A STUDY OF THE RELATIONSHIPS BETWEEN ROAD ACCESS, TRAFFIC SAFETY AND TRAVEL SPEED, AND APPLICATIONS TO ACCESS PLANNING AND SPEED ZONE DESIGN

by

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THE UNIVERSITY OF BRITISH COLUMBIA

January, 1996

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Date March 21, 1996
ABSTRACT

This thesis investigates the impacts of highway access on traffic safety and traffic operation efficiency, and develops a model for planning access provisions. A comprehensive data base was created for the analyses by taking advantage of an existing photographic log of selected sectional highway information for one Canadian province. The data base includes information on access types, access density, traffic volume, speed limit, geometry, and accident records.

A review of the existing traffic situation and an examination of some established relationships of access and accidents indicated a need to update current knowledge of the impact of highway access. Accident models are developed with particular emphasis on access and its combined effects with traffic volume and road geometric factors on highways. Furthermore, a conceptual hazard model is developed which includes traffic conflicts as additional information to extend the knowledge of access as a road hazard. The literature on access and traffic operation relationships is sparse, particularly on two-lane highways, perhaps because of the high cost of comprehensive data collection for this complex problem. Consequently the present investigation is restricted to the use of a surrogate of average travel speed to define access and speed relationship.

In addition to the investigation of access impact, a framework for determining the optimal number of access points is formulated to help to set up access planning criteria. The optimization process is the
application of the defined relationships between access and traffic safety and operations in highway planning and design. The process minimizes the total social cost as an integer nonlinear programming problem, solved by piecewise approximation. Finally, the relevant issues of access and speed zone design are addressed, in which particular attention is given to deriving speed transition zone length.

The derived accident models show that accesses are major contributing factors of accidents on all types of highways. In some situations, the combined effects of access and geometric factors undermine further the safety level. The analysis for two-lane highway indicates that non-linear relationships may exist between accesses and travel speed. The derived optimization framework for planning access provisions provides a quantitative base for functional classification of roads. In traffic control design, a procedure of speed zone design is outlined with consideration of access provisions.
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Chapter One
INTRODUCTION

1.1 The Background

Recent concerns over pervasive problems of road safety and deteriorating levels of service of highways have intensified interest in roadway access management as an effective non-capital intensive means to ameliorate concerns of traffic safety and roadway traffic operations. Access gives utility to a road network, providing benefits to road users and to those who make use of the abutting lands. On the other hand, access can be a major problem to main road traffic in terms of delay and accidents and the cost implications of these. A review of the literature indicates substantial research has been done in the past on various aspects of this problem as reflected in existing references. The literature review points to a need for new studies on the relationship of access density and roadway traffic to: (1) update past knowledge of the subject; (2) examine the subject at both the strategic (or planning) level and the tactical (or traffic engineering) level; (3) cover as many road classes as practically possible; (4) derive an optimization framework in planning access points. Thus, the overall purpose of the present study is to investigate the impact of various road access types on traffic safety and operations; and from the empirical studies undertaken in the project to develop an optimization model for planning access provisions.
Adequate traffic access for abutting lands and efficient traffic flow on main roads are two fundamental, but contradictory, functions of a highway system. Specifically, speed differences between through traffic and accessing traffic, and vehicle maneuvers (crossing, merging, and weaving) set up potential for traffic delays, driver confusion and motor vehicle accidents. Reducing access points per unit distance (access density) can enhance traffic service on the main road and reduce accidents. On the other hand, increasing access points reduces driver delay for accessing traffic. The classical approach to resolving this dichotomy is to develop a hierarchical highway system in which each class will enhance one function (through traffic vs. access traffic) over another. Including all specialized functions in one network system will ideally provide the functions needed for all traffic. Some classes mainly serve the movement of high speed inter-urban traffic (freeway and arterial classes) while other classes mainly serve abutting land access (collector roads and local roads). The hierarchy range in practice consists of full access control (i.e., no access) partial access control and no access control. This functional classification of highways and streets has been widely accepted and used by highway authorities.

In the functional classification for urban and rural roads as documented in the Manual of Geometric Design Standards for Canadian Roads (Roads and Transportation Association of Canada, 1986) the freeway classification is relatively straightforward: with "no accesses" allowed. However, for the lower classes of roads the relative priority suggested for through traffic versus access traffic is not well defined. This is particularly the case for two-lane rural roads (nominally
"arterial" class) for which there is no specific guidelines for the type of access, nor the density of access associated with this class. A second problem with the functional classification system as documented is the absence of a safety criterion in the definition. Conventional wisdom is that freeways are the safest class, but again the lower classes are not well defined in terms of relative safety. Insights gained in the study of access and traffic may be useful to refine the classification of roadways and is consequently one objective of the present study.

At the planning level of analysis, the project objective is to assess the balance between delay and hazard resulting from a large number of access points (access density) which provide the accessibility needed for abutting lands. Since increased access density provides utility for various land use activities such as neighborhood, commercial and industrial economic activities it is important for the purposes of this present study to determine the appropriate access density which is complementary to a desired level of traffic service. Given the need for access in order to provide utility to the road system, traffic operational imperatives can be reduced to the need to provide optimal levels of road safety and traffic flows consistent with land use utility planning. The operation problem then becomes one of balancing the benefits of access for entering/existing traffic against the disruption of flow and the accident potential for main road traffic. One prime objective of this study is to develop an optimizing framework which could be used to identify appropriate access points consistent with traffic flow levels and which would minimize total cost of delay and
accidents. Such a framework will provide the traffic planner with some quantitative guidelines for planning access points for a road section, and coincidentally allow revision in the highway classification system.

At the traffic operations and road design level of analysis, the concerns are with questions related to the relative mix of access types, the relative location of access points, the effect of access points on accidents and traffic efficiency. Since there are many road, driver, and vehicle factors which can impact on accident potential and traffic operation efficiency, the role of access as a negative influence needs to be separated from other factors. Some previous studies (e.g. Dart and Mann, 1970) have documented the relationship between accident rates and access points. However, most previous studies were conducted decades ago. One objective of the present study is to examine the understanding of the access/accident relationship from past studies, and update the quantitative relationship between access and accidents, using an existing data base from one Canadian province. Unlike access/accident studies, relatively few references are available which examine the effect of access on operational efficiency. The Highway Capacity Manual (3rd edition, 1994) recommends a speed reduction factor for the crude number of access points per mile for multilane rural and suburban highways, but not for two-lane highways. Therefore, a further objective of this present study is to determine the quantitative relationship between access and traffic efficiency for two-lane highways. The third objective is to improve speed zone design, particularly speed transition zone design, with consideration of actual access density and land use development.
1.2 Access and Traffic Safety

Studies conducted to reveal the relationship between access and accidents, such as Kihlberg and Tharp (1968), Box and Associates (1970), and Dart and Mann (1970), have found that access control has an important impact on traffic safety. However, access is often taken to be a broad concept, with implications for both highway planning and traffic operations. At the planning level, access provides utility for various activities, such as neighborhood, commercial, and industrial activities, and is intimately linked to land use considerations. In traffic operational terms, access can mean entry to traffic for individual vehicles, or to exit from the main traffic stream. The later case may represent an "access" to local streets. In this sense of traffic operation, we may define "access" as the "way" of vehicles moving from one transportation facility to another.

The relationship between access and safety can then be described for both cases: the undesirable events (accidents) of providing utility of transportation facilities, and the undesirable events resulting from vehicles moving from one facility to another. In the first case, we deal with the issue at a macroscopic level, the concern being centered mainly on the frequency and magnitude of the interference resulting to through traffic from vehicles joining to or crossing the through traffic. Specifically, it is the relationship between accident occurrence and the frequency and spacing of access points. In the second case, we deal with the issue at a microscopic level, as the relationship between accident occurrence and the geometric design elements of highways.
1.2.1 Access and Safety at the Macroscopic Level

At the macroscopic level, provision of access control is vital to a road in terms of service and safety. As stated in AASHTO (1990), "With control of access, entrances and exits are located at points best suited to fit traffic and land-use needs and are designed to enable vehicles to enter and leave safely with a minimum of interference with through traffic". "If access points are adequately spaced, and entering and exiting volumes are light, the street or highway functions efficiently. If access points are numerous, and entering and exiting volumes are heavy, the capacity and safety of the facility are reduced" (AASHTO, 1990). Thus trade-offs must be made to effectively serve through traffic and land-use establishments while at the same time observing a safety constraint. One recent report (Bochner, 1991) provides guidance in planning site access in urban areas. However, Bochner's emphasis is put on the forecasting of traffic generation, distribution, and assignment. The safety factor was mentioned but the actual relationship between access and accidents was not investigated.

AASHTO in its study (1990) indicated that "the most significant factor contributing to safety is the provision of full access control". The beneficial effect of this factor has been documented in many reports, such as in Interstate System Accident Research (U.S. Department of Transportation/Federal Highway Administration, 1970). One of the principal findings of the FHA study is that the absence of access control invariably increased accident rates. Furthermore, other studies
(Stover et al, 1970, and OECD, 1976) indicated that the number of access points and the amount of traffic entering the traffic stream at these points determines the overall accident rates for any particular length of highway. A study involving rural highways in the USA found roughly a seven percent rise in accident rates for each additional access point per mile (Highway Users Federation for Safety and Mobility, 1970). Besides, access-controlled motorways of up to date design standards constitute the safest and most efficient routes with substantially lower accident risks when compared to other road classes (OECD, 1986, and Prisk, 1957). The provision of restricted access by-passes at seven sites in the UK resulted in a net reduction of injury accidents of the order of 25 percent (Newland and Newby, 1962). The effectiveness of access control increases as average daily traffic increases (Smith, W. and Associates, 1961).

Roy Jorgensen Associates, Inc. (1978) presented a systematic analysis on access control for the following categories of roads: multilane divided rural highways, multilane undivided rural highways, two-lane rural highways, and urban arterial highways. The following review summarizes the Jorgensen report. In addition, however, residential collector roads are also discussed.

1.2.1.1. Multilane Divided Rural Highways

AASHTO (1990) indicated that the accident rate increases with an increase of business activity and at-grade intersection density. Figure 1.1 shows the relationship for 4-lane divided highways. The figure shows
that the influence of intersections increases with the increase of intersection density, and the effect of business activity increases at the same rate for each intersection access density.

It is also demonstrated in Kihlberg and Tharp's report (1968) that as the access control increases, the accident rate on four-lane divided rural highways decreases. Results showing accident rates for different levels of access control are shown in Table 1.1. As one would expect, with increasing access density, the increase in single vehicle accident (e.g., run-off road) is less than the increase in multivehicle accident, because of the increased conflict between vehicles entering and exiting with partial or no access control (higher density of access).

