on this interesting phenomenon.

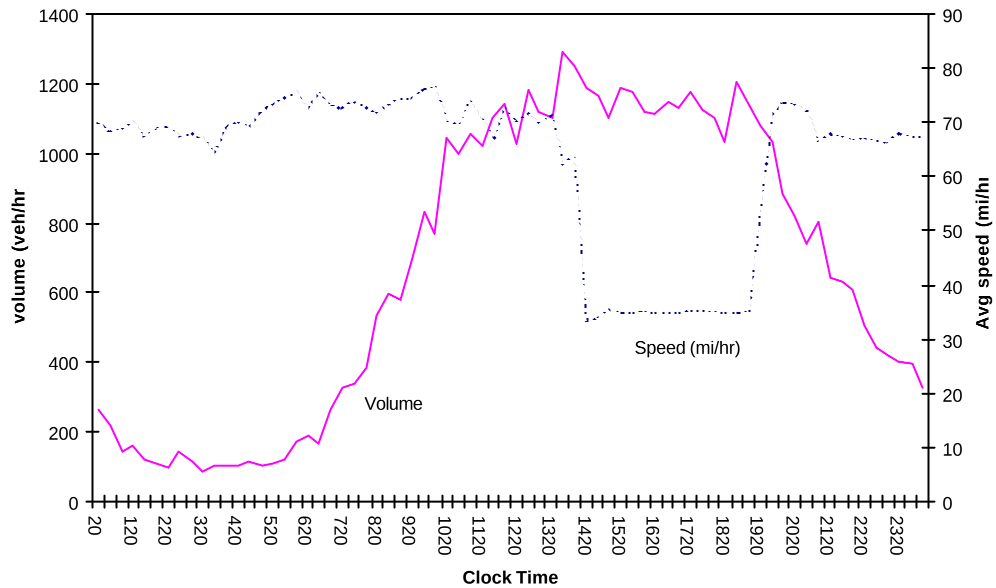

Figure 4.3: Typical volume-speed graph for the I-65 test bed, northbound end, 6/6/99

(with ILMS present)

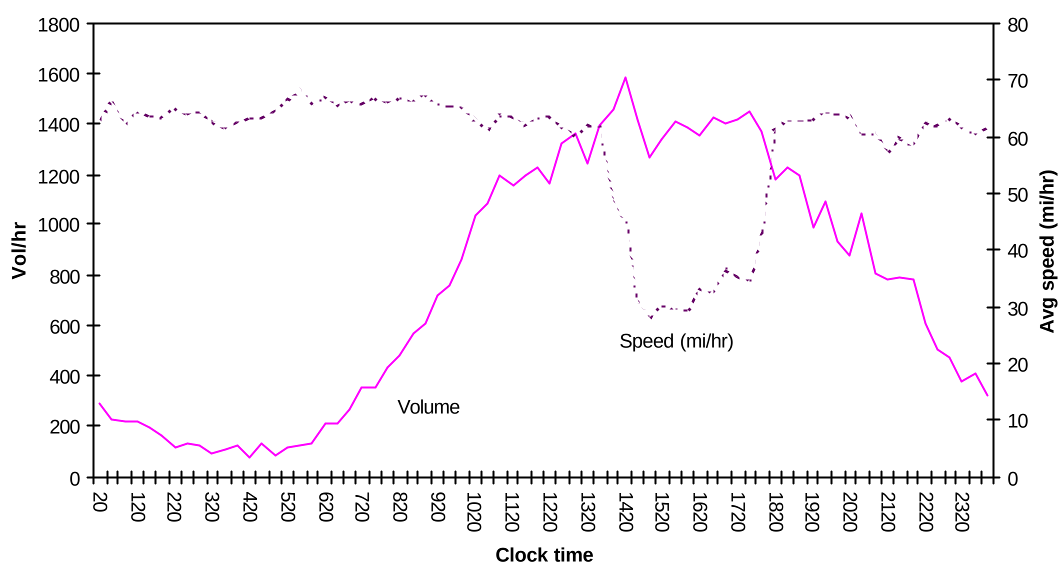

Figure 4.4: Typical volume-speed graph for the I-65 test bed, northbound end, 6/20/99

(without ILMS)

4.4 Modeling

4.4.1 Analysis of Covariance

Most of the explanatory variables are binary variables while the others are continuous variables. Therefore, the authors decided to use Analysis of Covariance (ANCOVA) to investigate the effects represented by single variables and the joint effects represented by multiple variables (interactions). Only two-way interactions were included in the study since interactions of higher order are somewhat difficult to interpret. The applied ANCOVA model can be stated as follows,

$$Y_{ijkmhwo} = \mu_{.....} + M_i + R_j + LD_k + P_m + H(X_h - \bar{X}_h) + WS(X_w - \bar{X}_w) + (MR)_{ij} + (MD)_{ik} + (MP)_{im} + (RD)_{jk} + (RP)_{jm} + (DP)_{km} + \varepsilon_{ijkmhwo}, \quad (4.1)$$

where:

M = main effect of presence of ILMS,

i = for ILMS present, i = 0 for ILMS not present,

R = main effect of rain,

j = 1 if it is raining, j = 0 otherwise,

LD = main effect of type of lane drop,

k = 0 if left lane dropped, k =1 if right lane dropped,

P = main effect of police variable,

m = 0 if police not present, m = 1 otherwise,

H = estimation parameter for the continuous variable, HEAVY,

X_h = percent of heavy vehicles,

WS = estimation parameter for the continuous variable, WSPEED,

X_w = wind speed in km/hr,

MR, MD, etc. = two-way interaction effects ,

$\varepsilon_{ijkmhwo}$ = error term, $N(0,\sigma^2)$,

$\mu_{.....}$ = overall mean.

The statistical modeling was performed using the Statistical Analysis Software (SAS). The first step was to identify the presence of outliers in the sample. The studentized residual values were compared with Bonferroni critical value (here = 3.7) and the largest studentized residual value was less than the critical value. Hence it was concluded that no outliers were present. The models were then developed and tested in the following manner. Independent variables were added at each step and the variables were tested for their statistical significance and also, the stability of the standard errors of the existing variables were observed. The changes in the standard errors were checked after bringing in and taking out independent variables. The final variables, which were included in the model, had a significant impact on the model. The slopes of the final independent variables were fairly stable under covariate inclusion and exclusion.

For testing statistical significance, the p-values and F-values of the independent variables were noted. The p-value is the probability that the estimated coefficient is greater than or equal to the value shown when the true value of the co-efficient is zero. A significance level of 5% was adopted for testing the null and alternate hypotheses. If p-value was less than 5%, then the variable is accepted; else it was rejected. The final model included those variables, which satisfied this criterion (Table 4.3). The goodness of fit was measured using R^2 values.

ILMS, rain, police and heavy vehicles turned out to be the only significant variables at the 5 % significance level (Table 4.3). All the variables had a negative (reducing) impact on capacity. Type of lane drop and wind speed turned out to be insignificant. ANCOVA has showed that all the two-way interaction effects are insignificant. This implies that all main effects are independent of each other. The R^2 value obtained for the model is 0.853. This indicates that the model is reasonably good and that it has a considerable predictive power.

In the next step, a regression model was developed to provide a more convenient and frequently used tool for predicting work zone capacities. Following the findings from ANCOVA, only the main effects are included.

Table 4.3: F-values and p-values for the explanatory variables

Source	DF	Type III SS	Mean square	F value	Pr > F
HEAVY	1	23912.90	23912.9	10.4	0.0016
RAIN	1	161394.08	161394.1	70.19	0.0001
ILMS	1	73630.19	73630.19	32.02	0.0001
POLICE	1	221858.66	221858.7	96.48	0.0001

4.4.2 Regression Additive Model

Capacity was assumed to be linearly dependent on explanatory variables. The linear model is of the form,

$$Cap = ß_0 + ß_1 M + ß_2 R + ß_3 LD + ß_4 P + ß_5 H + ß_6 WS \,, \tag{5.2}$$

where:

Cap = capacity of the work zone expressed in veh/hr,

M = indicator variable for ILMS,

M = 1, when ILMS present; M = 0, when ILMS not present,

R = indicator variable for rain,

R =1, when it is raining; R = 0, when there is no rain,

LD = indicator variable for type of lane drop,

LD = 1, if right lane dropped; LD = 0, if left lane dropped,

P = indicator variable for police presence,

P = 1, if police is present; P = 0, if no police present,

H = percentage of heavy vehicles in the traffic stream,

WS = wind speed near the location, in km/hr.

After performing multivariate analysis in SAS, the final model was obtained. As with ANCOVA, the variables were tested for their significance at the 5% significance level. The results shown in Table 4.4 are as anticipated. As seen from the ANCOVA results, wind speed and lane-drop are statistically insignificant. Police, rain, ILMS and percentage of heavy vehicles are significant and all of them have a negative impact on capacity. R^2 value of 0.853 obtained for the regression model is the same as for the ANCOVA model. The form of the model is as follows:

$$Cap = 1433 - 76\,M - 140\,R - 196\,P - 4.04\,H, \tag{5.3}$$

$$R^2 = 0.853,\ \text{Adjusted}\ R^2 = 0.849,\ \sigma(\varepsilon) = 48\ \text{veh/hr},$$

where:

Cap = capacity of the work zone expressed in veh/hr,

M = indicator variable for ILMS,

R = indicator variable for rain,

P = indicator variable for police presence,

H = percentage of heavy vehicles in the traffic stream.

The t-ratios, p-values, and the standard errors of the estimates have also been included. A plot comparing the predicted and the observed values of capacity is presented in Figure 4.5.

Table 4.4: The statistical estimates of the explanatory variables

Variable	Estimate	Standard error	t-ratio	p-value
Intercept	1432.82	11.85	120.87	0.0000
M	-76.29	13.48	-5.66	0.0000
R	-139.50	16.65	-8.38	0.0000
H	-4.04	1.25	-3.22	0.0016
P	-196.04	19.96	-9.82	0.0000

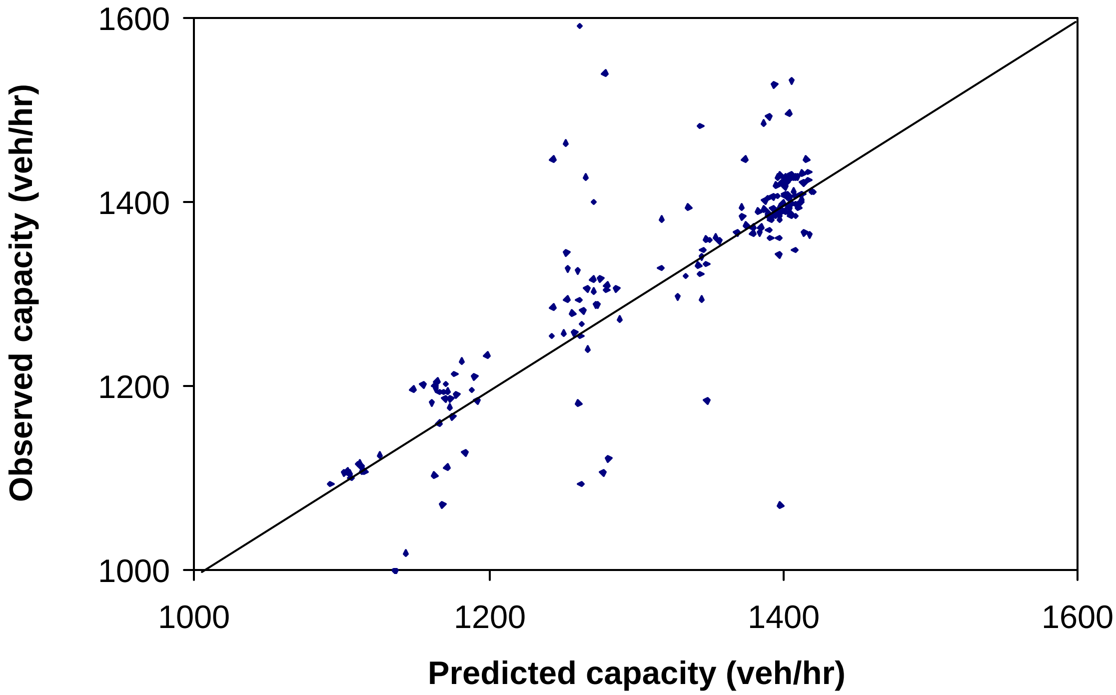

Figure 4.5: Predicted vs. observed capacity values, additive regression model

4.4.3 Regression Multiplicative Model

In the regression model, the capacity is expressed as an additive formula, where the effects of each variable are added to the capacity under ideal conditions. Another way is to adjust the ideal capacity with effects of various factors. From the previous analysis it has already been established that only the effects of ILMS, rain, heavy vehicles, and police should be included in the model. Hence, the adjustment equation for capacity on work zones may be given as:

$$Cap = Cap_{ideal} \cdot f_M \cdot f_R \cdot f_H \cdot f_P, \qquad (5.4)$$

where:

Cap = capacity in one direction for prevailing roadway, traffic control and traffic conditions,

Cap $_{ideal}$ = capacity under ideal conditions. The ideal conditions in this case are defined as with no traffic control (no ILMS), under normal weather conditions (no rain), no police presence and no trucks,

f_M = adjustment factor for Indiana Lane Merge System (ILMS),

f_R = adjustment factor for rain,

f_p = adjustment factor for police,

f_H = adjustment factor for heavy vehicles in the traffic stream, computed as

$$f_H = \frac{1}{1 + P_H (E_H - 1)},$$

P_H = proportion of heavy vehicles in the traffic stream, expressed as decimal,

E_H = passenger car equivalent for heavy vehicles.

The model has been fit by minimizing the error sum of squares, i.e., $(Cap_{obs} - Cap_{est})^2$, where:

$$Cap_{est} = Cap_{ideal} \cdot f_M \cdot f_R \cdot f_H \cdot f_P.$$

The Newton-Raphson convergence technique was used for obtaining the best-estimated values. The initial solution used was: $Cap_{ideal} = 1320$, $f_M = f_R = f_P = 1$, $E_H = 2$ and the convergence critical value was assumed at 0.0001.

The final estimated values for the adjustment factors and the ideal capacity are given in Table 4.5. The estimated value of ideal capacity corresponds very closely to the intercept (ideal capacity) obtained from the regression model. The standard error of estimation, 53veh/hr is close to the value obtained in the regression model, 48 veh/hr. A comparison between the observed and estimated values is given in Figure 4.6. The form of the model is consistent with the equations provided in the highway capacity manual.