<table>
<thead>
<tr>
<th>Access Control</th>
<th>Accident Rates *</th>
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<tbody>
<tr>
<td></td>
<td>Total</td>
<td>Multi-vehicle</td>
<td>Single-vehicle</td>
</tr>
<tr>
<td>Full</td>
<td>0.67</td>
<td>0.38</td>
<td>0.31</td>
</tr>
<tr>
<td>Partial</td>
<td>0.91</td>
<td>0.58</td>
<td>0.33</td>
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<tr>
<td>None</td>
<td>1.84</td>
<td>1.38</td>
<td>0.42</td>
</tr>
</tbody>
</table>

* Mean Annual No. of Accidents per 0.3-mile (0.4827 km) Segment.  
Source: Kihlberg and Tharp (1968)

Table 1.1 Accident Rates Correspondent to Different Levels of Access Control(Multilane Divided Rural Highways)

Figure 1.1 Accident Rate on 4-lane Divided Non-Interstate Highways by Number of At-Grade Intersections per Mile and Number of Businesses per Mile
1.2.1.2 Multilane Undivided Rural Highways

The Kihlberg and Tharp study (1968) included all rural highways. A decrease in accident rates from those with no access control, to partial, to full, for divided four-lane facilities was observed. Table 1.2 shows smoothed rates (mean annual number of accidents per 0.3-mile segment) for rural highways in Ohio with a range of traffic volumes from 4,600 to 6,899 ADT. In comparing Table 1.2 and 1.1 it is clear that accesses on two-lane highways have a dramatic impact on the accident rate compared to multilane divided highways, particularly on multivehicle accident rates, although two-lane highways with no access control appear to be safer than multilane divided highways with no access control. In addition, four lane undivided highways as illustrated by Table 1.2 have a very much higher accident rate than all multilane divided highways shown in Table 1.1; as one would expect.

<table>
<thead>
<tr>
<th>Highway Type</th>
<th>Total</th>
<th>Multi-Vehicle</th>
<th>Single-Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-lane, no access control</td>
<td>1.75</td>
<td>1.22</td>
<td>0.45</td>
</tr>
<tr>
<td>2-lane, partial access control</td>
<td>1.39</td>
<td>1.06</td>
<td>0.32</td>
</tr>
<tr>
<td>4-lane, no median, no access control</td>
<td>2.56</td>
<td>1.98</td>
<td>0.48</td>
</tr>
</tbody>
</table>

Source: Kihlberg and Tharp (1968)

Table 1.2 Accident Rates Correspondent to Different Levels of Access Control (2-lane/4-lane Rural Highways)
1.2.1.3 Two-lane Rural Highways

Several studies have shown that as the roadside development increases and the number of entrances increases, the accident rate increases. Dart and Mann (1970) found that the accident rate increases as the conflicts per mile increase for rural highways in Louisiana, as shown in Figure 1.2. Traffic conflicts were defined by Dart and Mann as the total number of traffic access points (on both sides) per mile of highway section and included only minor road intersections and principal access driveways to abutting property along highway sections. Intersections with major roads were considered as break points between study sections.

Fee et al (1970) employed an especially large accident data base with an example of the results shown in Table 1.3 which relates access points per km to the accident rate. The data shown are for an ADT of 8,140 vehicles. It may be noted from Table 1.3 that a ten-fold increase in the number of accesses to the highway more than doubles the accident rate; a 100-fold increasing access points raises the accident rate approximately 14 times.

In the study mentioned in section 1.2.1.1 and shown in Table 1.2, Kihlberg and Tharp (1968) demonstrated clearly that: (a) two-lane highways had lower accident rates than four-lane highways when there was no median and no access control; (b) access control had the most powerful accident reducing effect, and partial control access was also effective.
Figure 1.2 Accident Rate Versus Traffic Conflicts  
(Two-lane Rural Highways)

<table>
<thead>
<tr>
<th>Access points per km</th>
<th>Businesses per km</th>
<th>Accident Rate*</th>
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</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.6</td>
<td>78</td>
</tr>
<tr>
<td>1.2</td>
<td>6.0</td>
<td>167</td>
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<tr>
<td>12.0</td>
<td>60.0</td>
<td>1060</td>
</tr>
</tbody>
</table>

* Rate is number of accidents per 100-million vehicle-km.

Source: Fee et al (1970)

Table 1.3 Accident Rates Correspondent to Different Levels of Access Density(2-lane Rural Highways)
Prisk (1957) found that head-on, angle, and pedestrian collisions comprise 10 percent of all freeway accidents, while facilities with partial or no access control experience 35 to 50 percent accidents in those categories. The total accident rate, fatality rate, and injury rate on full access-control facilities were found to be lower than on partial access control or no control for both rural and urban facilities. The following results were found for the period 1949 to 1955 as shown in Table 1.4. As expected, freeways with full access control have the most effect on reducing fatal accidents.

Schoppert (1957) demonstrated that access to the highway by driveways or intersections is directly correlated to accidents at all ADT levels. The number of access points is a reasonably good predictor of the number of potential accidents within an ADT group, although the most important factor in the prediction of traffic accidents is the vehicle volume on the highway. The number of accidents increases with the number of situations presenting a change in conditions and thus requiring a decision on the part of the motor vehicle operator. Accidents increase when (a) vehicle volumes increase, (b) access points increase, and (c) sight distance is impaired and/or cross section is reduced. While volume is the best predictor, the number of points of access is second in importance. Design features such as lane width, shoulder width, and sight restrictions are third.

Raff (1953) found that the more roadside establishments per mile, the higher the accident rate, although this was significant only for one set of data (two-lane tangent sections).
<table>
<thead>
<tr>
<th>Type</th>
<th>Fatalities per 100 mvm*</th>
<th>Injuries per mvm*</th>
<th>Total Accidents mvm*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural: full control</td>
<td>3.4</td>
<td>1.36</td>
<td>1.56</td>
</tr>
<tr>
<td>Rural: partial control</td>
<td>6.3</td>
<td>1.48</td>
<td>2.09</td>
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<td>Rural: no control</td>
<td>10.3</td>
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<tr>
<td>Urban: full control</td>
<td>2.2</td>
<td>0.89</td>
<td>1.94</td>
</tr>
<tr>
<td>Urban: partial control</td>
<td>5.6</td>
<td>1.64</td>
<td>5.23</td>
</tr>
<tr>
<td>Urban: no control</td>
<td>4.1</td>
<td>2.61</td>
<td>5.01</td>
</tr>
</tbody>
</table>

* mvm represents million-vehicle-miles.
Source: Prisk (1957)

Table 1.4 Fatalities and Injuries at Different Access Control Level
(2-lane Rural and Urban Highways)

1.2.1.4 Urban Arterials

Generally, all major urban arterial studies reviewed found control of roadside access factors to be second only to exposure (traffic volume) in importance for road safety. Specifically, four safety relationships were established by several studies:

A. As accesses increase, the total accident and injury rates increase.

Cribbins and Aray et al (1967) used an access point index as a measure of roadside access points and their volumes. The access point index is the estimated total of all movements entering or leaving the site from commercial and industrial roadside development, private
drives, and intersecting roadways expressed on a per mile basis. Thus, the index is proportional to the number of conflict points along the facility and the exposure at the points. The independent variables used in the predicting equation are: access-point index, signalized openings, speed limit, volume, and level of service.

B. As the number of commercial driveways per mile and/or commercial units per mile increases, the accident rate increases.

Mulizzani and Michael (1967) conducted research in which 100 study sections were stratified by type of arterial. It was concluded that the number of median and heavy volume commercial driveways per mile was significantly related to the accident rate for low-volume (1,200 to 5,800 ADT) sections. The accidents per mile for an urban arterial can be expressed using the following variables: volume (ADT) on the section in thousands of vehicles; number of heavy volume intersections per mile; number of traffic signals per mile; number of heavy and medium volume commercial drivers per mile; parking allowed; and number of 4-way intersections per mile.

Head (1959) developed several linear equations for arterials, grouped by ADT ranges and number of lanes (two or four). The following variables are employed in the equations: ADT (the average daily traffic divided by 100); the number of commercial units per mile; the number of intersections per mile; the number of traffic signals per mile; the indicated speed; and the pavement width in feet. In each equation, the number of commercial units per mile was significantly related to the...
accident rate. Also, commercial units were found to be the most important predictor of accidents.

C. There is no significant difference in accident rates between roads having no access points and those having access points serving only noncommercial purposes.

Stover et al (1970) used Kipp’s original research (1952) to investigate the influence of medial and marginal accesses on arterial operations. They found that little-used access points act similarly to no access points at all.

D. As the number of intersections per mile increases, the total accident and injury accident rates increase.

In Cribbins and Aray et al’s report (1967), all intersection openings per mile were tested, only signalized intersection openings were significant and used in the final regression equation.

In Cribbins and Horn et al’s report (1967), intersections were grouped as: (a) signalized with left-turn storage; (b) signalized without left-turn storage; (c) all intersections with left-turn storage; and (d) all intersections without left-turn storage, each on a per mile basis. All were found to be significantly related to the accident rate.
1.2.1.5 Residential Collector Roads

Daff and White (1990) revealed that more than half of all the accidents involved a collector road, and a high proportion of mid-blocks (at least 46 percent) involved collisions with parked cars or driveway maneuvers, and would probably not have occurred if parking and residential driveways were not present.

1.2.2 Access and Safety at the Microscope Level

At this level, geometric design of access facilities is considered. The access facilities are driveways, ramps, speed-change lanes, intersections, and medians.

1.2.2.1 Driveways

It was found that 11 percent of the city's total accidents were accounted for by accidents at driveways. Of these accidents, approximately two thirds involved left turns into or out of driveways (Box, 1969).

A number of studies of driveway accidents have been made, in an attempt to relate accidents to driveway characteristics. As Stover et al (1970) indicated, the conclusions of such studies have rather general terms. For example, roadways along which left turns into or out of the driveways are permitted have had higher accident rates than those with median barriers which restrict access to one direction of the through
lanes. Box (1969) reported a positive correlation between uncontrolled driveway width and accident rates. These results, however, were not adequate to establish a precise, quantitative relationship between driveway width and accident rate. Results of other studies have been affected by high degrees of variability and other biases resulting from lack of information on various types of driveways. In another paper, Box (1970) indicated that traditional "abbreviated data-processing tabulation systems provide too coarse a summation to completely establish driveway influences on accident".

On the other hand, Stover et al (1970) argued that it is questionable whether driveway accidents actually constitute the most appropriate criteria for driveway design. He believed that, at least for major roadways, the efficient movement of traffic is a primary consideration. This is complemented by findings relative to traffic safety. It is because: (1) "conflicts from poor or inadequate control of access may have reduced the efficiency long before it is reflected in statistically significant accident rates"; (2) "potential conflicts which account for much of accident hazard are also those conflicts to which other inefficiencies of the traffic stream can be attributed, so that, if these inefficiencies are corrected, many of the accident hazards will be also."

1.2.2.2 Ramps

Some studies were made to determine which geometric features play important roles in ramp safety. The measure of safety used in ramp studies is accident rates per million ramp vehicles, in which the number
of vehicles using the ramp is counted, while distance traveled along the ramp is not considered.

Lundy (1967) investigated 722 freeway ramps, and he found that the accident rates of on-ramps were consistently lower than off-ramp accident rates. Diamond ramps have the lowest accident rates, and the scissors\(^1\) and left side ramps have the highest rates. On scissors ramps, the primary concentration of accidents is at the scissors or cross-over (ramp with local road) facility. Straight ramps (on and off) have 12 percent lower overall accident rate than curved ramps. Yates (1970) also found that urban loop ramps with higher curvature have accident rates higher than those with low curvature. But rural loop ramps with low curvature have accident rate higher than those with high curvature.