Table 4.5: The estimates of the adjustment factors

Variable	**Estimated value**
Cap_{ideal}	1442
f_M	0.939
f_R	0.910
f_P	0.858
E_H	1.372

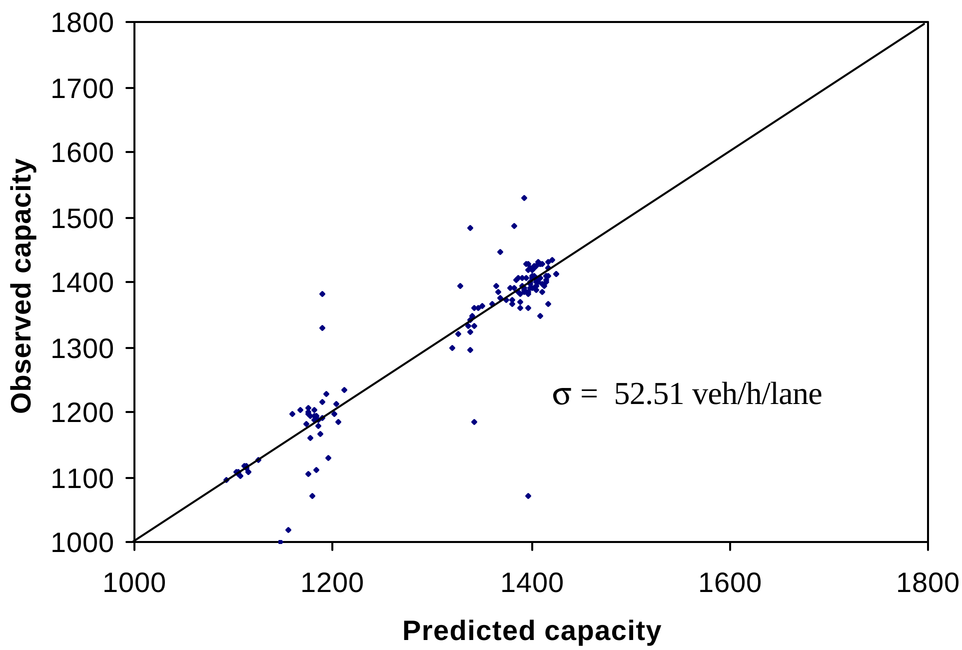

Figure 4.6: Predicted vs. observed capacity values, multiplicative regression model

4.5 Discussion

The results indicate that most of the investigated variables except type of lane drop and wind speed are significant. All the significant variables carry a negative sign, which means that all of them represent effects that reduce capacity. The signs of all variables except ILMS and POLICE are as expected. A discussion of the results is given below.

Intercept: The intercept value of 1433 veh/hr is the capacity under conditions that are defined ideal. These conditions are characterized by lack of precipitation, no ILMS, police absence and no heavy vehicles. The ideal capacity of 1433 veh/hr is nearly 100 veh/hr higher than the average sample value (1320 veh/hr) and higher than average capacity values in Highway Capacity Manual, 1340 veh/hr in Table 1. It should be noted that the values presented in HCM are representative of average conditions and not just of ideal conditions.

Rain: Rain, as expected, reduces capacity. On an average the reduction in capacity is 140 veh/hr or about 10 %. The reduction is close to the values quoted by the Highway Capacity Manual (1997). Slippery pavements and poor visibility during rain increases drivers' caution, which results in longer headways. It should be kept in mind that the obtained capacity equation describes non-winter conditions. During winter, snowy conditions should not be confused with non-rain conditions.

Heavy Vehicles: As expected, heavy vehicles reduce the capacity. The estimated reduction is about 4.04 veh/hr for each 1% increase in truck percentage. One truck is equivalent to 1.4 non-heavy vehicles. As explained earlier, the greater the number of trucks, the greater is the mean headway between vehicles. Also, since trucks move at lower speeds compared to other automobiles, the average speed also goes down as the number of trucks in the traffic stream increases.

ILMS: Indiana Lane Merge System (ILMS) was expected to increase the capacity due to reduced number of aggressive merging at the taper. The results indicate that ILMS reduces the capacity by about 76 veh/hr or about 5 % of capacity. Although the reduction is limited, from the statistical and practical standpoints it is significant. One reason of the capacity reduction hypothesized by the authors is the driver reaction to the new signs. Drivers tend to be more careful and thereby tend to slow down and keep longer headways. Since, ILMS is a new system not widely used there is a chance, that this effect will weaken as the drivers become more familiar with the system.

Police: The police effect is even more surprising than the ILMS effect. A stronger compliance rate to the ILMS DO NOT PASS signs was observed during the police presence. At the same time, the capacity was lower by nearly by 200 veh/hr. A plausible explanation is that drivers become extra cautious and keep larger gaps when they see police.

Type of lane drop: The effect of type of lane drop (left or right) on capacity was found insignificant. This result is valuable to roadway management personnel, who are fraught with the problem which lane should be dropped. The study shows that the side at which lane is dropped has no effect on capacity of work zone.

Wind speed: Strong winds were believed to reduce capacity. In presence of strong winds, drivers of tall vehicles are expected to keep longer gaps and drive slower. Although, the results do not confirm the above expectations, it should be noted that the wind speeds in the sample were on the lower range. The strongest wind was about 24 km/h.

4.6 Closure Remarks

The primary aim of this chapter was to analyze the effect of Indiana Lane Merge System on capacity of freeway work zones. The traffic volumes had to be collected on work zone approaches during traffic congestion with and without ILMS boards. Due to the high costs, the coordination difficulties, and liability concerns involved in organizing and executing traffic observations in work zones, the data were collected for an extended period in a single work zone. The selected work zone is considered typical for rural conditions. The capacity observed during congested periods over nearly four months averaged at 1320 veh/h/lane which conformed very closely with 1340 veh/h/lane reported by HCM for this type of work zones (rural, long-term, four lanes reduced to two). The effect of heavy vehicles represented by the equivalency factor 1.4 and 10-percent capacity reduction caused by rain obtained for the selected work zone are also very close to the values reported by HCM. **These comparisons indicate that the selected test bed truly represents the typical conditions and the results can be generalized to other typical long-term rural work zones in Indiana.**

The new system has reduced the work zone capacity by 5 % and the authors attribute this reduction to the unfamiliarity of the drivers to the new system. A similar reduction was observed in summer, 1996, on the southbound and northbound approaches to the freeway work zone on I-69.

The authors also tried to re-address some other factors like the effect of rain and heavy vehicles on capacity. As expected, the authors observed that these factors reduce capacity. The effect of the presence of police on the capacity of freeway work zones was also analyzed. Although a stronger compliance rate to the merge signs on the approach was observed due to police presence, there was a significant reduction in capacity (14 %). The authors attribute this effect to the extra cautious behavior of drivers. The authors also observed a capacity drop after queue of vehicles formed on the approach to the work zone. This phenomenon was observed only for a few cases and the process is not well described. In addition, some valuable insights were discovered about several unusual traffic behaviors, especially the two-truck phenomenon all of which have a significant effect on capacity.

5. CONFLICT FREQUENCY MODELS

Each crash is preceded by a dangerous situation; some of these situations turn into accidents, the rest into near misses. A dangerous situation itself is preceded by some kind of incipient danger. Therefore, the continuum of events preceding crashes can be pictured as a pyramid with normal events at the bottom and crashes at the apex. It is obvious that frequency of non-crash events is far greater than the frequency of crashes. One of the traffic events is a traffic conflict. A traffic conflict can generally be described as a situation in which a driver perceives that evasive action is required to avoid a collision or to secure a safe maneuver. Evasive action may be decelerating or weaving or any other move that the driver considers useful and expedient. In such cases these actions may be directly or easily observable, and in others collisions may occur without any evasive action being taken. This first formal definition of traffic conflicts was given by Perkins and Harris (1969). Spicer (1971) added a severity dimension to the traffic conflict technique. He defined conflicts as moderate, dangerous and critical conflicts. He noted that conflicts were not related to crashes, but serious-conflicts were very closely related to serious (injury) crashes. But, due to the extent of subjectivity involved, many of these studies have been questioned. Hayward (1972) proposed a new measure called *time to collision* that is used to define traffic conflicts in a more objective manner. Numerous studies have focused on the technique's reliability, objective definition and conflict-crash relationship. Hutchinson (1988) found a lot of observer disagreement in defining different types of conflicts. A significant study on the relationship between crashes and conflicts was done by Brown (1994). In his study, conflicts were observed at intersections and evaluated against 5-year crash records at the same sites. Although, the correlation between the overall number of crashes and conflicts was not strong, it was higher for individual crash and conflict categories (merging conflicts and merging crashes, rear end crashes and braking conflicts etc.). Such stratification seemed to yield better and statistically significant results and the correlation factors seemed to concur with results obtained in similar research.

The traffic conflict technique is sometimes used to evaluate safety for the following reasons. Unlike crashes, traffic conflicts are more frequent and hence easier to observe. As mentioned earlier, when analyzing the safety effects of more permanent highway improvements, crashes may be a good measure of safety. For example consider the installation of signal a system at a new intersection. To study the improvement in safety due to the system, crash data can be collected for some years before the signal was installed. After the system has been installed, the safety engineer then waits for the same time and collects the crash data during this followed period. The safety benefits can be obtained by analyzing crashes before and after the system is installed. When evaluating new ITS strategies, however, such long study periods are not possible. Sometimes this is due to continuous improvement in technology. In other words, by the time the study is completed the system would have become obsolete. This calls for faster evaluation methods. Since conflicts are more frequent than crashes, safety studies using conflicts can be completed in a matter of months. Further, while observing conflicts, a better feel for pre-conflict driver behavior and the circumstances leading to the conflict can be obtained.

This helps the safety engineer in deciding which elements of the highway system need improvement.

As discussed in Chapter 2, ILMS had not been implemented anywhere in the state and INDOT needed the evaluation of the system as fast as possible. It was clear that traditional before and after studies using crashes was not an answer to the problem. It was decided to use a combined method of crashes and conflicts as an alternative. This chapter focuses on the development of conflict frequency models.

5.1 Data Collection

Data needed to develop conflict models was collected from the I-65 work zone near West Lafayette, Indiana. The capacity analysis presented in the previous chapter has given strong evidence that the traffic operations were quite typical for rural long-term work zones. **The traffic conflict data collected there is believed to represent other rural work zones.** The data was collected over a 4-month period extending from April 1999 to July 1999. Traffic approaching the work zone was recorded on videotapes to be watched and analyzed later. Data was collected during congestion and non-congestion conditions, weekdays and weekends, sunny and rainy conditions, in the presence of ILMS and without ILMS. In several cases videotaping was done from overpasses while in other cases videotaping was done from the median using a special video mast. Various aspects of the data collection process is discussed in detail in the following sections:

1. The site,
2. Equipment,
3. Videotaping process,
4. Extracting traffic conflicts.

5.1.1 The Site

The site selected for study was an I-65 work zone site near West Lafayette, Indiana, extending approximately from MP 180 near US 43 interchange to MP 193 near the US 231 interchange. The construction zone was in place from the beginning of April 1999 to about August 1999. The construction included both the northbound and the southbound directions. It was a two-lane highway where one of the lanes was closed all the time. The average daily traffic is about 26,000 veh/day on weekdays and up to 42,000 veh/day during the weekends. Congestion occurred usually during the weekends: on Fridays in the southbound direction and on Sundays in the northbound direction. Typically congestion in the southbound direction started around 3:00 pm and ended by around 5:00 pm. In the northbound direction, the respective times were 2:00 pm and 6:00 pm. The area provided potential sites for videotaping. Usually, videotaping was done from, the median or from the overpasses. Two overpasses were available, with close proximity to the taper, CR 800 S overpass near the southbound entrance (Figure 5.1) and CR 600 N overpass near the northbound taper (Figure 5.2).

5.1.2 Equipment

An equipped van (Figures 5.3 and 5.4) was used to videotape traffic. It had a portable, collapsible mast on the top of which two cameras (Figure 5.5) were mounted on pan / tilt mechanisms. The mast could be raised with the help of an electric winch up to 45 ft. The video signal from the cameras was transmitted through a cable down to a monitor. A control panel, (Figure 5.6) connected by means of the same integrated cable, allowed for adjusting the camera pan and tilt angles. To prevent swaying of the mast at high winds, eight guy wires were attached to the mast and each wire was then tied at the ground to a 100-pound concrete block. These blocks made sure that the mast was very stable even at high winds and that clear and stable pictures were available for analysis

5.1.3 Videotaping Process

The objective of the videotaping was to capture all the traffic details needed to recognize different types of conflicts. The segment in view could not be too short, because this would hamper the observation of conflicts especially in congested conditions. Neither the segment could be too long because this would place too much of a demand on the part of the observer to view all the conflicts and events happening along the long stretch of the segment. This would lead to missing conflicts, which might corrupt the validity of the model. The typical lengths of videotaped sections varied from around 500 ft to about 1500 ft. To observe different sections on the approach, videotaping was done at different locations along the approach segment. This included locations very close to the taper, some at medium distance from the taper and some far off from the taper. The videotaping was done over a period of 4-months from April 1999 to July 1999. For the first half of the videotaping process, videotaping was done without the ILMS turned on. Then the system was turned on and an adjustment period of 2 weeks was given to enable the drivers to get used to the system. After this period, videotaping was done with ILMS turned on. A total of 46 two-hour tapes were recorded. This represented 92 hours of traffic data for purpose of conflicts analysis. The counts were recorded in 15 min intervals.

Figure 5.1: Southbound side of the construction zone (MP 180 near US 43 interchange)

view from CR 800S overpass

Figure 5.2: North bound side of the construction zone (MP 180 near US 43 interchange)

view from CR 600N overpass

Figure 5.3: The Video detection vehicle (side view) on I-65 near West Lafayette, Indiana

Figure 5.4: The Video detection vehicle (rear view) on I-65 near West Lafayette, Indiana

Figure 5.5: Digital camcorders used for videotaping

Figure 5.6: Remote control panel for controlling camera movements and the monitor

5.1.4 Extracting Traffic Conflicts

Data analysis involved the observation of conflicts, recording of conflict data and then processing the data using appropriate statistical tools. The observation of conflicts is the one of the most important phases of the traffic conflict technique process. For obtaining valid and error free results, an observer has to be focused, creative and consistent at all times. The viewing of the videotapes was done over a long period of time to prevent any observer fatigue. The viewing was restricted to only 2 tapes (4 hours) a day. Sufficient breaks in between were also given to make sure that the entire process did not become monotonous. This helped keep the observer focused at all times. Also only one observer was chosen for the counting process to remove any inter-observer bias and disagreements. The observer also underwent a training session to make his conflict judgments less subjective. To remove any bias in the tapes interpretation by the observer, the tapes were watched in random order. To check the consistency in observation skills, the same tapes were watched at different times to see if the conflict counts were consistent. A high value of consistency (less than 10% error) was observed.

It was mentioned in the introduction that the linear correlation between crashes and conflicts becomes significant when crashes and conflicts are disaggregated. A study conducted by the authors on crashes on approaches to work zones had indicated that about 90% of the crashes on the approaches are either merging or rear end crashes. Hence, the authors decided to focus on these crashes while building conflict models. The conflicts for this study are thus broken down into two categories:

1. Merging followed by sudden braking,
2. Sudden braking without any preceding merging.

Dangerous and sudden merging can be defined in the following conditions. The first case is when the vehicle suddenly merges into the open lane. Such a maneuver might lead to a crash if the drivers in the continuous lane are not careful enough and do not take any evasive maneuvers. The second case when a vehicle tries to merge when there is actually not enough space in the open lane. Both these conditions are identified by sudden disruptions in the traffic in the continuous lane. Dangerous and sudden braking can be identified by sudden flashing of braking lights and general disruption in nearby traffic. The sudden braking can be looked upon as a condition where the driver had to take a sudden evasive maneuver, such as a sudden reduction in his speed so as to avoid an impending accident.

5.2 Analysis of Data

To analyze the effectiveness of the system in improving safety, we could have counted the number of conflicts before and after the system (ILMS) is installed. However, since all the other parameters such as length along which conflict counts are taken, the section where the conflicts are measured, and traffic volumes are not the same, absolute conflict counts cannot be used for the purpose. A better option was to develop a regression model to estimate the expected number of conflicts given the following characteristics:

1. Time for which counts are observed, T (min),

2. Length of the segment along which counts are observed (in meters) , LS,
3. Traffic volume, Q (veh/hr),
4. Percentage of heavy vehicles, H,
5. Presence of ILMS (ILMS absent = 0 , ILMS present = 1), M,
6. Presence of congestion (congestion = 1, no congestion = 0), CG,
7. Presence of rain (No rain = 0, Rain = 1), R,
8. Distance from the taper to the center of the videotaped segment (in meters), DI,
9. The dropped lane (left / right), LD (Left = 0, Right =1).

The time parameter was eliminated from the study, since all the counts were taken in 15-minute intervals. The length of the segment could be considered as measure of exposure to conflicts. If the rest of the factors remain constant, then we should expect twice as many conflicts on a segment twice as long as another one. Length of segment was measured in meters. Traffic volume also can be considered as a measure of exposure of risk to conflicts. The volume was converted from the hourly counts to an average value over 15-minute intervals. The volumes were obtained from a set of loop detectors, which were installed at both northbound and southbound tapers. ILMS is a binary variable which takes the value of 1 if ILMS is turned on and 0 if it is turned off. Congestion is represented by another binary variable, which takes into account the effect of congestion in the open lane. This is determined by observing traffic conditions in the open lane.

Weather, especially rain, may affect the traffic behavior. Slippery pavements and reduced vision may lead to very serious traffic conflicts. A binary variable was included to represent rain and no rain conditions.

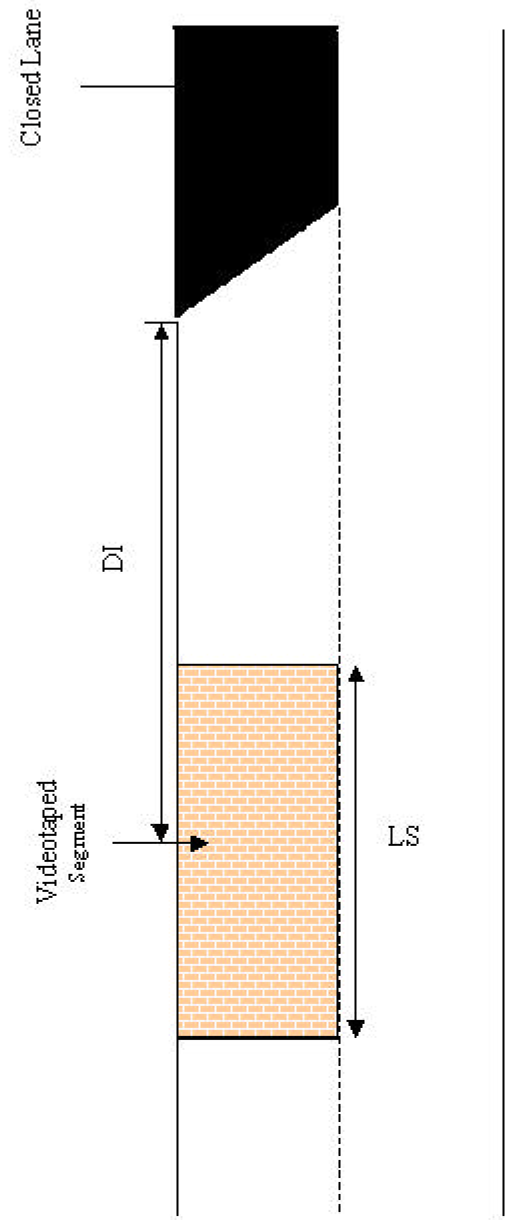

Figure 5.7: The videotaping segment for conflict observation

The distance from the taper to the center of the segment (Figure 5.7) with observed conflicts (DI) is a proxy variable to differentiate between different portions of the queue. For example, very low DI value means that we are analyzing the head portion of the queue, i.e., at the start of the taper. Very large DI values indicate portions far off from the taper, or in most cases the end portions of the queue. Collecting data from different sections of the queue allowed for investigating safety level along the queue.

The type of lane drop (left lane dropped / right lane dropped), LD was included as an independent variable to check its effect on safety. This binary variable took the value 0 when left lane was dropped and 1 when the right lane was dropped.

5.3 Regression Model

For analyzing conflicts, two types of regression models were in consideration: (1) Linear model and (2) Negative Binomial model. We assumed that conflicts like crashes are discrete, random events and hence can be described with a Poisson process. We used the Negative Binomial model, which accounts for the extra variation due to omitted variables in the model. The extra variation is represented by a measure called the over dispersion parameter. The Negative Binomial model will give the expected number of conflicts given the independent variables. The general form of the model developed for this case may be represented as follows:

$$CON = K\,(Q)^{a_1}\,(LS)^{a_2} \exp\left(?_1 M + ?_2\, CG + ?_3\, DI + ?_4 R + ?_5 H + ?_6 LD\right), \qquad (5.1)$$

where:

CON = expected number of conflicts (/15 min),

Q = average volume of traffic (in 1000 vehicles/hr),

LS = length of the segment under consideration (m),

M = binary variable representing whether ILMS is installed or not,

CG = binary variable representing the status of congestion,

R = binary variable to indicate the presence of rain,

H = percentage of heavy vehicles,

DI = proxy variable to represent the segment of the queue; represents the distance of the queue from the taper (m),

LD= binary variable to represent the dropped lane.

The statistical analysis was done using LIMDEP. Multivariate analysis was done to determine the statistically significant variables and reject those variables that were found to be statistically insignificant. The analysis was done by starting with a single independent variable and then the variables were added to the model to determine the reduction in the over dispersion parameter. The final model was selected by choosing only those variables that were found to be statistically significant. It was found that even after adding the insignificant variables to this model the reduction in over dispersion parameter was not significant. It is to be noted that since all data points had the same duration of 15 minutes, the duration as an independent variable had been dropped from the model. The dependent variable, CON, became the expected rate of conflicts per 15 min. Separate models were built for predicting conflicts in the two categories: braking and merging.

5.4 Discussion of Results

Negative Binomial models were used for predicting the expected number of conflicts. The models were developed and tested in the following manner. Independent variables were added at each step and the variables were tested for their statistical significance and also, the improvement in the overdispersion parameter was observed. For testing statistical significance, the p-values and t-values of the independent variables were noted. The p-value is the probability that the estimated coefficient is greater than or equal to the estimated value when the true value of the co-efficient is zero. The t-value represents the ratio between the estimated slope of the independent variable and the standard error. A significance level of 10% was adopted for testing the null and alternate hypotheses. If p-value is less than 10% then the variable is accepted, else it is rejected. The final model includes those variables that satisfy this criterion. By adding the statistically insignificant variables, the improvement in the overdispersion parameter was not significant.

The goodness of the model is measured using ρ^2 values and the overdispersion parameter, α. The overdispersion parameter was statistically significant in both the models. The α value for the merging conflict model was 0.635 and α value for the rear end conflict model was 0.596 (Table 5.1). Therefore, the errors of models are 80 % and 77 % respectively. The ρ^2 values measured are 0.27 and 0.29 for the merging conflict model and the braking conflict model respectively. The relatively low ρ^2 values may be due to unknown variables not included in the model. The p-values, the t-statistics and the standard errors for the estimates in both the models are given in Tables 5.2 and 5.3.

The slopes and signs of the independent variables seem to concur with the assumptions that that the authors had before developing the model. Comparisons between the predicted and observed values for merging and braking conflicts are given in Figures 5.8 and 5.9. From the graphs it is evident that not much bias is present in the models. In the subsequent section the interpretation for the slopes and signs for each of the independent variables is given.

Table 5.1: Regression models for braking and merging conflicts

Conflict type	Model	ρ^2	α
Merging	$CON = 0.0077 \ (Q)^{1.61} (LS)^{0.656} \exp(-0.511\ M + 0.711\ CG - 0.0002\ DI)$	0.27	0.635
Braking	$CON = 0.0077 \ (Q)^{1.66} (LS)^{0.658} \exp(-0.498\ M + 0.697\ CG - 0.0002\ DI)$	0.29	0.597

where:

CON = number of conflicts (/15 minutes),

Q = traffic volume (in 1000 veh/hr),

LS = length of the section (in meters),

M = binary variable to indicate the presence of ILMS; M = 1 if ILMS is present, M = 0 otherwise,

CG = binary variable indicating the presence of congestion; C= 1 if congestion is present, CG =0 otherwise,

DI = Distance from the segment under consideration to the taper (in meters).

Table 5.2: Regression parameters for merging conflicts

Merging conflicts				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-4.082	1.861	-2.194	0.0283
Q	1.610	0.552	2.916	0.0035
LS	0.657	0.299	2.197	0.0280
M	-0.512	0.149	-3.439	0.0006
CG	0.714	0.259	2.758	0.0058
DI	-0.0002	0.0001	-1.860	0.0629
α	0.635	0.259	2.441	0.0156

Table 5.3: Regression parameters for braking conflicts

Braking conflicts				
Variable	Coefficient	Standard error	t-ratio	p-value
K	-4.082	1.822	-2.240	0.0251
Q	1.664	0.543	3.065	0.0022
LS	0.658	0.293	2.247	0.0246
M	-0.498	0.145	-3.429	0.0006
CG	0.697	0.254	2.741	0.0061
DI	-0.0002	0.0001	-1.917	0.0552
α	0.596	0.257	2.317	0.0203

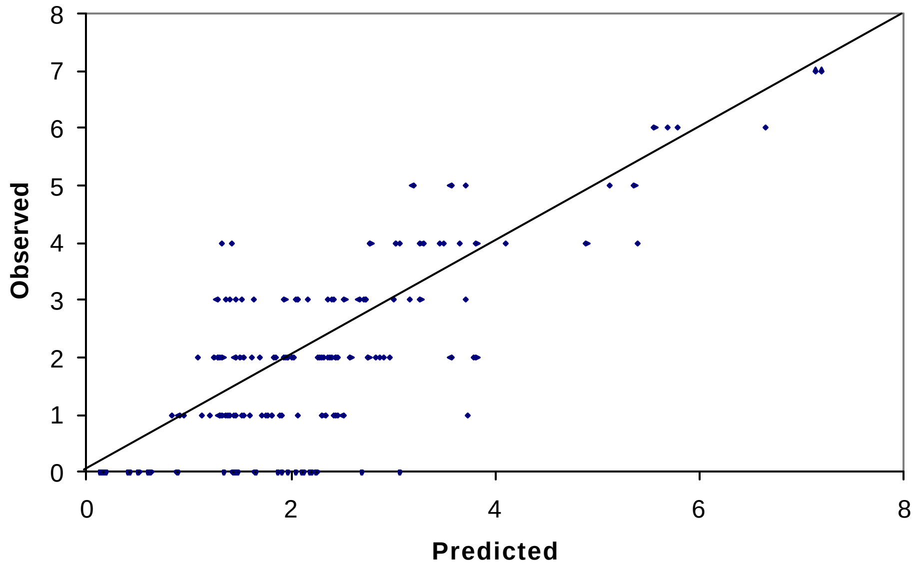

Figure 5.8: Predicted and observed values for merging conflicts

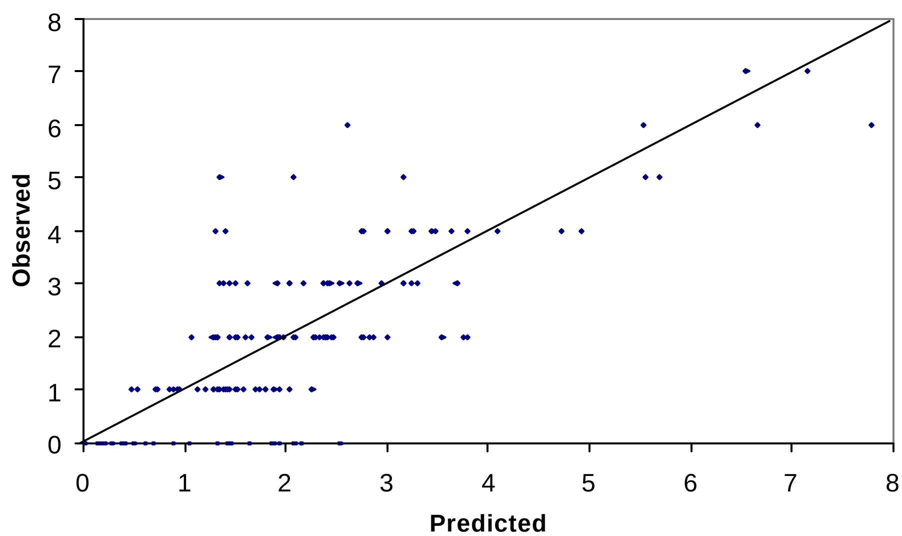

Figure 5.9: Predicted and observed values for braking conflicts

The final form of the model indicates that length and volume are significant and their regression parameters carry a positive sign. ILMS has a decreasing effect on conflicts as was expected. The congestion variable, C has a positive sign. Among the other independent variables, only DI (distance to the taper) was significant and had a negative sign.