For on-ramps, it is shown that 52 percent of accidents occurred in the merge area, and 48 percent in the ramp area (Lundy, 1967). Concerning the effectiveness of entrance ramp controls, Blumentritt (1981) indicated that using responsive ramp controls would reduce accidents by 10 to 33 percent.

A technical synthesis, NCHRP Report 35 (1976), identified the more successful design and operating practices used at freeway off-ramp terminals. The safety problem was addressed. The main points are as follows:

\(^1\) Lundy defined in his study (1967): "A scissors ramp is one that has opposing traffic crossing the ramp traffic. The ramp traffic has the right of way and a stop sign is placed to stop the crossing vehicle."
Diamond ramps have the lowest accident rate, scissors and left-handed exit ramps have the highest rates.

44 percent of the accidents occurred in the diverge area, 56 percent in the ramp area (Lundy, 1967).

The extent to which geometric and traffic characteristics (19 elements) are judged to have contributed to accidents was examined. As expected, an increase in traffic volume results in an increase in accidents, geometric alone apparently accounts for only a small portion of the variance in accidents.

Wrong-way entry is another type of incident occurring at off-ramps. Five percent of all fatalities on interstate highway are attributed to accidents resulting from wrong-way movements. The wrong-way accidents usually occur at night, and 75 percent of the wrong-way drivers who cause accidents have been drinking excessively (NCHRP Report 35, 1976).

1.2.2.3 Speed-Change Lanes

Cirillo (1970) conducted research to analyze speed-change lane length. Results indicated that increasing the length of acceleration lanes will reduce accident rates if the percentage of merging vehicles is greater than six percent of mainline volume. Increased length of deceleration lanes will also reduce accident safety but to a lesser degree. As the percentage of traffic entering or leaving the mainline through a speed-change lane increases, the accident increases. Lundy
(1967) concluded those acceleration lane lengths greater than 800 feet can be expected to have below average accident rates, deceleration lane lengths with 900 feet or longer have lower accident rates. The shorter radius, large central angle curved off-ramps seem to have lower rates than the ramps with median range radii and deltas. Cirillo et al (1969) found that a 100-ft increase (up to 2,600 ft) in the stopping sight distance decreased annual accidents per 1,000 vehicles per day (vpd) an amount of 0.0008 (accidents/year/vpd) for a deceleration lane.

1.2.2.4 Intersections

AASHTO (1990) recognized that intersections are access points. Some earlier studies, e.g., Peterson and Michael (1965), indicated that intersection accidents increased when:

- Percent green time on the bypass decreased.
- Bypass or cross-street ADT increased.
- Percent left turn from the bypass increased.
- Maximum approach speed increased.
- Number of intersection approaches increased.
- Total width of driveways within 200 ft of the intersection increased.

Some studies have addressed the role of intersection sight distance in producing accidents, and revealed the relationship between accidents and intersection sight distance. Wu (1973) studied the relationship between accident rate and what he called "clear vision
right-of-way" at 192 signalized intersections. He concludes that intersections where vision is poor have significantly higher accident rates, but no specific numbers are given and no statistical tests are cited. David and Norman (1975) studied the relationship between accident rate and various intersection geometric and traffic features. The study revealed significant accident rate differences between "obstructed" and "clear" intersections. However, the results are reported without regard for number of legs, number of lanes, type of control, presence of turning lanes and speed limit.

1.2.2.5 Medians and Left-Turn Lanes

Highway Users Federation for Safety and Mobility (1970) indicated that main-road left-turn storage lanes can significantly reduce accident rates. Some studies also found that the presence of left-turn storage lanes at median openings reduces the number of rear-end collisions on urban arterial (Cribbins et al, 1967; Sawhill and Neuzil, 1963). Several studies also concluded that at intersections the presence of left-turn lanes reduces the accidents (Shaw and Michael, 1963; Thomas, 1966; Tamburri and Hammer, 1968; and Hoffman, 1974).

Cribbins (1967) conducted a study in North Carolina, and he found that as traffic volumes increase, use of median openings rapidly becomes hazardous. When combined with intensive roadside development, use of median openings under high-volume conditions becomes more hazardous.
In short, though there are quite a few studies on access and safety, the majority of them were conducted decades ago. The relationship needs to be re-examined, considering changed technical, social and economic circumstances.

1.3 Access and Traffic Operations

Although some efforts have been made to deal with the effects of access on traffic operations, they are either not intended to establish quantitative relationships, or intended unsuccessfully to establish the relationships. One exception is that the newest revision of Chapter 7 of the Highway Capacity Manual (HCM), 1992, which does establish this relationship on multilane highways.

Chapter 7 of the Highway Capacity Manual (Revised Chapter 7 of HCM, 1992) has considered the effect of access points on multilane rural and suburban highways in its procedure of analyzing highway level of service. The research conducted by Reilly et al (1989) for the Revised Chapter 7 of HCM found that the number of access points has an important influence on free-flow speed, in that for every 10 access points per mile, travel speed will be reduced by 2.5 mph. The access point is a composite variable made up of the separate influences of different characteristics of access. However, Chapter 8 of the HCM, Two-Lane Highways, has not yet been updated to include the effect of access points on traffic operations.
One study conducted in the UK (Brocklebank, 1992) on two-lane rural highways found that access variables are generally difficult to fit in models of traffic operations. The composite variable (intersections, lay-bys, and accesses) was not significant in the UK study. Non-residential accesses and lay-bys separate access variables were either not significant or occasionally significant. The intersection variable was rather unstable with a coefficient value -1.8 (km/h). One possible explanation for the result is that there are so many other independent variables considered in the model (a total of 31 independent variables), the effect of access variables may be subsumed. In any case the results do not fully conform to our a priori expectation, and a further study is perceived to be needed. In addition, two-lane highways in urban and suburban areas should also be considered because the higher traffic and more roadside developments will intensify the influence of access points.

Glennon et al (1975) have developed techniques to counteract vehicle delay due to access points, and although it was recognized that the mainstream through traffic delay is caused by turning vehicles, no quantitative relationship has been established.

1.4 Optimal Number of Access Points

Most previous studies concentrated on minimum distance between access points. They are highway design oriented. For example, OECD (1971) recommended 250 metres to be the minimum spacing between intersections on arterial streets. NAASRA (1972) recommended
intersections at a minimum of 350 to 550 metres. Another earlier
guideline (Swedish National Board of Urban Planning, 1968) specified
minimum intersection spacing from 300 to 600 metres, depending on road
status. Similar minimum access spacing criteria for driveways were also
recommended by Glennon (1975). In general, the above studies are based
on stop sight requirements. On the other hand, Stover et al (1970)
raised intersection spacing issue which was mainly concerned with the
economic and legal implications of spacing. Two other studies utilized
quantitative methods to derive the spacing problem. They are Del Mistro
(1980) and Del Mistro and Fieldwick (1981). Both of them related
accident estimates to access spacing by statistical analysis. It was
found that intersection should be spaced not less than 250 m apart (Del
Mistro and Fieldwick, 1981). Del Mistro (1980) recommended the number of
intersections which are permissible in terms of the population of the
residential development and through traffic.

Basically, only safety was taken account in those studies. The
impact of access on traffic operation was not considered. Besides, most
of these studies concentrated on minimum access spacing rather than
optimal access spacing. This perhaps was because they were geometric
design oriented.

1.5 Scope of the Study

A complete highway system consists of three components: highway
facility; vehicle; and driver. An access point is a part of the highway
facility. To study the performance of a highway facility, it is
desirable to also take the vehicle and driver into account. This is because different types of vehicles combined with various driver characteristics will result in different levels of performance for the same highway facility. However, complete study of all of these aspects would require enormous resources, which is beyond the scope of this work. Thus the present study follows the general engineering practice of "freezing" some factors while focusing only on the major factor of concern, and making appropriate assumptions on those "frozen" factors. We assume passenger cars as the general vehicle type (passenger vehicle) and let an ordinary driver represent general driver characteristics. Therefore, only the highway facility (access points) will be specifically examined here. Besides, we will primarily concentrate on the macroscope level of access provisions, and study the effect of frequency of access points on traffic safety and operations. Geometric design of the various types of access was not considered in this study.

More specifically, with the above caveat, this present study differs from previous studies in the following areas: (1) all access types are included, from signalized intersection to agricultural access, and roadside pullout. (2) The effects of individual access types on safety and traffic operations are investigated, instead of using an aggregated access points measure, such as general access points. (3) The joint effects of access and other road characteristics are analyzed. (4) Several accident measures are used to develop statistical models to find the best statistical representation of accidents in the relationship. (5) Local data are used for analysis to specifically reveal the relationship between access and accidents on one Provincial highway.
network. (6) Apply the quantitative models of access-accidents and access-operation into access planning and access design procedure.

Figure 1.3 shows the basic structure of the thesis. In Chapter 2, the data to be used in the analyses is described specifically. Chapter 3 estimates the relationships between accesses and accidents for different highway classes. Furthermore, a conceptual hazard model is suggested which includes traffic conflicts information in safety analysis. Chapter 4 estimates the relationships between accesses and traffic operations (i.e., travel speed, for two-lane highways). A travel speed model is constructed. In chapter 5, an optimization model is formulated to obtain the optimal number of access points for highway sections based on inputs from previous chapters. The objective of the model is to minimize the total social cost. A "piecewise" linear approximation technique is employed to solve the integer nonlinear programming problem in the model. In chapter 6, access and speed zone design, particularly speed transition zone design, is discussed.

Figure 1.3 also shows that the thesis contains three levels of study: operation (chapters 3 and 4), planning (chapter 5), and design (chapter 6). Finally, Chapter 7 summarizes the results and possible future research.
Chapter 2 Data Base

Chapter 3 Accident Analysis

Chapter 4 Traffic Operation Model

Chapter 5 Optimal Access Points

Chapter 6 Speed Reduction Design

Legend:

- Actual Study Block

- Relevant Studies Block

- Type of the Study

Figure 1.3 Structure of the Study
Chapter Two
THE DATA

The data used in the study are extracted from the existing photolog of selected sections of highways in British Columbia. For the purpose of collecting accident, traffic and access data, a three year period (1988-1990) was used. The data base consists of four major parts: access, traffic, road characteristics, and accidents. Since each single highway section will be used as a data point, we start with the definition of highway section.

2.1 Highway Sections

About twenty numbered highway routes in the province of British Columbia were selected for study. The total tested road length was approximately 750 km, and divided into 376 sections. The following criteria were adopted in selecting highway routes and defining sections: (1) geographic representation of the provincial highways; (2) homogeneity of roadway characteristics within a section; (3) consistent road condition during the study period; (4) a wide range of traffic volume representation (average annual daily traffic, AADT); (5) no construction work in process in the section.

The section length ranged from 0.3 km to 13.8 km, with an average value of 2.23 km per section. Road sections were grouped on the basis of road classification, i.e., four-lane rural road, four-lane suburban road, two-lane rural road, and two-lane suburban road. Figure 2.1 shows
the sample size representation in each category. Basically, the sample size is greater than five percent of the population in each road category. Number of sections and the average length of the sections in each category are also given in the figure.

The section boundaries were selected at locations where no or very few accidents were recorded during the study period with the aid of an accident histogram (Figure 2.2). The section boundaries were defined in this manner to ensure that hazardous locations (i.e., black spots) were fully represented in only one section. Other considerations in defining section boundaries were the consistency of traffic control (e.g., speed limit), and road characteristics (e.g., with or without auxiliary lane).

Traffic volume is another consideration in section definition. An effort was made to include a wide range of traffic volume in road sections. For example, Table 2.1 shows the sample size distribution by traffic volume range and its relation to the provincial road inventory for two-lane rural highways. It can be seen that the sample used in this study represents six percent of the total two-lane rural highway system in British Columbia, and that the distribution of the observations reflects reasonably well the distribution of traffic volume classes on the road system as defined by the AADT.
Total Sample Size
Tested road length = 750 km
Percentage of total = 7%
No. of sections = 376
Average section length = 2.23 km

General 4-lane highways
Tested road length = 101 km
Percentage of total = 18%
No. of sections = 82
Average section length = 1.2 km

General 2-lane highways
Tested road length = 659 km
Percentage of total = 7%
No. of sections = 295
Average section length = 2.2 km

4-lane rural highways
Tested road length = 57 km
Percentage of total = 50%
No. of sections = 36
Average section length = 1.6 km

4-lane suburban highways
Tested road length = 45 km
Percentage of total = 10%
No. of sections = 46
Average section length = 1 km

2-lane rural highways
Tested road length = 560 km
Percentage of total = 6%
No. of sections = 164
Average section length = 3.4 km

2-lane suburban highways
Tested road length = 99 km
Percentage of total = 20%
No. of sections = 131
Average section length = 0.8 km

2-lane suburban highways (AADT <= 10,000)
Tested road length = 29 km
Percentage of total = -
No. of sections = 34
Average section length = 0.9 km

2-lane suburban highways (AADT > 10,000)
Tested road length = 70 km
Percentage of total = -
No. of sections = 97
Average section length = 0.7 km

Figure 2.1 Sample Size Representation
Highway 97, Segment 1110, 9 km - 17 km, Speed Limit = 80 km/h, 2-lane

<table>
<thead>
<tr>
<th>Landmark</th>
<th>Kilometer</th>
<th>Accident Frequency</th>
<th>Section No.</th>
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</thead>
<tbody>
<tr>
<td>Crossing Road 20</td>
<td>9.0</td>
<td>P</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Road 19</td>
<td>10.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>10.5</td>
<td>P</td>
<td></td>
</tr>
<tr>
<td>Crossing Road 18</td>
<td>11.0</td>
<td>P</td>
<td></td>
</tr>
<tr>
<td>Road 17</td>
<td>11.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Road 15</td>
<td>12.0</td>
<td>PPP</td>
<td>Section 1</td>
</tr>
<tr>
<td>Road 14</td>
<td>12.5</td>
<td>IP</td>
<td></td>
</tr>
<tr>
<td>Road 13</td>
<td>13.0</td>
<td>I P</td>
<td>Section 2</td>
</tr>
<tr>
<td>Road 12</td>
<td>13.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crossing Road 11</td>
<td>14.0</td>
<td>PPP</td>
<td>Section 3</td>
</tr>
<tr>
<td>Road 10</td>
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<td></td>
<td></td>
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<tr>
<td>Road 9</td>
<td>15.0</td>
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<td>Road 8</td>
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<td></td>
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<td>Road 7</td>
<td>16.0</td>
<td>P</td>
<td>Section 5</td>
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<td></td>
</tr>
<tr>
<td>Road 5</td>
<td>17.0</td>
<td></td>
<td>Section 6</td>
</tr>
</tbody>
</table>

Legend
- I = Injury
- P = Property Damage Only

Public Road
Business Access
Private Access

Figure 2.2 Sample of Section Definition
<table>
<thead>
<tr>
<th>AADT</th>
<th>Inventory (km)</th>
<th>Sample (km)</th>
<th>% of Inventory</th>
<th>No. of Obs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 5000</td>
<td>7625.1</td>
<td>520.9</td>
<td>6.8%</td>
<td>133</td>
</tr>
<tr>
<td>% of Total</td>
<td>83.1%</td>
<td>93.0%</td>
<td>81.1%</td>
<td></td>
</tr>
<tr>
<td>5001 - 10000</td>
<td>1261.5</td>
<td>28.4</td>
<td>2.3%</td>
<td>22</td>
</tr>
<tr>
<td>% of Total</td>
<td>13.8%</td>
<td>5.1%</td>
<td>13.4%</td>
<td></td>
</tr>
<tr>
<td>10001 and</td>
<td>284.1</td>
<td>10.7</td>
<td>3.8%</td>
<td>9</td>
</tr>
<tr>
<td>over</td>
<td>3.1%</td>
<td>1.9%</td>
<td>5.5%</td>
<td></td>
</tr>
<tr>
<td>% of Total</td>
<td>Total</td>
<td>9167.1</td>
<td>6.1%</td>
<td>164</td>
</tr>
</tbody>
</table>

Table 2.1 Distribution of Sample Size by AADT
(Two-lane rural highway)

2.2 Access Data

Ten types of road access were identified for general road classification. They are: (1) signalized intersection; (2) four-way unsignalized intersection; (3) three-way unsignalized intersection; (4) commercial access; (5) industrial access; (6) residential access; (7) agricultural access; (8) roadside pullout; (9) on-ramp and (10) off-ramp. After preliminary model testing, it was later determined to group some types according to functional characteristics of access to simplify the complexity of the model. At the end, six access types were used in the analysis: (1) signalized intersection; (2) unsignalized intersection (including four-way and three-way intersections); (3) business access (including commercial and industrial); (4) private access (including residential and agricultural); (5) roadside pullout; and (6) ramp. Note that the word "intersection" in this study is defined as the general area where two or more public roads join or cross. This definition will
distinguish the intersection from other access types, such as private access, where one private driveway is connected to one public road.

The coding of access data was done with photologging technology. The photologging involves taking a series of photographs of a roadway section at 20 metres' intervals from a specially equipped vehicle. Besides the camera system the vehicle is also equipped with special instruments to record road characteristics, such as grade and horizontal curvature. The type and location of accesses were recorded by viewing the study route. The access location reference was recorded using the vehicle's odometer reading, which was later adjusted to match the accident location reference. Access points were summarized for each road section by type, and then the density (number of access per km) of each access type was calculated for statistical modeling purposes.

2.3 Traffic Volume and Speed

Two main traffic variables were observed, speed limit and average annual daily traffic (AADT). Speed limit was obtained from the photolog, and AADT was obtained from published count station reports. It should be noted that the accessing traffic volumes of each access point were not available, thus only major road traffic was included. However, it is believed that the grouping of accesses by functional characteristics as outlined above provides a proxy for access volumes. For example, access volume is highest for group one access (signalized intersections) and least for group five (roadside pullouts).
Average travel speed of traffic is the measurement that will be used in Chapter 4 to study the relationship between access and traffic operations. However, it is not directly given in the photolog database. Travel speed of the test vehicle is employed as the surrogate of the average travel speed of main road traffic. The detail discussion of this representation will be presented in Chapter 4.

2.4 Road Characteristics

Some road characteristics' information was also available from the photolog, such as median type, grade, traverse slope, horizontal curvature, and the direction of curvature. The latter measure gives a record if the curvature is left or right. On the basis of this measurement, the frequency of change in direction per kilometre within the study segment was calculated. The intention of recording this information is to reflect the amount of driver's attention needed to traverse a given segment. Horizontal curvature is defined as the central angle of a 100 metre arc. This measurement, adapted from the Imperial definition of curve, is used rather than radius because the former is the measure recorded in the photolog database of British Columbia used for this research. The other road characteristic considered to be important is the presence of auxiliary lanes in the section. It was defined as a dummy variable in the data base with 1 indicating an auxiliary lane, 0 otherwise. Some studies have found significant correlation between accidents and lane and shoulder widths (e.g., Craus, Livneh and Ishai, 1991). However, because the lane and shoulder widths are largely fixed throughout the study area (lane width = 3.6m, and
paved shoulder width = 2.0m), these variables were not included in the analysis.

2.5 Accident Data

Accident records are available for all selected routes. After the determination of section boundaries, section length was available to calculate accident measures for each section. Five measures of accident were generated to find the best representation of accidents in the analysis. These measures are accident rate (ACDR, accidents/million-vehicle-kilometre), accident frequency (ACDD, accidents/km), accident severity ratio (ASR, a weighted severity ratio with weight factor 100 assigned to fatal accident, 10 assigned to injury accident, and 1 assigned to property-damage-only accident), severe accident rate (SEVR, fatal plus injury accidents/million-vehicle-kilometre), and severe accident frequency (SEVF, fatal plus injury accidents/km). As indicated earlier, three years (1988-1990) accident records are coded into the data base.
Chapter Three
THE EFFECT OF ACCESS ON HIGHWAY SAFETY

The first section of this chapter examines the established relationship between access and accidents. Sections 2 and 3 construct accident models using multiple regression analysis for two-lane rural, two-lane suburban, four-lane rural, and four-lane suburban highways. In section four, we will discuss the hypothesis of a hazard model.

3.1 Examination of Established Relationship

To see if the established relationships between accidents and access are still valid in the current situation, a preliminary test was conducted for two-lane rural highways. A similar graph as Figure 1.2 was plotted using our initial data (shown in Figure 3.1). To do this, similar access types (minor intersections, business access, and private access) were selected and aggregated as access points per mile (same definition as in Figure 1.2: number of minor intersections and principal access driveways per mile). However, it seems that the linear correlation between accident rate and access is not significant in our situation, which indicates that many other contributing factors other than access points should also be considered. Thus, it is presumed that this circumstance is applicable to other highway classes as well. Therefore, a complete reexamination of the relationships between access and accidents is needed.
Figure 3.1 Access Density vs. Accident Rate
(2-lane Rural Highways)
3.2 Model Construction

First of all, independent variables that will be used in the model construction are summarized in Table 3.1. These variables include three types: access, traffic, and road characteristics. Dependent variables are those accident measurement variables described in section 2.5.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$X_1$</td>
<td>Density of signalized intersection (No./km)</td>
</tr>
<tr>
<td>$X_2$</td>
<td>Density of unsignalized intersection (No./km)</td>
</tr>
<tr>
<td>$X_3$</td>
<td>Density of business access (No./km)</td>
</tr>
<tr>
<td>$X_4$</td>
<td>Density of private access (No./km)</td>
</tr>
<tr>
<td>$X_5$</td>
<td>Density of roadside pullout (No./km)</td>
</tr>
<tr>
<td>$X_6$</td>
<td>Density of on/off ramp (No./km)</td>
</tr>
<tr>
<td>$X_7$</td>
<td>Median type (solid double line=1, painted median=2, barrier=3, wide grass median=4)</td>
</tr>
<tr>
<td>$X_8$</td>
<td>Speed limit (SPL, km/h)</td>
</tr>
<tr>
<td>$X_9$</td>
<td>Average Annual Daily Traffic (AADT)</td>
</tr>
<tr>
<td>$X_{10}$</td>
<td>Grade (percent)</td>
</tr>
<tr>
<td>$X_{11}$</td>
<td>Traverse slope (percent)</td>
</tr>
<tr>
<td>$X_{12}$</td>
<td>Frequency of changing direction of curvature (No. of changes/km)</td>
</tr>
<tr>
<td>$X_{13}$</td>
<td>Horizontal curvature (degrees)</td>
</tr>
<tr>
<td>$X_{14}$</td>
<td>Dummy variable (with auxiliary lane = 1, without auxiliary lane = 0)</td>
</tr>
<tr>
<td>$X_{15}$</td>
<td>Section length (km)</td>
</tr>
</tbody>
</table>

Table 3.1 Independent Variables Used in Model Construction

A multiple linear regression technique was employed to develop models describing the combined effect of access, traffic and road characteristics on highway safety. To find the best model, a stepwise
procedure (backward elimination) was implemented. The criteria of choosing the best model were $R^2$, standard error (SE), mean absolute error (MAE), F-test, and t-Test.