Length: As was expected, the length of the segment turned out to be significant. The regression parameter associated with this variable has a positive sign and has a value of 0.656 and 0.658 in merging and braking conflicts respectively. However, the non-linear dependence of conflicts on length is a very surprising result. This does not conform with the assumption of the length as an exposure-to-risk variable. This means that after a certain length, the marginal increase in conflicts will tend to zero. This non-linear effect of length may indicate that counting conflicts on long segments was more difficult than expected. Consequently, some conflicts were overlooked and the models may under estimate the number of conflicts for very long segments.

Volume: The volume of the traffic also turned out to be a significant factor. The slope for this variable is 1.610 and 1.660 for merging and braking conflicts respectively. This indicates that as the volume of the traffic increases, the number of conflicts also increases, but not in a strict linear fashion. The cause might be that, some variables correlated with traffic volume have not been included in the model.

ILMS: The primary purpose of the study was to examine the effect of ILMS (Indiana lane Merge System) on safety in work zone approaches. By inducing the vehicles to merge ahead of the queue, it was believed that the merging process will be smoother and hence much safer. The ILMS variable, as expected, turned out to be a significant variable with a decreasing effect on the number of conflicts. The regression parameter associated with this variable has a value of -0.5114 for merging conflicts and a value of –0.4983 for braking conflicts. This means that when ILMS is installed, we can expect a $1- e^{-0.5114}$ (or 40 %) reduction in number of merging conflicts and a $1- e^{-0.4983}$ (or 39 %) reduction in braking conflicts assuming that ILMS does not change capacity.

Congestion: This variable, CG, was used to incorporate the effects of congestion on the number of conflicts. As was expected, the number of conflicts increases with congestion. The expected increase in number of conflicts turns out to be $e^{0.711} – 1$ (or 100 %). This means that the expected number of conflicts/ crashes almost doubles in the presence of congestion.

Distance to taper: This variable, DI, was included in the model to analyze the variation in the number of conflicts along the queue. The regression parameter had a value of about –0.0089, which indicates that a decrease in the number of conflicts is observed as the distance away from the taper increases.

5.5 Closure Remarks

Since crashes and conflicts are linearly correlated, this means that

$$(1-\frac{Crashes_{after}}{Crashes_{before}}) = (1-\frac{Conflict_{after}}{Conflict_{before}}), \tag{5.2}$$

$$\text{Crash}_{\text{reduction}} = \text{Conflict}_{\text{reduction.}} \tag{5.3}$$

Thus the crash reduction can be estimated through the conflict reduction. The number of crashes reduced due to the introduction of a new traffic improvement/ITS system (example, ILMS) is given by:

$$\text{SI} = \text{E(Crashes}_{\text{before}}) - \text{E(Crashes}_{\text{after}}), \tag{5.4}$$

$$\text{SI} = \text{E(Crashes}_{\text{before}}) \,.\, \text{Crash}_{\text{reduction}}, \tag{5.5}$$

$$\text{SI} = \text{E(Crashes}_{\text{before}}) \,.\, \text{Conflict}_{\text{reduction}}. \tag{5.6}$$

Now, with the help of conflict frequency models, we can estimate the number of crashes reduced due to ILMS.

6. SENSITIVITY ANALYSIS

In Chapter 2, a new method of evaluating the safety effects of ILMS was proposed. In the subsequent chapters, various components of the method were developed and discussed in detail. To evaluate the final effects of the system, these models have to be integrated and tested for a variety of scenarios. The final task is to develop simple guidelines for the use of the system on rural freeway work zones. For this purpose a spreadsheet based application was developed in Microsoft Excel© using the Visual Basic programming language. The various components of the evaluation tool are discussed in detail in the following section.

6.1 Constraints

Three scenarios are possible: (a) queues and delays grow according to the demand excess over capacity without any effect on motorists' behavior, (b) queues grow up to some limit and then, motorists start diverting from the freeway at the rate that keeps the queues fixed, and (c) delays increase up to some limit and then motorists start diverting from the freeway at the rate at which delays remain constant.

1. *No constraint scenario*: Here we assume that there are no constraints on the length of the backed up queue and that drivers accept long queues and delays. Because of this, the analysis works for low-to-moderate AADT values from 40,000 veh/day to about 50,000 veh/day. For higher AADT values, length of queue reaches unrealistic values of 50 km and more. In reality, hardly any driver would accept such large queues and delays. Hence, the user is given the option of imposing queue constraints or delay constraints.

2. *Queue constraint scenario*: It is observed that, when the queues in work zones or at any other locations reach a certain length, some drivers divert to an alternate route instead of joining the long queue. The user may input the maximum length of queue beyond which vehicles will be re-routed. The maximum queue should be determined based on the local opportunity for re-routing. In other words, the maximum queue is determined as the distance at which an off-ramp for an alternate route is available. To prevent the length of the queue from increasing beyond this value, the model allows the portion of demand equaling capacity to stay on the freeway while the demand excess is re-routed on the alternative path. For example, consider the maximum queue of 10 km. The demand for that particular hour when the queue is 10 km is 1500 veh/hr and the capacity is 1400 veh/hr. Then we assume that only 1400 vehicles will use the work zone and the rest 100 vehicles will use the alternate route.

3. Delay constraint: This constraint is applied similarly as the maximum queue constraint proposed in the last section. Here, the drivers are assumed to be more responsive to the delay time than to the queue. This may be true for an urban or near urban corridor where most of road users are commuters. These people will have a good

knowledge of alternative routes in the network. Hence, drivers will be deciding on alternate routes even before they actually start the journey or in some cases might even decide to forego the particular trip. Based on the driver preferences in the work zone area, the user is asked to input the maximum delay. This delay is then converted to the corresponding maximum queue. The mechanism of applying this constraint is the same as already explained above for queues. Due to this equivalency between the two constraints, sensitivity analysis has been done only for the queue constraint.

Re-routing of vehicles changes the safety level on surrounding road sections. As mentioned in Chapter 2, along with the safety impact on the approach to the work zone, the safety impact is also evaluated on the alternate route, inside the work zone segment and on the open freeway sections. The following additional information is needed:

1. Length of the alternate route,
2. Distance from the end of the work zone segment up to the point it meets the freeway again.

The entire process can be summarized in the flowchart given in Figure 6.1.

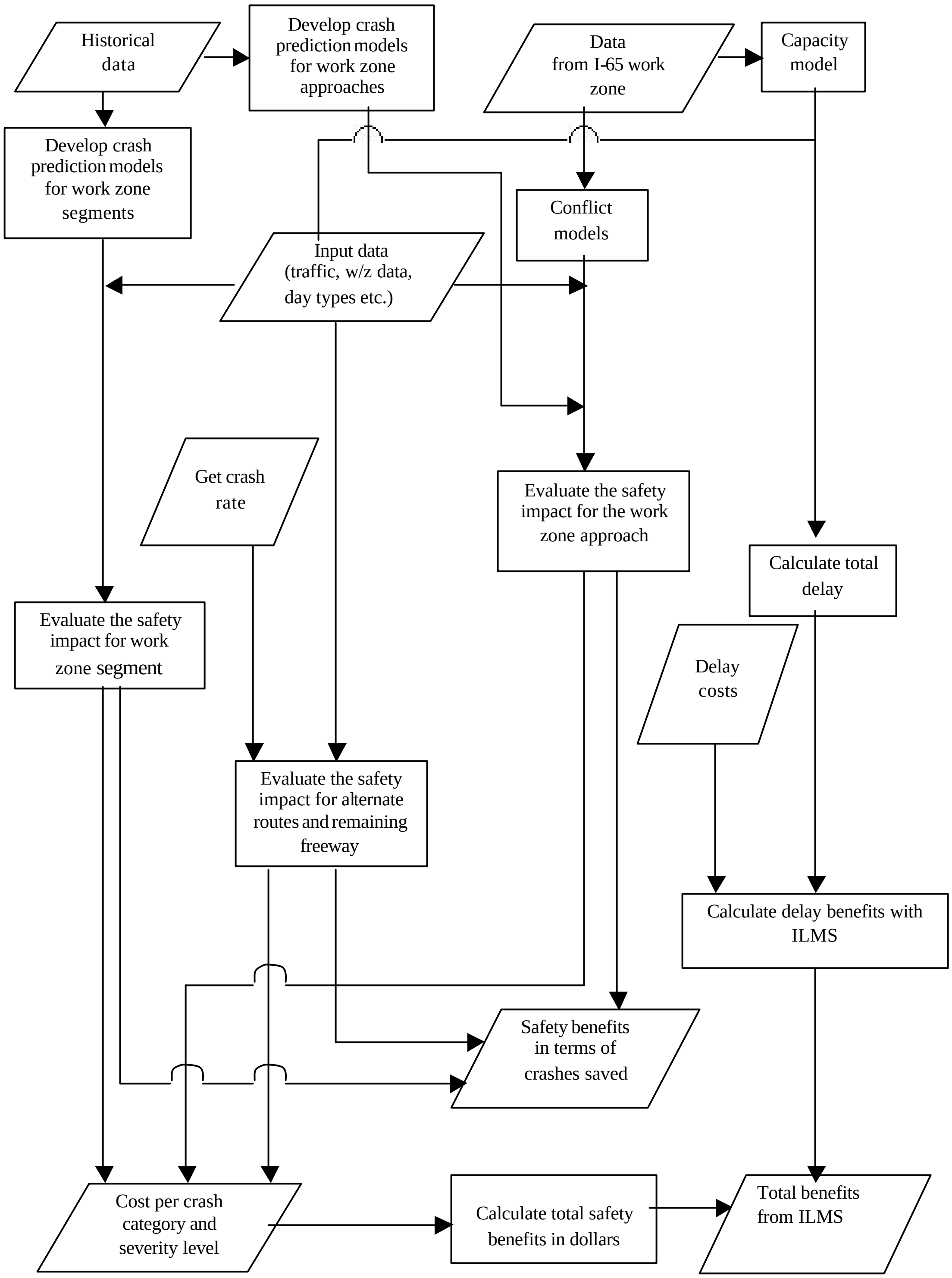

Fig. 6.1 Flow chart for overall safety and delay impact evaluation of ILMS

6.2 Factors Considered

To better understand the safety impacts of the Indiana Lane Merge System, it is imperative to conduct a detailed sensitivity analysis, by investigating the changes in safety and delay to changes in input parameters. The effect of some of the parameters is complex and hence it needs to be studied in detail, while the effect of other parameters is easily predictable. The following input parameters were included in the sensitivity analysis.

1. The capacity effect of ILMS: Three cases of capacity effect of ILMS are tested. One of them is the reducing effect of ILMS (-76 veh/hr) as observed in the test bed work zone. The other two are: (a) no capacity effect of ILMS and (b) an increasing capacity (+76 veh/hr) effect.
2. Queue constraints: Here the safety and delay benefits sensitivity on queue length is tested.
3. Traffic volume: The effect of volume is tested for all the three capacity cases and for both levels of queue constraints: (a) no queue constraint, and (b) queue constraint.

The objective of the analysis is to determine conditions where safety benefits can be expected. The final results should include simple guidelines for the use of the system. A typical rural freeway work zone with the following parameters is considered for analysis.

1. The length of the work zone – 8 km,
2. Duration of the project – 8 months,
3. Estimated cost of the project – $ 8 million ,

4. Starting month – February,
5. Length of the alternate route – 25 km,
6. Distance on the freeway from the end of the work zone to the point where the alternate route meets the freeway again – 5 km,
7. Ideal capacity – 1433 veh/hr,
8. Truck effect - -4.04 veh/hr/percentage of heavy vehicles,
9. Day types – rural weekday and rural weekend,
10. Percentage duration of day type – weekday – 71.43 %, weekend – 28.57 %.

The sensitivity analysis is performed in two stages. In the first stage, default daily vehicle profile values and default heavy profile values constructed using data from rural interstates in Indiana is used. However, the data provided no information whatsoever regarding the directional distribution of traffic. The authors assumed simple 50 % directional split of traffic in either direction. However, this may not be true especially for rural freeways connecting two major cities due to wide fluctuations in weekend traffic. To analyze whether the directional split has a major influence on the final safety and monetary benefits, the authors performed the same set of sensitivity analyses using traffic data obtained from I-65 near West Lafayette, Indiana. This freeway section has almost equal distribution of traffic on weekdays and Saturday. On Fridays, the southbound volume to Indianapolis is heavier while on Sundays the northbound volume to Chicago is heavier. Guidelines have been prepared using both the cases.

6.3 Equal Directional Splits

6.3.1 The Effect of Traffic Volume

The traffic volumes ranged from 40000 veh/day to 60000 veh/day. High traffic volumes are rare and may not be applicable to rural work zones. However, they have been included in the analysis for purpose of illustration. When using queue constraint, the constraint was set at 10 km with the corresponding delay about 0.5 hrs. For all the considered cases, ILMS deployment length was calculated as a function of the maximum congested segment. The ILMS deployment length is calculated as,

$$\text{Deployment length} = 1.91 \cdot (L_c)^{0.56}, \qquad (6.1)$$

L_c is the length of the maximum congested queue. The maximum number of ILMS boards is limited to six and the maximum distance between two boards is limited to one kilometer. Therefore, the maximum ILMS deployment length was fixed at 5 km.

6.3.1.1 No Constraint Scenario

In the preceding sections we discussed the problem of infinite queue. From AADT value of about 53,000 veh/day onwards, the queue does not discharge at midnight. Therefore for the no constraint option, we limit our analysis to AADT value of 50,000 veh/day. It was found that for all the three capacity cases no safety impact is observed until 42,000 veh/day (Figure 6.2). Beyond this value of AADT, if ILMS does not reduce capacity, safety benefits are observed. For the capacity-decrease case, loss in safety is observed; the rate at which safety benefit decreases is relatively low and reaches 0.5

crashes. The maximum benefit for the capacity-increase case is three crashes saved. The no-change-in-capacity case lies somewhere between the two cases. In the capacity-decrease case, the reduction in the number of crashes is offset by increase in the level of congestion. Therefore, safety benefits for the capacity-increase case are much higher than those for the capacity-decrease case. As we can see from Figure A.1, in the capacity-increase case, the relative change in the number of crashes at lower AADT values of 45,000 veh/day is higher than that at higher AADT values. This is because at lower AADT values queues are smaller and therefore the entire congested segment is included in the ILMS deployment zone. For higher AADT values, the queues are longer than the maximum ILMS deployment length of 5 km. Therefore, the relative change in the number of saved crashes decreases.