The calibration procedure started with the simplest model form, i.e., all variables were entered into the model in their original forms. The advantage of estimating this model is that the direct linear relationships between the dependent and independent variables can be revealed, and the major contributory factors can be detected. However, since the actual relationship is very complex in most situations, a linear model can only explain a small portion of the variation in the dependent variable. Therefore, a polynomial model was produced to increase the explanatory power of the model and to allow for interaction of access and road characteristics' variables. As is the case with many polynomial ordinary least squares (OLS) models, heteroscedasticity can pose a significant statistical problem. In this study, a combination of variable transformation and weighting was used to normalize estimation bias. It is noted (Carroll and Ruppert, 1988) that transformations and weighting could be combined together when the variance depends on a covariate, or when it is unclear whether a transformation or weighting is preferable. Both conditions were present in the development of the models. As a result, a weighted least squares (WLS) method incorporating with transformation (in the form of square root) was applied to estimate the models whenever necessary.

The results of regression models for accident rate (ACDR), severe accident rate (SEVR), accident severity ratio (ASR), accident frequency
(ACDD), and severe accident frequency (SEVF) are summarized in Table 3.2. The table shows the results of the stepwise regression estimation. All variables are significant at the 0.05 level (t-Test)\(^1\). The F-test for each model is also satisfactory at 0.01 level of significant, indicating that all independent variables' coefficients are collectively significantly different from zero. The coefficients of variation \((R^2)\) of the models appear satisfactory for most models, that is greater than 50 percent. It should be noted that the \(R^2\) resulting from a stepwise regression estimation is always smaller than that resulting from a standard regression estimation, due to the exclusion of some independent variables based on the elimination criteria (in this case, the 0.05 level of significance). Therefore, compared with the explanatory power of previous models in this field, our results seem to be a marked improvement over previous models. Preliminary review of the results indicated that the ASR model provided poor correlation between independent variables and the dependent variable (ASR), probably due to the arbitrary weighting factors assigned to severe accidents.

Some coefficients may appear to have the wrong sign in the equations; for example, the SEVF model in A1N4 group (general 4-lane road) has items \(+1.7224X_i-0.3834X_i^2\). The result indicates that if there are five or more signalized intersections per km \((X_i)\), there will be a negative impact on the dependent variable. The reasons for this are: first, the regression model is limited by the range of data, and it is difficult to extend the result beyond the data range that basically

\(^1\) Individual t values are not reported upon here. The statistics software ensures all variables meeting t test, \(\alpha = 0.05\)
includes road sections with no more than five signalized intersections per km. Since our general 4-lane road includes only rural and suburban road sections, it is unusual to have signalized intersection density greater than five per kilometre. Second, it may also be true, logically, that more signalized intersections will reduce accidents and severity of accidents because signalized intersections are safer than unsignalized intersection with the same traffic volumes. Higher signalized intersection density will slow down average travel speed which has the effect of reducing accident severity.

To show the analysis procedure the models for two-lane rural highways (identified as A2 group in Table 3.2) were selected for discussion and are presented below.

Figure 3.2 shows comparisons of observed and predicted values by each model, except ASR model, in A2 group. The graphs present schematic illustrations of the amount of variation in the observed data, along with the best fit lines representing the equations for two-lane rural highways. It is noted that the accident frequency model (ACDD and SEVF) explains more variation than the accident rate model (ACDR and SEVR); and the all-accidents models (ACDD and ACDR) can explain more variation than the severe-accident-only models (SEVR and SEVF). The possible explanations are as follows: in the first case, accident rates were calculated based on traffic volume (AADT). However, the access volume was not available as indicated earlier. Thus AADT only represents traffic volumes on the main road. The incomplete information about traffic volumes may result in an imperfect accident measure in accident
<table>
<thead>
<tr>
<th>Group</th>
<th>Regression Equations</th>
<th>R²</th>
<th>SE</th>
<th>MAE</th>
<th>F*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>( ACDR^{65} = 1.8540 + 0.2405X_1 + 0.01102X_2 + 0.0120X_4X_{13} - 0.000074X_9 )</td>
<td>0.68</td>
<td>0.30</td>
<td>0.22</td>
<td>19.8</td>
</tr>
<tr>
<td></td>
<td>( ACDD^{65} = 0.002X_1 + 0.000196X_2X_{13} + 1.46818 \times 10^{-8}X_4X_9 )</td>
<td>0.78</td>
<td>0.00</td>
<td>0.00</td>
<td>43.1</td>
</tr>
<tr>
<td></td>
<td>( ASR^{65} = 0.1040X_3 + 0.0263X_8 + 0.2471X_{10} + 0.0756X_{11} - 0.0669X_{12} )</td>
<td>0.98</td>
<td>0.24</td>
<td>0.17</td>
<td>448.2</td>
</tr>
<tr>
<td></td>
<td>( SEVR^{65} = 0.8216 + 0.2584X_1 + 0.2316X_2 - 0.0153X_4 - 0.009469X_3 + 0.00028e^{65} )</td>
<td>0.62</td>
<td>0.19</td>
<td>0.12</td>
<td>9.1</td>
</tr>
<tr>
<td></td>
<td>( SEVF^{65} = 5.1930 + 4.0964X_1 + 0.3541X_2 + 0.000355X_9 - 0.6021X_{12} )</td>
<td>0.47</td>
<td>3.99</td>
<td>3.14</td>
<td>8.6</td>
</tr>
<tr>
<td>A2</td>
<td>( ACDR^{65} = 0.7040 + 0.3220X_2 + 0.002466X_3X_5 + 0.0130X_4X_{13} + 0.0456X_5X_{13} - 0.005429X_9^{65} )</td>
<td>0.54</td>
<td>0.07</td>
<td>0.05</td>
<td>24.5</td>
</tr>
<tr>
<td></td>
<td>( ACDD^{65} = 0.0223 + 0.0087X_3^{65} + 0.00129X_4^{65}X_5 + 0.0015X_5^{65}X_{13} + 0.00091X_9^{65} )</td>
<td>0.82</td>
<td>0.02</td>
<td>0.01</td>
<td>124.5</td>
</tr>
<tr>
<td></td>
<td>( ASR^{65} = 1.4172 + 0.3366e^{X_1} - 0.6717X_{10}^{65} + 0.1070X_{12} + 1.6942X_{14} )</td>
<td>0.20</td>
<td>0.98</td>
<td>0.63</td>
<td>11.4</td>
</tr>
<tr>
<td></td>
<td>( SEVR^{65} = 0.2714 + 0.1391X_3^{65} + 0.0017X_4^{65}X_5 + 0.0094X_5^{65}X_{13} + 0.0303X_5^{65}X_{13} + 0.0629X_{10}^{65} )</td>
<td>0.45</td>
<td>0.05</td>
<td>0.03</td>
<td>27.9</td>
</tr>
<tr>
<td></td>
<td>( SEVF^{65} = -0.4390 + 0.2995X_3^{65} + 0.002124X_4^{65}X_5 + 0.0208X_5^{65}X_{13} + 0.0346X_5^{65}X_{13} + 0.01884X_9^{65} )</td>
<td>0.66</td>
<td>0.07</td>
<td>0.05</td>
<td>46.2</td>
</tr>
<tr>
<td></td>
<td>( + 0.0679X_{10} + 0.2111X_{14} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Variable Description:
- \( X_1 \) Density of signalized intersection (No./km)
- \( X_2 \) Density of unsignalized intersection (No./km)
- \( X_3 \) Density of business access (No./km)
- \( X_4 \) Density of private access (No./km)
- \( X_5 \) Density of roadside pullout (No./km)
- \( X_6 \) Density of on/off ramp (No./km)
- \( X_7 \) Median type (solid double line=1, painted median=2, barrier=3, wide grass median=4)
- \( X_8 \) Speed limit (SPL, km/h)
- \( X_9 \) Average Annual Daily Traffic (AADT)
- \( X_{10} \) Grade (percent)
- \( X_{11} \) Traverse slope (percent)
- \( X_{12} \) Frequency of changing direction of curvature (No. of changes/km)
- \( X_{13} \) Horizontal curvature (degrees)
- \( X_{14} \) Dummy variable (with auxiliary lane = 1, without auxiliary lane = 0)
- \( X_{15} \) Section length (km)

Note: A1 = 4-lane rural road; A2 = 2-lane rural road.

Table 3.2 Accident Estimation Models (I)
## Table 3.2 Accident Estimation Models (II)