The safety benefits in monetary values also follow the same trend as the number of crashes saved on approach. When the delay benefits are added to the safety benefits to obtain total benefits, the trend becomes different. This is because the delay benefits are much higher and they dominate the safety component (Figure 6.3). The total benefits for capacity effect = +76 veh/hr follow an upwardly trend and reach a maximum of about $ 1.2 M, while, for capacity decrease losses up to $ 1.5 M are observed.

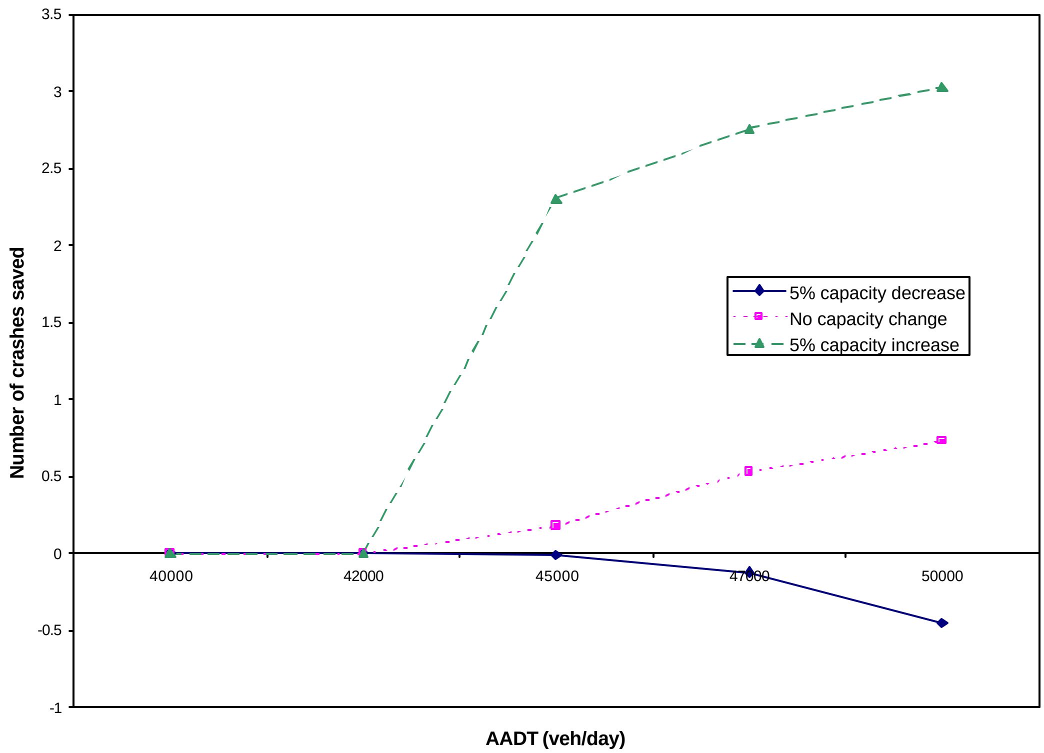

Figure 6.2: Number of crashes saved on approach (no constraints)

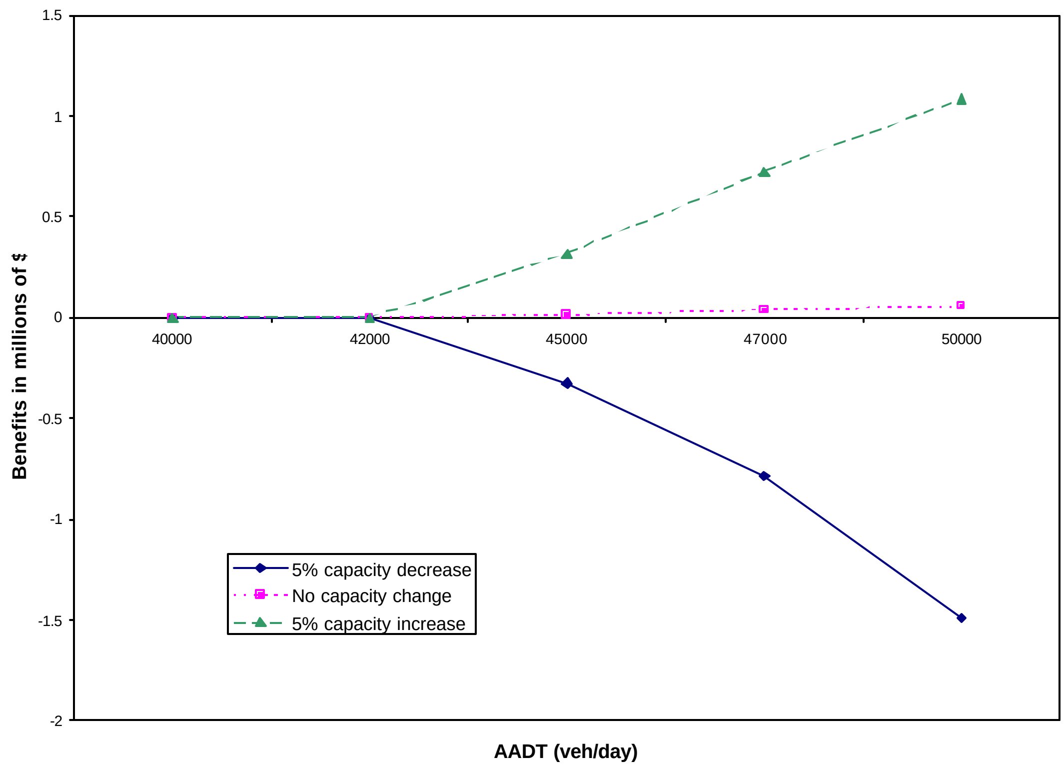

Figure 6.3: Total benefits (no constraints)

6.3.1.2 Queue Constraint Scenario

Since, queue and delay constraints are similar, it was decided to carry out the sensitivity analysis only for the queue constraint. Due to re-routing of vehicles a safety impact of ILMS is present on the approach and on the surrounding sections. The impacts are analyzed in relation to changes in traffic volumes. As before, AADT values between 40,000 veh/day and 60,000 veh/day are assumed. First, we analyze the safety impact on the approach to the work zones. For all the volumes, the maximum queue is fixed at 10 km. For all the capacity cases (capacity-increase, no-change-in-capacity, and capacity-decrease) we see a similar trend, i.e., no impact until AADT values of about 45,000 veh/day and then, a steady increase in the number of saved crashes (Figure 6.4). For low and moderate volumes, the change in the number of saved crashes in the capacity-decrease case is lower than the other two cases. However, for large volumes (AADT = 60,000 veh/day) the relative change in the number of saved crashes seems to be stronger in the capacity-decrease case than the other two cases. This is a rather surprising result. This might be due to the fact that in no-change-in-capacity case and capacity-increase case, fewer vehicles are re-routed. Since queue lengths are fixed, during the re-routing period the relative change in the number of conflicts with and without ILMS might be smaller when volumes are lower. For all capacity cases, crash savings occur on the approach and reach up to 2 crashes. This corresponds to a significant safety improvement on the approach.

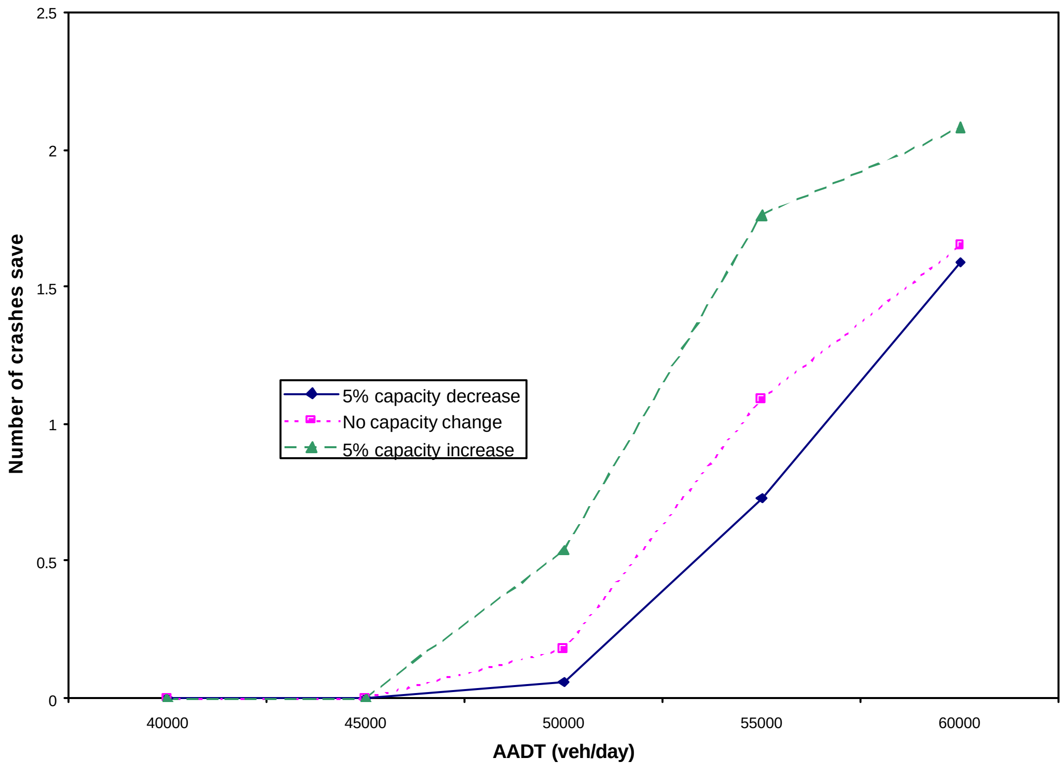

Figure 6.4: Number of crashes saved on approach (queue constraint = 10 km)

If only approach crashes are considered the results might be very confusing. A considerable safety improvement is concluded at high volumes even if ILMS reduces capacity of the work zone. This result is possible only when the shift of traffic to the alternate route and the increased hazard on the alternate route is not considered. It is necessary to include the safety impacts on the surrounding sections to have the complete picture. When we consider safety on all the affected road sections, then it is obvious that reducing work zone capacity decreases the positive impact of ILMS on safety (Figure 6.5). In the capacity-decrease case, safety benefits are not observed for any AADT values and safety impact goes on deteriorating with increase in AADT values.

The trend in total benefits (Figure 6.6) seems to be different from that in safety benefits. As volume increases, we should expect a steady increase in delay benefits for capacity-increase case and steady decrease for the capacity-decrease case. Instead, in both the cases the trends reverse at certain volumes.

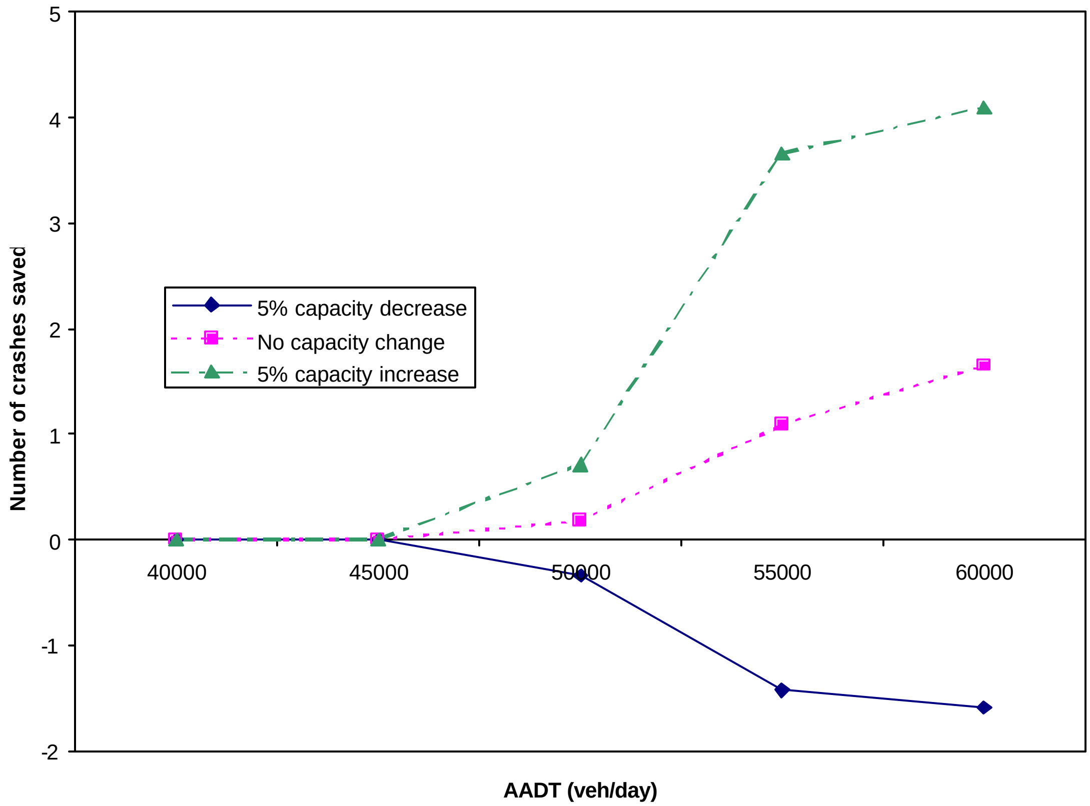

Figure 6.5: Total number of crashes saved (queue constraint = 10 km)

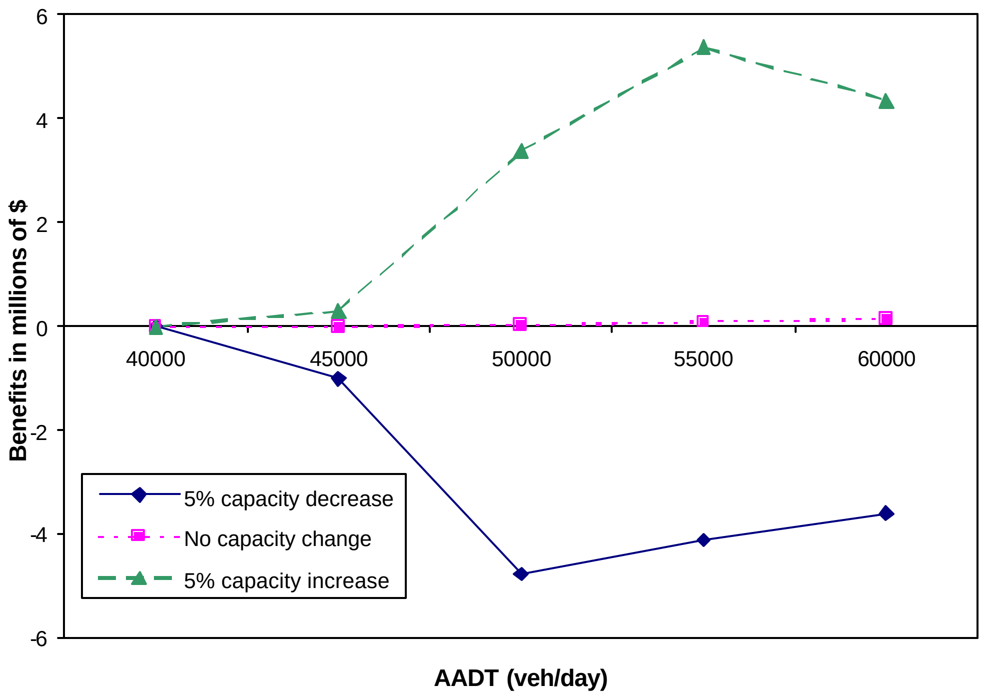

Figure 6.6: Total benefits (queue constraint = 10 km)

6.3.2 The Effect of Queue Constraint

The effect of maximum queue ranging from 4 km to 10 km on crashes and delays is analyzed. The ILMS deployment length was calculated as a function of congested segment with the maximum ILMS deployment length set as 5 km. AADT values of 50,000 veh/day and 60,000 veh/day were assumed. For an AADT value of 50,000 veh/day not much impact of queue constraint could be observed. Hence it was decided to analyze the impacts at a higher volume (60,000 veh/day) to see if there is any change in the general trends. As seen from Figure 6.7, the queue length has some impact on the number of crashes saved. We can see that in all the three capacity cases, the number of crashes tends to peak and then taper off. This is because the maximum ILMS deployment length was limited to 5 km. Hence, as the queues are near the deployment length we observe higher benefits. When the queue lengths become greater than the ILMS length the number of conflicts will naturally increase and we see a decrease in the number of crashes saved. The total benefits due to ILMS (Figure 6.8) shows a slight increasing trend for the capacity-increase case and a slight decreasing trend for the capacity-decrease case. However, for practical purposes the variation can be considered negligible.

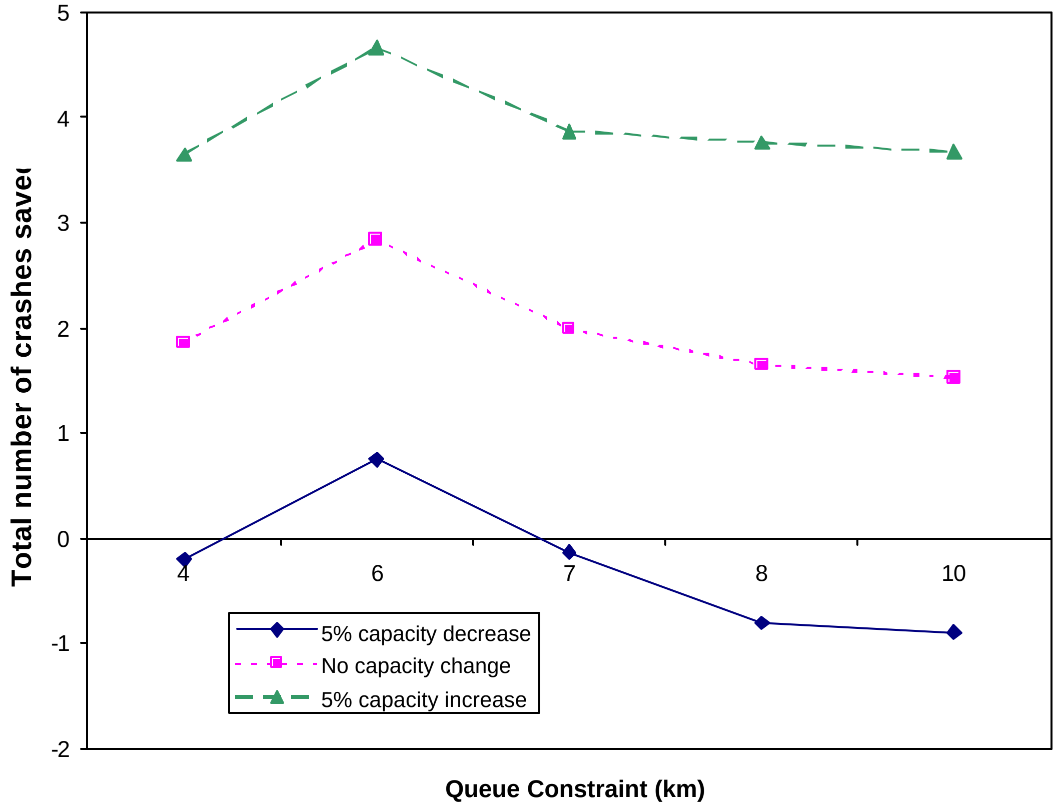

Figure 6.7: Total number of crashes saved (AADT = 60000 veh/day)

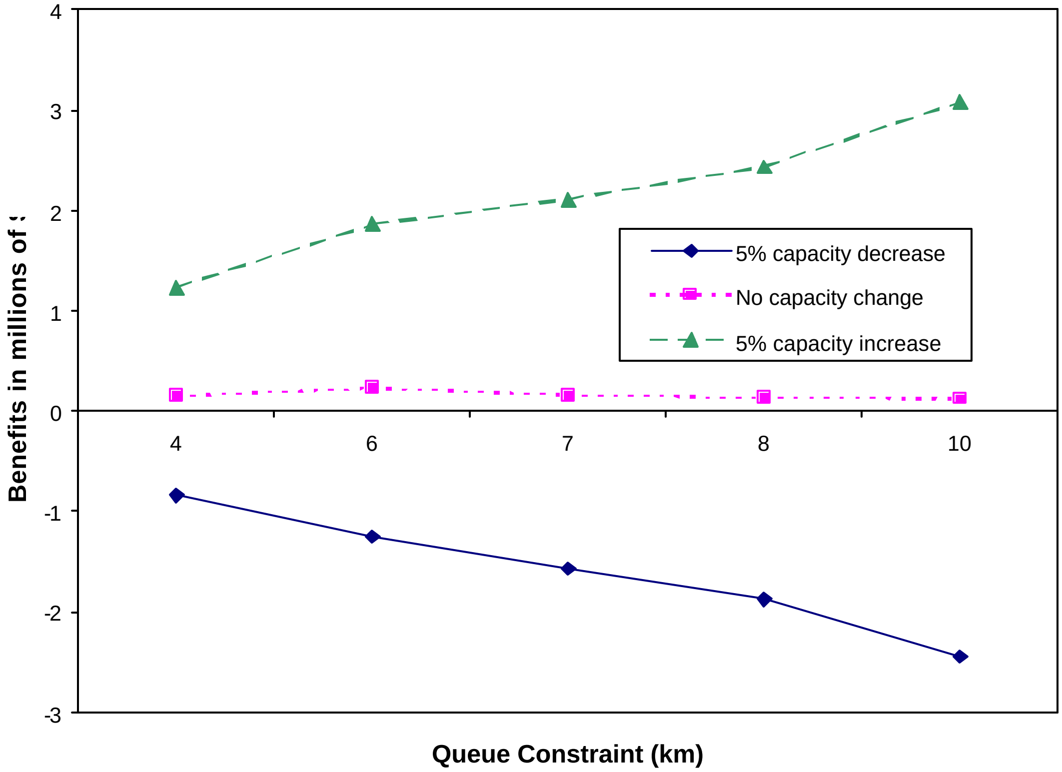

Figure 6.8: Total benefits (AADT= 60000 veh/day)

6.4 I-65 Data With Actual Directional Splits

6.4.1 The Effect of Traffic Volume

The range of traffic volumes considered here again ranged from 40000 veh/day to 60000 veh/day. The no-queue-constraint scenario has not been discussed here as the results obtained are very similar to the previous case. Also, for the queue-constraint case the trends are very similar. As for the case with equal directional splits, safety benefits on work zone approaches are observed for all capacity cases (Figure 6.9). But as before, the total safety benefits (Figure 6.10) follow different trends. We see an increasing trend with volume in safety benefits for the capacity-increase case and a decreasing trend for the capacity-decrease case. The trends for the total benefits (Figure 6.11) follow a similar pattern as before.

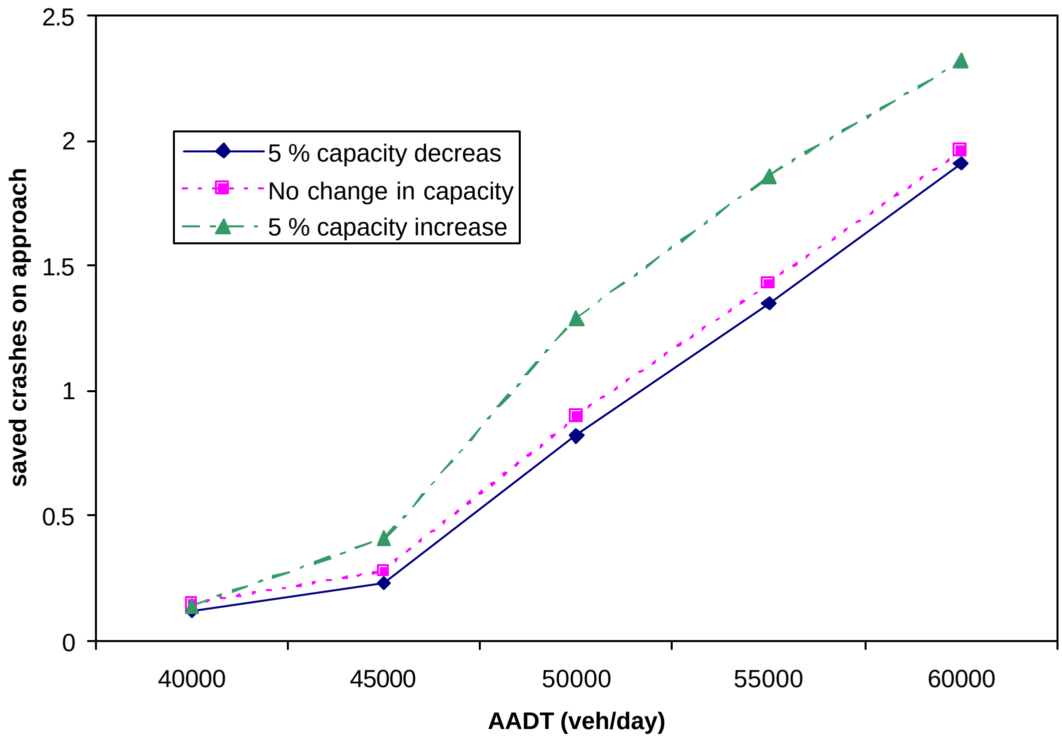

Figure 6.9: Number of crashes saved on approach (queue constraint = 10 km)

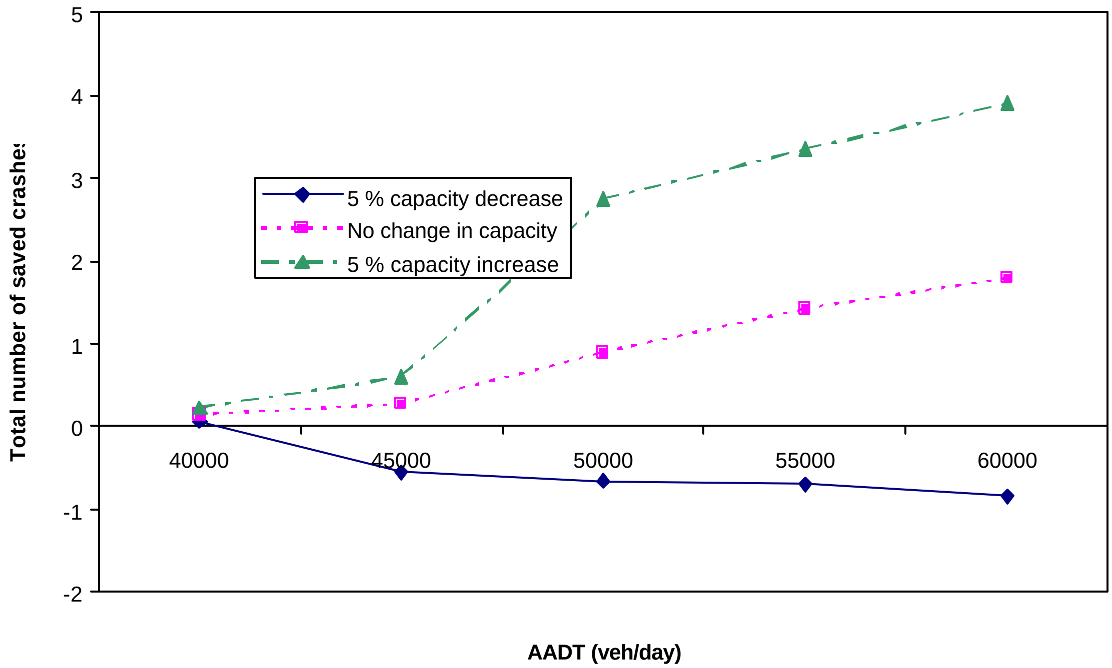

Figure 6.10: Total number of crashes saved (queue constraint = 10 km)