<table>
<thead>
<tr>
<th>Group</th>
<th>Regression Equations</th>
<th>R²</th>
<th>SE</th>
<th>MAE</th>
<th>F*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A3a</td>
<td>( ACDR^{0.5} = 0.9423 + 0.6996X_1^{0.5} + 0.2982X_2^{0.5} + 0.0437X_3 - 0.0083X_4 + 0.0886X_5 + 0.0164X_6 )</td>
<td>0.61</td>
<td>0.48</td>
<td>0.35</td>
<td>35.4</td>
</tr>
<tr>
<td></td>
<td>( ACDD^{0.5} = -0.0162 + 0.1005X_1^{0.5} - 0.0409X_2 + 0.1023X_3^{0.5} + 0.000069e^{X_4} + 0.0066X_5 + 4.6099\times10^4X_6 ) + 0.0108X_7</td>
<td>0.67</td>
<td>0.06</td>
<td>0.04</td>
<td>39.5</td>
</tr>
<tr>
<td></td>
<td>( SEVR^{0.5} = 0.6060 + 0.3941X_4^{0.5} + 0.1858X_5^{0.5} + 0.0242X_6 - 0.0026X_7^{0.5} + 0.0529X_8 )</td>
<td>0.49</td>
<td>0.32</td>
<td>0.24</td>
<td>26.5</td>
</tr>
<tr>
<td></td>
<td>( SEVF^{0.5} = 0.3973 - 1.5020X_1 + 2.9955X_2 - 0.7707X_3 + 1.6373X_4^{0.5} + 0.000907e^{X_5} + 0.0036X_6 )</td>
<td>0.48</td>
<td>0.80</td>
<td>0.49</td>
<td>20.9</td>
</tr>
<tr>
<td>A31a</td>
<td>( ACDR^{0.5} = 0.4138 + 0.7510X_1^{0.5} + 0.3142X_2^{0.5} + 0.0737X_3 + 0.1047X_4 )</td>
<td>0.64</td>
<td>0.49</td>
<td>0.36</td>
<td>43.3</td>
</tr>
<tr>
<td></td>
<td>( ACDD^{0.5} = 0.0950X_1^{0.5} + 0.0398X_2^{0.5} + 0.0089X_3 + 3.9527\times10^5X_4 + 0.0027X_5 )</td>
<td>0.91</td>
<td>0.06</td>
<td>0.05</td>
<td>186.2</td>
</tr>
<tr>
<td></td>
<td>( SEVR^{0.5} = 0.2616 + 0.3669X_1^{0.5} + 0.1917X_2^{0.5} + 0.0436X_3 + 0.0117X_4 )</td>
<td>0.56</td>
<td>0.32</td>
<td>0.24</td>
<td>31.7</td>
</tr>
<tr>
<td></td>
<td>( SEVF^{0.5} = 1.1083 + 0.7581X_1^{0.5} + 0.2739X_2^{0.5} + 0.0784X_3 - 0.0492X_4 )</td>
<td>0.59</td>
<td>0.26</td>
<td>0.19</td>
<td>13.1</td>
</tr>
<tr>
<td>A32b</td>
<td>( ACDR^{0.5} = 0.9400 + 0.8622X_1 + 0.1345X_2 - 0.000073X_3 + 0.0378X_4 )</td>
<td>0.79</td>
<td>0.29</td>
<td>0.21</td>
<td>31.8</td>
</tr>
<tr>
<td></td>
<td>( ACDD^{0.5} = 0.0743 + 0.0768X_1 + 0.0082X_2 - 0.0008X_3 + 4.6226\times10^{-6}X_4 + 0.0022X_5 )</td>
<td>0.81</td>
<td>0.02</td>
<td>0.02</td>
<td>28.8</td>
</tr>
<tr>
<td></td>
<td>( SEVR^{0.5} = 0.5496 + 0.3529X_1 + 0.0809X_2 - 0.000056X_3 + 0.0301X_4 )</td>
<td>0.59</td>
<td>0.26</td>
<td>0.19</td>
<td>13.1</td>
</tr>
</tbody>
</table>

**Variable Description:**

- \( X_1 \): Density of signalized intersection (No./km)
- \( X_2 \): Density of unsignalized intersection (No./km)
- \( X_3 \): Density of business access (No./km)
- \( X_4 \): Density of private access (No./km)
- \( X_5 \): Density of roadside pullout (No./km)
- \( X_6 \): Density of on/off ramp (No./km)
- \( X_7 \): Median type (solid double line=1, painted median=2, barrier=3, wide grass median=4)
- \( X_8 \): Speed limit (SPL, km/h)
- \( X_9 \): Average Annual Daily Traffic (AADT)
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- \( X_{11} \): Traverse slope (percent)
- \( X_{12} \): Frequency of changing direction of curvature (No. of changes/km)
- \( X_{13} \): Horizontal curvature (degrees)
- \( X_{14} \): Dummy variable (with auxiliary lane = 1, without auxiliary lane = 0)
- \( X_{15} \): Section length (km)

**Note:**
- A3 = 2-lane suburban road for whole AADT range; A31 = 2-lane suburban road with AADT > 10000;
- A32 = 2-lane suburban road with AADT < 10000.
- a. No equation for ASR.
- b. No equations for ASR and SEVF.
<table>
<thead>
<tr>
<th>Group</th>
<th>Regression Equations</th>
<th>R²</th>
<th>SE</th>
<th>MAE</th>
<th>F*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4</td>
<td>[ACDR = 2.7606 + 0.4209X_1 + 0.2652X_2^{0.5} + 0.005439X_1^2 - 0.012176X_2^{0.9}]</td>
<td>0.78</td>
<td>0.43</td>
<td>0.30</td>
<td>27.1</td>
</tr>
<tr>
<td></td>
<td>[ACDD = 0.0182 + 0.0134X_1 + 0.0046X_2 + 0.000119X_2^{2}]</td>
<td>0.74</td>
<td>0.01</td>
<td>0.01</td>
<td>27.3</td>
</tr>
<tr>
<td></td>
<td>[ASR = 3.3342 + 1.1764X_1^{0.5} - 1.0696\times10^7X_2^{1.2}]</td>
<td>0.27</td>
<td>2.76</td>
<td>2.00</td>
<td>63.3</td>
</tr>
<tr>
<td></td>
<td>[SEVR = 1.1973 + 0.3010X_1 + 0.0610X_2 - 0.005533X_2^{0.5}]</td>
<td>0.65</td>
<td>0.22</td>
<td>0.15</td>
<td>18.7</td>
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<td>[SEVF = 2.2457 + 3.1693X_1 + 0.7038X_2^{0.5}]</td>
<td>0.62</td>
<td>2.13</td>
<td>1.54</td>
<td>24.3</td>
</tr>
<tr>
<td>A2N3</td>
<td>[ACDR = 0.7241 + 0.3327X_1 + 0.1007X_2 + 0.0632X_3 - 0.01\times10^9X_4 + 0.1120X_5 - 0.0672X_6 + 0.02X_7 + 0.0241X_8]</td>
<td>0.59</td>
<td>0.73</td>
<td>0.49</td>
<td>71.1</td>
</tr>
<tr>
<td></td>
<td>[ACDD = 0.0871X_1^{0.5} + 0.0304X_2^{0.5} + 0.0149X_3^{0.5} + 0.000526X_4^{0.5} + 0.0012X_5]</td>
<td>0.93</td>
<td>0.05</td>
<td>0.03</td>
<td>830.3</td>
</tr>
<tr>
<td></td>
<td>[ASR = 1.8294 + 0.142X_1^{0.5} - 0.0730X_6 + 0.3337X_1^{2.5} - 0.2348X_6 + 0.0509X_8 + 0.4027X_9]</td>
<td>0.20</td>
<td>0.10</td>
<td>0.06</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td>[SEVR = 0.5775 + 0.2816X_1^{0.5} + 0.1871X_2^{0.5} + 0.0299X_3 + 0.1755X_2^{1.5} - 0.002407X_4^{0.5} - 0.0538X_5]</td>
<td>0.45</td>
<td>0.35</td>
<td>0.24</td>
<td>35.5</td>
</tr>
<tr>
<td></td>
<td>[SEVF = 0.7930 + 0.750X_1^{0.5} + 0.4106X_2^{0.5} + 0.1503X_3^{0.5} + 0.3513X_4 + 0.04889X_5^{0.5} - 0.1851X_6]</td>
<td>0.59</td>
<td>0.73</td>
<td>0.49</td>
<td>71.1</td>
</tr>
<tr>
<td>AIN4</td>
<td>[ACDR = 0.2278 + 1.7871X_1 - 0.3768X_2 + 0.1330X_3 + 6.3964\times10^8X_4^{0.5} + 0.0353X_5 + 0.130X_6]</td>
<td>0.77</td>
<td>0.39</td>
<td>0.28</td>
<td>42.1</td>
</tr>
<tr>
<td></td>
<td>[ACDD = -0.0105 + 0.0195X_1 + 0.0011X_2 + 1.1343\times10^4X_9]</td>
<td>0.82</td>
<td>0.00</td>
<td>0.01</td>
<td>118.7</td>
</tr>
<tr>
<td></td>
<td>[ASR = -7.4867 + 4.1696X_1 - 1.6065X_2 + 0.0360X_3 + 0.5934X_4 + 0.001416X_5 + 1.0488X_6]</td>
<td>0.32</td>
<td>3.12</td>
<td>2.16</td>
<td>7.1</td>
</tr>
<tr>
<td></td>
<td>[SEVR = 0.5053 + 0.3516X_1 + 0.0130X_2 - 0.00015X_9]</td>
<td>0.52</td>
<td>0.27</td>
<td>0.20</td>
<td>28.2</td>
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<tr>
<td></td>
<td>[SEVF = -0.0431 + 1.7224X_1 - 0.3834X_2 + 0.3265X_3 + 0.005787X_2^{1.5} + 0.299976X_3^{0.5} - 0.3459X_7] + 0.060739X_8 - 0.0482X_9</td>
<td>0.61</td>
<td>0.66</td>
<td>0.50</td>
<td>15.6</td>
</tr>
</tbody>
</table>

**Variable Description:**

- \(X_1\): Density of signalized intersection (No./km)
- \(X_2\): Density of unsignalized intersection (No./km)
- \(X_3\): Density of business access (No./km)
- \(X_4\): Density of private access (No./km)
- \(X_5\): Density of roadside pullout (No./km)
- \(X_6\): Density of on/off ramp (No./km)
- \(X_7\): Median type (solid double line=1, painted median=2, barrier=3, wide grass median=4)
- \(X_8\): Speed limit (SPL, km/h)
- \(X_9\): Average Annual Daily Traffic (AADT)
- \(X_{10}\): Grade (percent)
- \(X_{11}\): Traverse slope (percent)
- \(X_{12}\): Frequency of changing direction of curvature (No. of changes/km)
- \(X_{13}\): Horizontal curvature (degrees)
- \(X_{14}\): Dummy variable (with auxiliary lane = 1, without auxiliary lane = 0)
- \(X_{15}\): Section length (km)

**Note:**

- A4 = 4-lane suburban road
- A2N3 = general 2-lane road (rural + suburban)
- A1N4 = general 4-lane road (rural + suburban)

**Table 3.2 Accident Estimation Models (III)**
Figure 3.2 Comparison of Predicted and Observed Values
Figure 3.2 (cont'd) Comparison of Predicted and Observed Values

- c: Accident Density Model (ACDD)
- d: Severe Accident Density Model (SEVF)
rate models. On the other hand, in accident frequency models, AADT was only used as an individual independent variable, the dependent variable (accident frequency) could be precisely measured, therefore, the resulting uncertainty is much lower than that in accident rate model.

In the second case, the reason that all-accident models are superior to severe-accident-only models is perhaps because more accident information was included in the former models. As we know, property-damage-only accidents are a major portion of all accidents, and severe accidents do not occur in every road section. Many zero values in the observations of the dependent variable (accident measure) will provide little unsafety information to model construction. Therefore, the explanation ability of such models will be affected.

The component effect of each independent variable can be represented by "component + residual" plots. Figure 3.3 shows sample plots for four access related variables, $X_2^{0.5}$ (unsignalized intersection density, Figure 3.3.a), $X_4^{0.5}X_{13}$ (private access density and horizontal curvature, Figure 3.3.b) in the accident rate (ACDR) model. The straight (component) lines in the graph are defined as: $\beta_j(X_j - \bar{X}_j)$, which multiplies the centered value of $X_j$ by the associated value of its regression coefficient $\beta_j$. The closeness of the scattered points (component + residual) around the component line for $X_2^{0.5}$ (Figure 3.3.a) indicates that unsignalized intersection density is significantly contributing in explaining the variation in accident rate. The component effect of the interaction variable of private access density
Figure 3.3 Component Plot for the Dependent Variable ACDR
and horizontal curvature, \(X_4^{0.5}X_{13}\), provides less significant contribution to the explanatory power of the model (Figure 3.3.b).