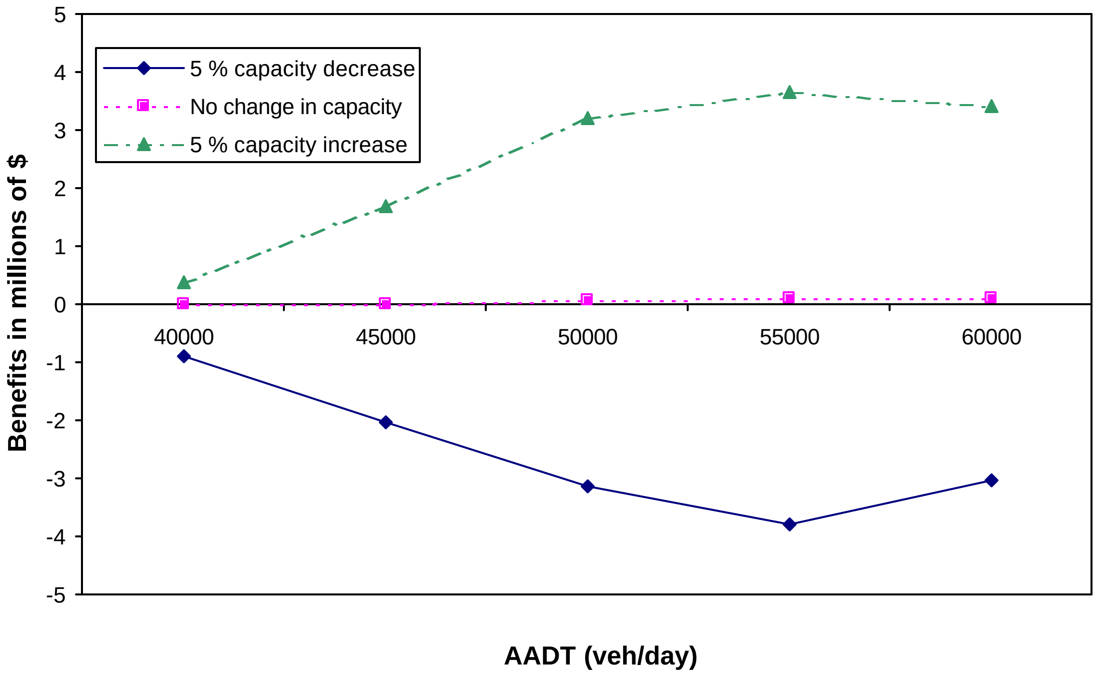

Figure 6.11: Total benefits (queue constraint = 10 km)

6.4.2 The Effect of Queue Constraint

Again the ILMS deployment length was calculated as a function of congested segment with the maximum ILMS deployment length set as 5 km. AADT value of 60,000 veh/day were assumed. As seen from Figure 6.12, the queue length has some impact on the number of crashes saved. We can see that in all three capacity cases, the number of crashes tends to peak and then taper off. This is because the maximum ILMS deployment length was limited to 5 km. The total benefits due to ILMS (Figure 6.13) shows a slight increasing trend for the capacity-increase case and a slight decreasing trend for the capacity-decrease case.

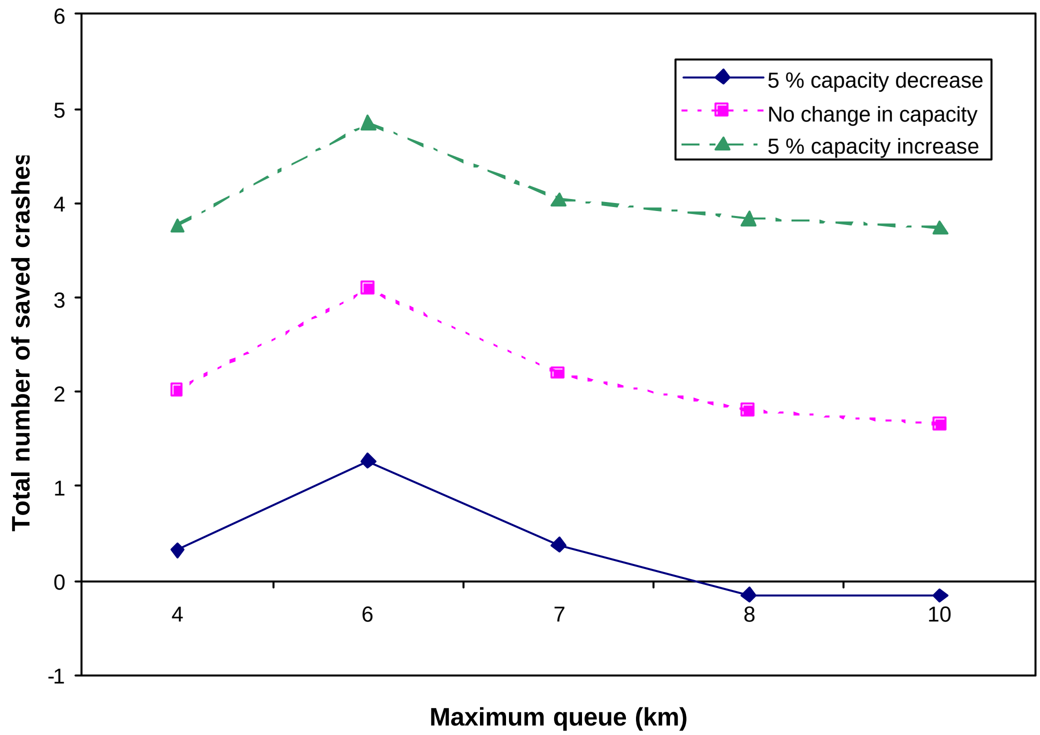

Figure 6.12: Total number of crashes saved (AADT = 60000 veh/day)

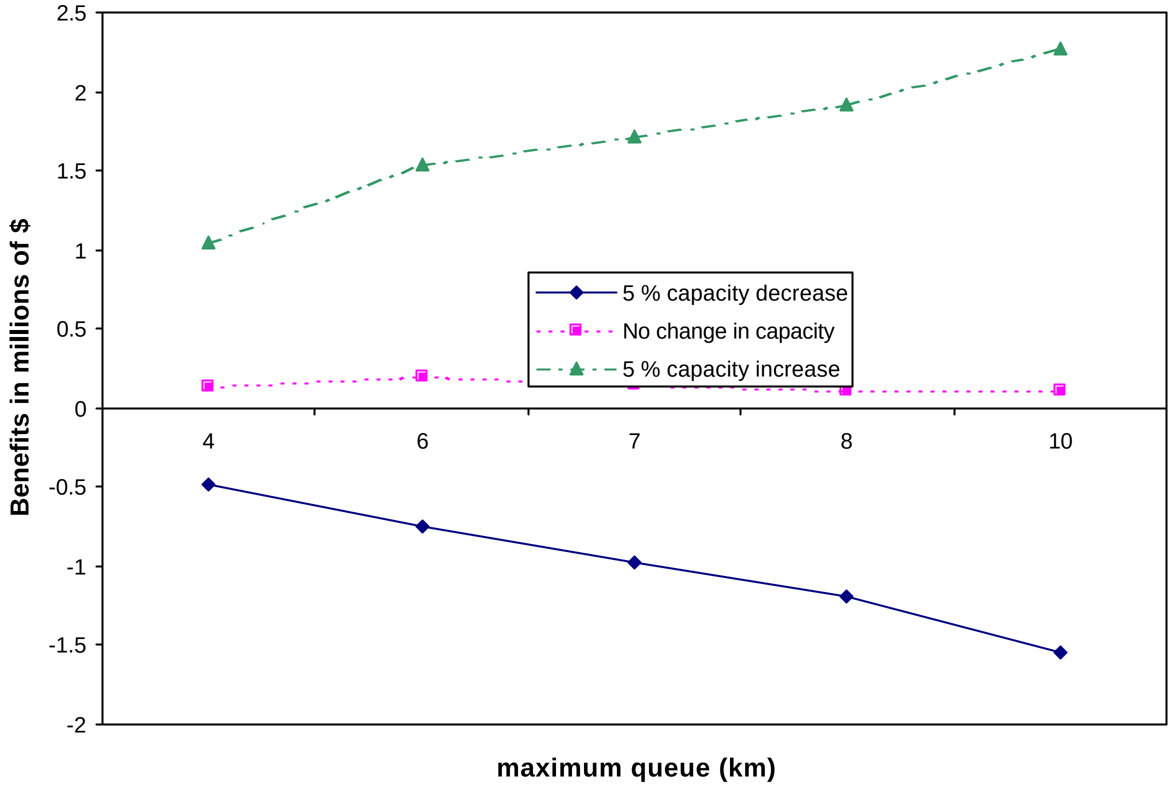

Figure 6.13: Total benefits (AADT= 60000 veh/day)

7. GUIDELINES FOR ILMS

The safety impacts of ILMS both on the approach and on the all affected road sections were investigated. Positive safety impacts are observed even when there is a reduction effect of ILMS on capacity. Since delay benefits are much larger than the safety benefits, the trends in total benefits of the system are driven mostly by the delay benefits. For the other two hypothetical capacity cases considered, no-change-in-capacity and capacity-increase considerable safety and delay benefits were concluded, irrespective of the traffic volume. The observations made in the sensitivity analysis can be laid down in a set of simple rules. These rules can then be used as a basis for estimating the expected safety and delay benefits at a given location and help the engineer/INDOT professional in deciding whether or not to implement ILMS at the location. The guidelines have been laid down separately for the case considering equal directional splits and for the case using actual splits form the I-65 data.

When no queue constraints are used,

1. ILMS should only be used in low to moderate volumes (up to AADT value of 50,000 veh/day).
2. The system does not cause any impact until AADT values reach 42,000 veh/day for all capacity cases.

3. No benefits are expected for the capacity-decrease case. ILMS deployment is not recommended if capacity is expected to decrease with ILMS deployment.
4. Benefits are expected for the no-change-in-capacity and the capacity-increase cases. ILMS deployment is recommended for such capacity cases.
5. Benefits from $ 0.5 M up to $ 1.0 M are expected for the capacity-increase case. Losses from $ 0.5 M up to $ 1.5 M are expected for the capacity-decrease scenario.

For queue constraints, the guidelines for deployment of Indiana Lane Merge System and expected benefits are given in Table 7.1.

Table 7.1: Guidelines for Indiana Lane Merge System using equal directional splits

	Traffic volume (AADT veh/day)			
	40,000 - 45,000	45,000 - 50,000	50,000 –55,000	55,000- 60,000
Capacity-decrease case (5%)	Zero safety and monetary benefits. *ILMS deployment not recommended*	Marginal safety benefits on the approach. No total safety and monetary benefits. *ILMS deployment not recommended*	Moderate safety benefits on the approach (around 1 saved crash). No total safety and monetary benefits. *ILMS deployment not recommended*	Moderate safety benefits on the approach (around 1.5 saved crashes). No total safety and monetary benefits. *ILMS deployment not recommended*
No-change-in-capacity case	Zero safety and monetary benefits. *ILMS deployment not recommended*	Marginal safety benefits (0.2 saved crashes) and monetary benefits. *ILMS deployment moderately recommended*	Moderate safety benefits (around 1 crash saved) and monetary benefits. *ILMS deployment recommended*	Moderate safety benefits (around 2 crashes saved) and monetary benefits. *ILMS deployment recommended*
Capacity-increase case (5%)	Zero safety and monetary benefits. *ILMS deployment not recommended*	Moderate safety benefits (1 saved crash) and high monetary benefits ($ 4M). *ILMS deployment highly recommended*	High safety benefits (4 saved crashes) and high monetary benefits ($ 6 M). *ILMS deployment highly recommended*	High safety benefits (5 saved crashes) and high monetary benefits ($ 4 M). *ILMS deployment highly recommended*

The other recommendations are,

1. The number of saved crashes on approach alone should not be used as a deciding criterion. Due to re-routing of vehicles along the alternate routes, safety levels along these routes may be affected as a result of ILMS. Hence, total safety impacts in the approach and all the affected roads should be used as a deciding criterion.
2. Safety benefits tend to increase with increase in ILMS deployment length.
3. Safety benefits tend to decrease when congestion (queue) lengths increase beyond the maximum ILMS deployment length.
4. The variation in total benefits with different maximum queue lengths is minimal.
5. The capacity impact of ILMS is a critical factor in deciding the expected safety and delay benefits from ILMS implementation.

A set of guidelines has also been prepared based on the sensitivity analysis results obtained from the I-65 data when actual directional splits have also been provided. Since both the sensitivity analysis results were very similar, the guidelines are practically the same. These guidelines have been given in table 7.2.

Table 7.1: Guidelines for Indiana Lane Merge System using actual directional splits from I-65 data

	Traffic volume (AADT veh/day)			
	40,000 - 45,000	45,000 - 50,000	50,000 –55,000	55,000- 60,000
Capacity-decrease case (5%)	Marginal safety benefits on the approach. No total safety and monetary benefits. *ILMS deployment not recommended*	Marginal safety benefits on the approach. No total safety and monetary benefits. *ILMS deployment not recommended*	Moderate safety benefits on the approach (around 1 saved crash). No total safety and monetary benefits. *ILMS deployment not recommended*	Moderate safety benefits on the approach (around 1.5 saved crashes). No total safety and monetary benefits. *ILMS deployment not recommended*
No-change-in-capacity case	Marginal safety benefits and monetary benefits. *ILMS deployment moderately recommended*	Marginal safety benefits (0.3 saved crashes) and monetary benefits. *ILMS deployment moderately recommended*	Moderate safety benefits (around 1 crash saved) and monetary benefits. *ILMS deployment recommended*	Moderate safety benefits (around 2 crashes saved) and monetary benefits. *ILMS deployment recommended*
Capacity-increase case (5%)	Moderate safety and monetary benefits. *ILMS deployment recommended*	Moderate safety benefits (1 saved crash) and high monetary benefits ($ 4M). *ILMS deployment highly recommended*	High safety benefits (3 saved crashes) and high monetary benefits ($ 4 M). *ILMS deployment highly recommended*	High safety benefits (5 saved crashes) and high monetary benefits ($ 4 M). *ILMS deployment highly recommended*

In the preceding sections, an attempt was made to determine the various impacts of the system so that a simple rule system can be used to help decide whether or not to implement the system at a particular location. The entire analysis was based on the models developed and the assumptions made about capacity impacts of the system. It was identified that capacity was the most crucial factor in deciding the final impacts of the system. Even though, the ILMS research team wanted to conduct further research on the capacity impact of ILMS, due to the unavailability of data, this had to be abandoned. Therefore authors recommend further research on the capacity impact of ILMS.

The sensitivity analysis and the guidelines are based on the assumption of daily profile values for rural and urban weekdays and weekends. The rural profiles were built using hourly counts at telemetry stations along I-65, I-69 and I-74. Similarly, urban profiles were also constructed. However, daily profiles can change from one location to another. Even a limited change in the daily profiles can cause significant difference in congestion levels if the rush hour volumes are close to the capacity values. Hence it is recommended that the developed evaluation tool be used with actual daily profiles for the investigated sites instead of approximate AADT values and typical daily profiles. If the actual profiles are not available, then the default daily profile values may be used with caution.

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LIST OF REFERENCES

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APPENDIX A

(Explanation of the components of the software program, example inputs and outputs)

Input values changed by the user										
	Day Type 1		Day Type 2		Day Type 3		Day Type 4		Day Type 5	
Time	veh prof	truck %	veh prof	truck %	veh prof	truck %	veh prof	truck %	veh prof	truck %
0:00-1:00			2.31	35.76	1.95	43.67				
1:00-2:00			1.83	43.67	1.49	47.19				
2:00-3:00			1.59	48.48	1.25	52.87				
3:00-4:00			1.58	51.54	1.27	55.19				
4:00-5:00			1.79	53.72	1.3	53.12				
5:00-6:00			2.77	41.22	2.61	38.22				
6:00-7:00			3.9	29.9	3.66	29.36				
7:00-8:00			4.2	22.94	3.68	16.67				
8:00-9:00			4.3	18.98	3.66	16.08				
9:00-10:00			4.73	20.31	4.199	16.4				
10:00-11:00			5.28	20.51	4.84	14.76				
11:00-12:00			5.48	20.58	5.41	14.09				
12:00-13:00			5.6	18.69	5.61	16.96				
13:00-14:00			5.63	15.71	5.74	14.16				
14:00-15:00			5.94	15.27	6.14	8.92				
15:00-16:00			5.89	12.51	6.44	5.95				
16:00-17:00			5.81	11.24	6.5	6.78				
17:00-18:00			5.7	8.28	6.54	10.12				
18:00-19:00			5.55	7.47	6.09	12.97				
19:00-20:00			5.02	12.83	5.77	10.79				
20:00-21:00			4.7	13.65	4.99	13.01				
21:00-22:00			4.23	17.43	4.28	21.54				
22:00-23:00			3.41	22.66	3.67	25.78				
23:00-0:00			2.76	33.1	2.95	43.16				

Figure A.1: The input table for daily vehicle profile values and daily truck profile values

	jan	feb	mar	apr	may	jun	jul	aug
	Day type 1		Day type 2		Day Type 3		Day Type 4	
% of duration of day type			71.43		28.57			
	jan	feb	mar	apr	may	jun	jul	aug
seasonal adjustment factor	1.096	1.169	1.073	1.051	1.01	0.964	0.903	0.917
Flow parameters								
AADT (veh/day)	60000							
Speed Limit on work zone approach	65	Mi/h						
Work Zone parameters								
Project Cost (in '000s of $)	8000							
Duration (months)	8							
Starting month	2							
Length of segment (km)	8							
Capacity Equation								
Capacity	1433	+	0	ILMS	-	4.04	truckpercent	
Capacity (under normal condns) = 1433								
ILMS correction (Default Value) = -76								
Alternate route information								
Length of alternate route	25	Km						
Distance from the end of the work zone to where the alternate route meets the freeway again	5	Km						

Figure A.2: The input section for work zone data, traffic data, capacity model and alternate route data

Daily vehicle profile represents the proportions of daily vehicle volumes in one-hour intervals. Due to cyclicity of traffic patterns the daily profiles will be same for a group of similar days. On rural locations, for example, the two most prominent groups are the weekdays and weekends. The convenience of using daily profiles and grouping together similar daily profiles is to make the computation simpler. By multiplying the daily profile values with daily volumes, hourly vehicle volumes can be obtained.

Daily truck profile represents the percentage of trucks in the total traffic population in one-hour intervals for a particular day. As before, the assumption is that the truck profiles like vehicle profiles will be similar for certain days and hence can be given by a single representative profile.

Different locations often have different significant *day types*. The assumption is that daily vehicle profile for a particular day type is the same for all days belonging to that day type. The assumption is true for daily truck profiles also. The user is provided with five default day types: (a) average day (b) average rural weekday (c) average rural weekend (d) average urban weekday and (e) average urban weekend. The default day types were constructed using actual values obtained from rural and urban interstates in Indiana. The user is also given the option of inputting his/her own day types. The default daily vehicle profiles and the default daily truck profiles are shown in Tables A.1 and A.2.The daily profile values provided in tables A.1 and A.2 are based on the assumption of equal directional split of traffic. But often, this need not be correct and hence another set of daily vehicle profile values and heavy vehicle values based on I-65 data with actual directional splits are used. These values are provided in tables A.3 and A.4. For this, the user would have input the daily profile values for one direction, compute the expected benefits in one direction and then input the corresponding values in the other direction and compute the expected benefits. These benefits may be then added to compute the total benefits.

After the user has chosen a particular day type, he has to input the *percentage duration* of that particular day type. For example, suppose the user has chosen rural weekday and rural weekend as the major day types. Suppose the total duration of work is 100 days and it has 70 weekdays and 30 weekends. Then the user should input 70% as the percentage duration for rural weekday day type and 30 % for rural weekend day type.

Table A.1: Default daily vehicle profiles provided to the user

Average day	Rural weekday	Rural Weekend	Urban weekday	Urban weekend
1.81	2.31	1.95	1.82	1.64
1.43	1.83	1.49	1.38	1.23
1.22	1.59	1.25	1.18	1.12
1.19	1.58	1.27	1.20	1.08
1.40	1.79	1.30	1.31	1.16
2.53	2.77	2.61	1.97	1.96
3.31	3.90	3.66	4.47	3.90
3.84	4.20	3.68	6.04	5.71
4.13	4.30	3.66	7.09	5.39
4.72	4.73	4.20	6.22	5.11
5.31	5.28	4.84	4.42	4.96
5.72	5.48	5.41	4.50	5.01
5.93	5.60	5.61	4.44	5.37
6.11	5.63	5.74	4.75	5.78
6.48	5.94	6.14	6.09	6.03
6.94	5.89	6.44	6.54	6.69
7.27	5.81	6.50	6.92	7.43
7.13	5.70	6.54	7.02	7.59
5.87	5.55	6.09	6.01	6.11
4.75	5.02	5.77	4.96	4.78
4.03	4.70	4.99	3.91	3.86
3.51	4.23	4.28	3.47	3.12
2.99	3.41	3.67	2.93	2.79
2.37	2.76	2.95	2.28	2.18

Table A.2: Default daily truck profiles provided to the user

Average day	Rural weekday	Rural weekend	Urban weekday	Urban weekend
38.65	35.76	43.67	51.25	44.94
45.43	43.67	47.19	56.19	50.81
51.49	48.48	52.87	59.11	55.31
57.26	51.54	55.19	56.85	47.05
54.15	53.72	53.12	53.44	43.79
43.22	41.22	38.22	52.67	42.95
30.77	29.9	29.36	40.22	40.49
17.97	22.94	16.67	29.13	28.55
16.77	18.98	16.08	27.29	27.03
16.21	20.31	16.4	24.78	25.67
15.66	20.51	14.76	25.1	25.38
14.24	20.58	14.09	23.03	24.13
18.33	18.69	16.96	24.49	26.41
14.15	15.71	14.16	23.23	23.69
9.55	15.27	8.92	21.36	20.45
6.54	12.51	5.95	20.61	18.57
7.89	11.24	6.78	20.42	19.15
9.89	8.28	10.12	17.04	18.46
10.77	7.47	12.97	17.82	19.29
11.59	12.83	10.79	19.25	20.42
13.11	13.65	13.01	21.61	22.36
19.3	17.43	21.54	25.17	27.24
23.43	22.66	25.78	31.79	32.61
36.76	33.1	43.16	40.06	43.41

Table A.3: Default daily profile values for the north bound direction from I-65

Average Weekday + Saturday		Friday		Sunday	
			151		
veh profile	truck profile	veh profile	truck profile	veh profile	truck profile
0.89	35.94	0.88	46.33	0.57	41.33
0.71	40.95	0.96	44.67	0.39	48.72
0.62	50.92	0.79	51.65	0.41	50.71
0.59	52.31	0.78	57.89	0.37	56.55
0.69	52.06	0.84	53.51	0.35	51.68
1.28	40.61	0.81	37.82	0.39	39.23
1.67	29.98	1.27	29.43	0.63	29.97
1.9	22.95	1.72	14.31	1.08	20.32
2.07	22.7	1.97	14.48	1.68	18.1
2.36	17.39	2.26	18.8	2.36	14.31
2.63	17.8	2.58	14.67	3.34	15.06
2.87	20.87	2.71	12.83	3.97	17.5
3.02	16.67	2.61	14.72	4.25	20.94
3.04	12.72	2.8	12.29	4.61	13.14
3.24	16.41	2.89	8.35	4.65	8.52
3.41	11.76	2.92	5.15	4.89	7.71
3.69	13.62	3.05	7.59	4.41	6.74
3.62	9.36	2.85	7.63	4.59	10
2.95	8.31	2.76	10.64	4.28	13.2
2.36	15.74	2.43	14.43	3.27	11.88
1.99	13.4	2.1	12.64	2.42	14.19
1.73	20.79	1.75	22.54	2.11	20.36

1.49	25.3	1.32	28.09	1.66	28.05
1.16	34.56	1.06	44.46	1.17	46.79

Table A.3: Default daily profile values for the south bound direction from I-65

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Average Weekday + Saturday		Friday		Sunday	
veh profile	truck profile	veh profile	truck profile	veh profile	truck profile
0.91	33.47	0.94	47.53	0.54	41.25
0.71	39.63	1	48.76	0.4	46.32
0.61	43.99	0.86	55.81	0.34	55.74
0.59	48.13	0.94	55.38	0.27	53.42
0.7	49.75	0.88	50.95	0.34	55.97
1.27	37.88	1.18	35.83	0.46	38.68
1.69	26.64	1.79	31.81	0.73	32.37
1.93	20.91	2.13	16.71	1.12	19.84
2.05	16.91	2.14	14.67	1.49	13.3
2.32	17.93	2.13	16.37	1.88	14.08
2.6	17.71	2.2	18.31	2.33	13.72
2.87	16.77	2.67	17.42	2.69	17.61
2.97	15.78	2.85	18.44	2.72	16.49
3.01	13.32	3.31	13.97	2.72	14.5
3.29	12.97	3.32	7.23	2.85	11.36
3.54	8.96	3.65	3.48	3.01	7.87
3.59	7.84	3.28	9.59	3.47	5.57
3.58	4.12	3.75	10.35	3.13	11.37
2.91	3.77	3.85	12.49	2.89	14.57
2.39	10.06	3.42	13.14	2.58	12.19
2	9.9	2.71	12.49	2.3	14.58
1.74	13.7	2.27	24.41	1.68	19.3

1.5	20.61	1.58	26.9	1.27	23.51
1.2	29.38	1.05	45.3	0.97	40.77

The user is asked to input the *average annual daily traffic* for the particular location. This is then converted to average daily traffic (ADT) values using proper *seasonal adjustment factors* corresponding to the months given for months with work activities. Default adjustment values were provided by the INDOT roadway management division. The user can input his/her own values.

Another set of input data includes *work zone parameters* such as the length of the work zone, the expected costs of the project, the month when the work starts and the duration of the work.

The *capacity equation* (Chapter 5) developed using the I-65 work zone data was used as the default capacity equation. The user is given the freedom to overwrite any of the values. For example, if the user feels that the actual ideal capacity is higher than the default value of 1433 veh/hr, he/she can change it. If the user feels that ILMS does not affect capacity, then he/she can use zero capacity change instead of the default value of –76 veh/hr. As mentioned in Chapter 5, the author felt that the decreasing capacity effect of ILMS may be only temporary.

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Intermediate Results

Day type	1	2	3	4	5
With ILMS					
Congestion start time		91	101		
Congestion end time		233	235		
Maximum congestion length in km		10	10		
Total delay		5860.794476	2249.382978		

	1	2	3	4	5
Without ILMS					
Congestion start time		91	101		
Congestion end time		233	235		
Maximum congestion length in km		10	10		
Total delay		5860.794476	2249.382978		

Figure A.3: Expected delay with and without ILMS for the day types considered

Final Results

	With ILMS	Without ILMS	
No. of braking conflicts/day	233.29	286.15	
No. of merging conflicts/day	254.48	313.42	
Delay (in veh hrs/day)	8110.18	8110.18	
Crash Statistics without ILMS			
Brake PDO crashes w.o ILMS	4.02		
Brake injury + fatal crashes w.o ILMS	1.65		
Merge PDO crashes w.o ILMS	2.7		
Merge injury + fatal crashes w.o ILMS	0.54		
Reduction in rear end crashes		Crash Cost Savings	
Crash reduction, rear end pdo	0.74	Cost Savings, rear end pdo	3331.31
Crash reduction, brake injury + fatal	0.3	Cost Savings, rear end Injury +fatal	96080.98
Reduction in merging crashes		Crash cost savings	
Crash reduction, merge pdo	0.51	Cost Savings, Merge pdo	2276.74
Crash reduction, merge injury+fatal	0.1	Cost Savings, merge injury + fatal	32347.48
Decrease in delay with ILMS			
Reduction in total delay/day	0		
Total reduction in delay	0	Total delay costs saved	0
Total Crash costs	134036.52		
Total Delay costs	0		

Figure A.4: A part of the output sheet showing the expected delay benefits and safety benefits (on the approach)

Surrounding zone safety impact

	with ILMS	without ILMS
Alternate route crashes	3.43	3.43
Work zone crashes	3.4	3.4
Remaining distance	1.84	1.84

Benefits in the surrounding zone due to ILMS

	Crashes saved	Cost Benefit
Alternate route crashes	0	0
Work zone crashes	0	0
Remaining distance	0	0

Overall Benefits of the system

Total crashes saved on approach	1.65
approach + network	1.65
Benefits in $ (on approach)	134036.51
Safety benefits (approach + network)	134036.51
Delay costs	0
Total Benefits	134036.51

Figure A.5: Output sheet showing the total safety and total monetary benefits

The output data contains the following information:

1. Expected number of merging and braking conflicts with and without ILMS (per day),
2. Total delay with and without ILMS,
3. Expected number of approach crashes without ILMS by crash category and severity level,
4. Reduction in number of crashes by crash category and severity level,
5. Safety benefits in each crash category and severity level in dollars,
6. Delay reduction/increase due to ILMS,
7. Delay benefits in dollars,
8. Expected number of crashes on the work zone, alternate route, and the remaining freeway,
9. Safety benefits in all these sections,
10. Total benefits due to the system.