The correlation analyses showed no significant multicollinearity problem among the independent variables. Table 3.3 through 3.6 show correlation matrixes of ACDR, ACDD, SEVR, and SEVF models for two-lane rural highways. It is logical that some variables be closely correlated, such as speed limit \(X_8\) and horizontal curvature \(X_{13}\); but when these are combined with other variables such as \(X_{7}^{0.5}X_4\) and \(X_{7}^{0.5}X_{13}\), the results change. The correlation analysis uses the composite variables in the model instead of original individual variables \(X_8\) and \(X_{13}\). The tables indicate that the effect of each variable or composite variable on the accident measure can be satisfactorily explained, resulting in a conclusion that any problems of multicollinearity are not serious.

From regression coefficients for two-lane rural highways in Table 3.2, it can be seen that unsignalized intersection density has the largest effect on accident occurrence. This finding conforms to prior expectation as public roads generate high access traffic volume and result in more traffic conflicts than any other access types.

The composite variable of business access density (\(X_3\)) and speed limit (\(X_8\)) clearly shows that a change in accidents as a result of \(X_3\) change also depends on the speed limit value. That is, the higher the speed limit, the larger the impact of the business access density on accident occurrence.
<table>
<thead>
<tr>
<th></th>
<th>$X_2^{0.5}$</th>
<th>$X_3^{0.5} X_4$</th>
<th>$X_4^{0.5} X_{13}$</th>
<th>$X_5^{0.5} X_{13}$</th>
<th>$X_9^{0.5}$</th>
<th>$X_{10}$</th>
<th>$X_{11}$</th>
<th>$X_{12}^{0.5}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Const</td>
<td>1.0000</td>
<td>.0985</td>
<td>.0394</td>
<td>-.3216</td>
<td>-.4751</td>
<td>-.6113</td>
<td>.0987</td>
<td>-.8129</td>
</tr>
<tr>
<td>$X_2^{0.5}$</td>
<td>.0985</td>
<td>1.0000</td>
<td>-.2908</td>
<td>-.1501</td>
<td>-.3650</td>
<td>-.5609</td>
<td>-.1019</td>
<td>-.0624</td>
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<tr>
<td>$X_3^{0.5} X_8$</td>
<td>.0394</td>
<td>-.2908</td>
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<td>.0798</td>
<td>.0265</td>
<td>.0779</td>
<td>.0497</td>
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<td>$X_4^{0.5} X_{13}$</td>
<td>-.3216</td>
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<td>$X_5^{0.5} X_{13}$</td>
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<td>.4037</td>
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<td>$X_9^{0.5}$</td>
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<td>.0265</td>
<td>.1751</td>
<td>.4037</td>
<td>1.0000</td>
<td>.0267</td>
<td>.3073</td>
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<tr>
<td>$X_{10}$</td>
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<td>-.1019</td>
<td>.0079</td>
<td>.0340</td>
<td>-.3877</td>
<td>.0267</td>
<td>1.0000</td>
<td>-.2701</td>
</tr>
<tr>
<td>$X_{11}$</td>
<td>-.8129</td>
<td>-.0624</td>
<td>.0497</td>
<td>.3659</td>
<td>.2123</td>
<td>.3073</td>
<td>-.2701</td>
<td>1.0000</td>
</tr>
<tr>
<td>$X_{12}^{0.5}$</td>
<td>-.8219</td>
<td>.1603</td>
<td>-.2287</td>
<td>.0778</td>
<td>.4992</td>
<td>.2990</td>
<td>-.2031</td>
<td>.5501</td>
</tr>
</tbody>
</table>

Table 3.3 Correlation Matrix for ACDR$^{0.5}$ (2-lane rural highways)

<table>
<thead>
<tr>
<th></th>
<th>$X_2^{0.5}$</th>
<th>$X_3^{0.5} X_4$</th>
<th>$X_4^{0.5} X_{13}$</th>
<th>$X_5^{0.5} X_{13}$</th>
<th>$X_9^{0.5}$</th>
<th>$X_{11}$</th>
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</thead>
<tbody>
<tr>
<td>Const</td>
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<td>-.1424</td>
<td>-.0683</td>
<td>-.0186</td>
<td>-.7328</td>
</tr>
<tr>
<td>$X_2^{0.5}$</td>
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<td>1.0000</td>
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<td>.1160</td>
<td>-.6537</td>
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<tr>
<td>$X_3^{0.5} X_8$</td>
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<td>-.3110</td>
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<td>-.0659</td>
<td>.0508</td>
<td>-.0600</td>
</tr>
<tr>
<td>$X_4^{0.5} X_{13}$</td>
<td>-.0683</td>
<td>-.3809</td>
<td>-.0659</td>
<td>1.0000</td>
<td>-.1949</td>
<td>.2011</td>
</tr>
<tr>
<td>$X_5^{0.5} X_{13}$</td>
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<td>.1160</td>
<td>.0508</td>
<td>-.1949</td>
<td>1.0000</td>
<td>.0381</td>
</tr>
<tr>
<td>$X_9^{0.5}$</td>
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<td>-.6537</td>
<td>-.0600</td>
<td>.2011</td>
<td>.0381</td>
<td>1.0000</td>
</tr>
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<td>$X_{11}$</td>
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<td>.1417</td>
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<td>.5269</td>
</tr>
</tbody>
</table>

Table 3.4 Correlation Matrix for ACDd$^{0.5}$ (2-lane rural highways)
<table>
<thead>
<tr>
<th></th>
<th>Const.</th>
<th>$X_{15}^{0.5}$</th>
<th>$X_{1}^{0.5}X_{4}$</th>
<th>$X_{1}^{0.5}X_{13}$</th>
<th>$X_{5}^{0.5}X_{13}$</th>
<th>$X_{16}$</th>
</tr>
</thead>
<tbody>
<tr>
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<td>-.1984</td>
<td>-.3346</td>
<td>-.0519</td>
<td>-.5712</td>
</tr>
<tr>
<td>$X_{2}^{0.5}$</td>
<td>-.2552</td>
<td>1.0000</td>
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<td>-.0349</td>
</tr>
<tr>
<td>$X_{3}^{0.5}X_{8}$</td>
<td>-.1984</td>
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<td>.2481</td>
<td>.0701</td>
</tr>
<tr>
<td>$X_{4}^{0.5}X_{13}$</td>
<td>-.3346</td>
<td>.0121</td>
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<td>.1282</td>
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<tr>
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<td>-.4634</td>
<td>.2481</td>
<td>.0023</td>
<td>1.0000</td>
<td>-.4341</td>
</tr>
<tr>
<td>$X_{16}$</td>
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<td>-.0349</td>
<td>.0701</td>
<td>.1282</td>
<td>-.4341</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

Table 3.5 Correlation Matrix for SEVR$^{0.5}$ (2-lane rural highways)

<table>
<thead>
<tr>
<th></th>
<th>Const.</th>
<th>$X_{1}^{0.5}$</th>
<th>$X_{3}^{0.5}X_{4}$</th>
<th>$X_{4}^{0.5}X_{13}$</th>
<th>$X_{5}^{0.5}X_{13}$</th>
<th>$X_{9}^{0.5}$</th>
<th>$X_{10}$</th>
<th>$X_{14}$</th>
</tr>
</thead>
<tbody>
<tr>
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<td>-.1460</td>
<td>-.2215</td>
<td>-.3082</td>
<td>-.8171</td>
<td>-.4196</td>
<td>.0621</td>
</tr>
<tr>
<td>$X_{2}^{0.5}$</td>
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<td>1.0000</td>
<td>-.2478</td>
<td>-.0844</td>
<td>-.5070</td>
<td>-.5775</td>
<td>-.1116</td>
<td>-.0922</td>
</tr>
<tr>
<td>$X_{3}^{0.5}X_{8}$</td>
<td>-.1460</td>
<td>-.2478</td>
<td>1.0000</td>
<td>-.2696</td>
<td>.1903</td>
<td>-.0014</td>
<td>.0787</td>
<td>.2164</td>
</tr>
<tr>
<td>$X_{4}^{0.5}X_{13}$</td>
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<td>-.0844</td>
<td>-.2696</td>
<td>1.0000</td>
<td>-.0594</td>
<td>-.0225</td>
<td>.1335</td>
<td>.3788</td>
</tr>
<tr>
<td>$X_{5}^{0.5}X_{13}$</td>
<td>-.3082</td>
<td>-.5070</td>
<td>.1903</td>
<td>-.0594</td>
<td>1.0000</td>
<td>.3760</td>
<td>-.3546</td>
<td>-.2397</td>
</tr>
<tr>
<td>$X_{9}^{0.5}$</td>
<td>-.8171</td>
<td>-.5775</td>
<td>-.0014</td>
<td>-.0225</td>
<td>.3760</td>
<td>1.0000</td>
<td>.1238</td>
<td>-.2913</td>
</tr>
<tr>
<td>$X_{10}$</td>
<td>-.4196</td>
<td>-.1116</td>
<td>.0787</td>
<td>.1335</td>
<td>-.3546</td>
<td>.1238</td>
<td>1.0000</td>
<td>.0115</td>
</tr>
<tr>
<td>$X_{14}$</td>
<td>.0621</td>
<td>-.0922</td>
<td>.2164</td>
<td>.3788</td>
<td>-.2397</td>
<td>-.2913</td>
<td>.0115</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

Table 3.6 Correlation Matrix for SEVF$^{0.5}$ (2-lane rural highways)
Similar interaction effects can be found in the other two access variables, i.e., private access and roadside pullout, with the interaction of horizontal curvature. These relationships indicate that the effect of access density on accident increases with greater horizontal curvature, implying that a sight distance problem may exist. The significance of the horizontal curvature with the private access and roadside pullout along with its insignificance with the unsignalized intersection density can be used to speculate that the sight distance is incorporated in intersection geometric design, but not in private driveway and roadside pullout design.

Besides the access variables, it is also worth examining the effects of other traffic and road characteristic variables. It is interesting that the coefficient of AADT in the accident rate (ACDR) model is negative, whereas it is positive in the accident frequency (ACDD) and severe accident frequency (SEVF) models. Because accident rate is calculated by dividing the number of accidents by AADT, a negative AADT coefficient indicates the increase in accident rate is proportionally smaller than the increase in AADT. Furthermore, it is noted that the accident frequency models (ACDD, SEVF) depict a nonlinear relationship that the rate of accident increase decreases as traffic volume becomes larger. This finding is in accordance with the findings of other researchers such as Ng and Haure (1989).

Previous studies have shown that steep grades result in higher accident rates (e.g., Glennon, 1987, Kilhberg and Tharp, 1968). This relationship was confirmed in the present study as can be seen from the
significant and positive coefficient of $X_{10}$ (grade). It should be pointed out that downgrades and upgrades were not distinguished in this study since all accident measures were calculated on the basis of two-way traffic.

Another geometric feature of the roadway, traverse slope, $X_{11}$, was significant in the all-accident models (ACDR and ACDD), but not in the severe-accident models (SEVR and SEVF), which indicates that the traverse slope of the road has more effect on property-damage-only accidents. The negative sign of the coefficient shows a decreasing relationship, a result that was previously reported by Dart and Mann (1970), Carlsson and Hedman (1989). Roadways with relatively flat traverse slopes are more accident-prone than those with some slopes. It should be noted that the value of this variable in the present study ranged between 0.9 and 4.9 degree.

Finally, it is noted that the frequency of change in curvature direction, $X_{12}$, is positively correlated with accident rate, and the presence of auxiliary lanes $X_{14}$ is positively correlated with severe accident frequency. The effect of $X_{12}$ could not be found in previous studies, perhaps due to lack of information. However, the positive effect of this variable on the accident rate seems to be intuitively correct, since larger values of this variable reflect a more complex driving environment. The effect of auxiliary lanes found in this study is different from some previous studies. Kalakota et al (1993) found that there was no significant relationship between auxiliary lanes and accident rates, while Hedman (1989) reported a negative relationship
with the total accident rate. In our case, the positive correlation of the presence of auxiliary lanes with severe-accident frequency may be due to accidents resulting from high speed merge/diverge maneuver. Since the variable is not significant in the all-accident models, it implies that auxiliary lanes do not have a positive influence on property-damage-only accidents.

3.3 Results and Discussion

As a result of the above analyses, the relationships between accidents and each access type were normalized for all road categories. The normalization is based on the accident rate (ACDR) model because the accident rate is widely used in safety analysis. This was done by varying one of the interaction variables while holding all other independent variables at their mean values. For example, to study the impact of business access density on accident occurrence, various speed limit values (60 km/h, 70 km/h, etc.) are considered while all other independent variables are fixed at their mean values.

Figure 3.4 through Figure 3.19 show the estimated relationships for four road categories, i.e., four-lane rural, four-lane suburban, two-lane rural, and two-lane suburban. Again, two-lane rural highways were selected as an example of the description of the effects of access density on accidents in the following paragraphs.

The variation of the accident rate with changes in access density and horizontal curvature is depicted in Figure 3.4 (for unsignalized
intersection), Figure 3.5 (for private access), and Figure 3.6 (for roadside pullout). All figures show a diminishing increase trend in accident rate with the increase of access density. For example, Figure 3.4 shows a 68% increase in accident rate due to a change in unsignalized intersection density from zero to one per kilometre. This increase in accident rate is only 20% when intersection density increases from one to two intersections per kilometre; and it is 8% when intersection density increases from five to six intersections per kilometre. The implication of this finding is that the safety benefit of access control is not proportional to the percentage reduction in access density.

Another type of relationship between access density and accidents can be found in Figure 3.7, where the use of interaction variables is evident by the widening gaps between contour lines as the value of access density increases. The increase in accident rate due to an increase in access density is greater at higher speed limit than that at lower speed limit. In addition, the diminishing rate of increase in accident rate is still present in Figures 8 as was the case in Figures 3.4, 3.5, and 3.6.
Figure 3.4 Estimated Relationship between Accident Rate and Unsignalized Intersection Density by Horizontal Curvature
(2-lane Rural Highways)

Model: ACDR
All other variables are held at their mean values

Accident Rate (No./mi-veh-km)

Unsignalized Public Road Intersection Density (No./km)

Horizontal Curvature

7 degrees*
5 degrees
3 degrees

* See note on page 35.
Model: ACDR
All other variables are held at their mean values

Figure 3.5 Estimated Relationship between Accident Rate and Private Access Density by Horizontal Curvature
(2-lane Rural Highways)

* See note on page 35.
Figure 3.6 Estimated Relationship between Accident Rate and Roadside Pullout Density by Horizontal Curvature
(2-lane Rural Highways)

Model: ACDR
All other variables are held at their mean values

Horizontal Curvature

7 degrees*
5 degrees
3 degrees

* See note on page 35.
Figure 3.7 Estimated Relationship between Accident Rate and Business Access Density by Speed Limit
(2-lane Rural Highways)
Figure 3.8 Estimated Relationship between Accident Rate and Signalized Intersection Density by Speed Limit
(Two-lane Suburban Highways)
Figure 3.9 Estimated Relationship between Accident Rate and Signalized Intersection Density by Grade

(Two-lane Suburban Highways)
Figure 3.10 Estimated Relationship between Accident Rate and Unsignalized Intersection Density by Speed Limit
(Two-lane Suburban Highways)
Figure 3.11 Estimated Relationship between Accident Rate and Unsignalized Intersection Density by Grade

(Two-lane Suburban Highways)
Figure 3.12 Estimated Relationship between Accident Rate and Business Access Density by Speed Limit
(Two-lane Suburban Highways)
Figure 3.13 Estimated Relationship between Accident Rate and Business Access Density by Grade

(Two-lane Suburban Highways)
Figure 3.14 Estimated Relationship between Accident Rate and Signalized Intersection Density

(Four-lane Rural Highways)
Unsignalized Intersection Density (No./km)

Accident Rate (No./mil-veh-km)

Figure 3.15 Estimated Relationship between Accident Rate and Unsignalized Intersection Density

(Four-lane Rural Highways)
Figure 3.16 Estimated Relationship between Accident Rate and Private Driveway Density by Horizontal Curvature

(Four-lane Rural Highways)

* See note on page 35.
Figure 3.17 Estimated Relationship between Accident Rate and Signalized Intersection Density
(Four-lane Suburban Highways)
Figure 3.18 Estimated Relationship between Accident Rate and Unsignalized Intersection Density

(Four-lane Suburban Highways)
Figure 3.19 Estimated Relationship between Accident Rate and Business Access Density (Four-lane Suburban Highways)
Another way of representing the relationship between the density of various types of access and accident rate is using equivalency factors. It can be seen that the curves are rather flat after the initial sharp increase in accident rates, we then approximate curves to linear functions for each pair of access and accident relationship within a specified variable range, i.e., ignore the low left portion of the curves. This is a reasonable simplification because our major interests are not in very low access density situation. The linear slope was then calculated for each access type by taking an average horizontal curvature (degree of curve = 5) and a speed limit of 80 km/h. Finally, the equivalency factors are obtained by calculating the linear slope ratio between each access type and a selected base access type (unsignalized intersection). Note that the resulting equivalency factors are valid only over the specified access density range. For example, in the case of private access (Figure 3.7), the linear range is 2-16 access per kilometre. The calculation results for two-lane rural highways are shown in Table 3.7. It indicates that, in terms of impact on accident rate, one unsignalized access is equivalent to two business accesses, and 10 private accesses.

Similar results can also be found for other road categories as shown in Figure 3.8 through Figure 3.19. However, due to the apparent nonlinear characteristic in many relationships, the equivalency factors were not calculated for other road categories.
### Table 3.7 Access Type Equivalency Factors for Two-lane Rural Highways (80 km/h, average degree of curve = 5 degrees*)

<table>
<thead>
<tr>
<th>Access Type</th>
<th>Applicable Access Density Range</th>
<th>Change in ACDR per unit Change in Access Density</th>
<th>Equivalent Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intersection (Base Type)</td>
<td>1 - 6 /km</td>
<td>0.34</td>
<td>1</td>
</tr>
<tr>
<td>Business Access</td>
<td>1 - 8 /km</td>
<td>0.17</td>
<td>0.5</td>
</tr>
<tr>
<td>Private Access</td>
<td>2 - 16 /km</td>
<td>0.035</td>
<td>0.1</td>
</tr>
<tr>
<td>Roadside Pullout</td>
<td>0.5 - 1.5/km</td>
<td>0.37</td>
<td>1</td>
</tr>
</tbody>
</table>

* See note on page 35.

In conclusion, these figures could be used to determine the contributing effects of access on accident occurrence (accident rate). Given information about access, traffic volume, speed limit, and geometrics of a road section, the equations presented in Table 3.2 could be employed when the estimation of the safety (represented by different accident measures) for that road section is required.

### 3.4 A Conceptual Model of Road Hazards

Regression analysis was used to construct accident models on highways in the previous sections in which accident records alone were used as the measure of safety in the regression models. But it is becoming increasingly clear that the use of accident statistics as derived from accident reporting is themselves a problem in defining a measure of safety. On means of complementing accident data for a more comprehensive measure of traffic safety is the use of traffic conflict data, in which a traffic conflict is a precise, observable driver response to a perceived hazard, usually in the form of driver behavior...
induced by a "near miss" situation. It is presumed that a combination of accident statistics and "near misses" is some combination will provide a more accurate concept of road safety than accident statistics alone. This section discusses a hypothesis of an Empirical Bayes hazards model by using a combination of accidents and traffic conflicts. The discussion here is mainly on conceptual model derivation based on empirical evidence from 10 urban intersections. Some exercises are presented to show the potential application of the model.

3.4.1 Introduction

It is known that an accident is a rare event, and random variations are inherent in accident statistics. Therefore, historical accident data do not always reflect long-term accident characteristics accurately (see, e.g., Higle and Witkowski, 1988). A site with a low accident rate in the long run may still have a high accident rate over a short period of time, and vice versa. In short, the hazard of an entity (e.g., a road section) cannot always be satisfactorily represented by the previous accident record of that entity. To overcome this problem, the Empirical Bayes (EB) method for estimating hazard has been explored elsewhere (see, e.g., Abbess, Jarrett, and Wright, 1981; Hauer, 1992).

It is pointed out by Hauer (1992) that a hazard can be defined as the expected value of accident occurrence. As such, the randomness feature of the hazard could be represented in the safety definition. With this consideration, Bayesian methods have been used for road safety studies. In essence, Bayesian methods are distinguished from regression
methods by the fact that parameters are regarded as random variables having specific probability distributions.

Two kinds of clues are used to explain the hazard of an entity: first, clues contained in traits such as traffic, geometry, and driver behavior; and second, clues of the history of accident occurrence. The first kind of clues might be represented by the estimation of hazard in a reference population which the study entity belongs to. The second kind of clues could be represented by actual hazard records (e.g., number of accidents) of that entity.

In Higle et al's study (1988), the observed accident rate at each site is used in combination with the gross estimate of the regional probability distribution to obtain the site-specific probability density functions by using Bayes' theorem,

\[ f_i(\lambda|N_i, V_i) \propto f(N_i|\lambda, V_i) \cdot f_R(\lambda) \tag{3.1} \]

where, \( N_i = \) number of accidents at location \( i \),
\( \lambda = \) accident rate at location \( i \),
\( V_i = \) number of vehicles passing through location \( i \),
\( f_i = \) accident rate probability density function at location \( i \),
\( f_R = \) accident rate probability density function across the region.

More specifically, the relationship can be given by a Gamma distribution:
\[ f_i(\lambda|N_i, V_i) = \frac{\beta_i^{\alpha_i}}{\Gamma(\alpha_i)} \lambda^{\alpha_i-1} e^{-\beta_i \lambda} \]  

(3.2)

where, \( \alpha_i \) and \( \beta_i \) are parameters to be estimated by moments estimates (MME) or maximum likelihood estimates (MLE).

Therefore, site \( i \) is hazardous if the probability is greater than \( \delta \) that its expected accident rate \( \lambda \) exceeds the observed regional accident rate \( \lambda_R \), i.e.,

\[ P(\lambda_i > \lambda_R|N_i, V_i) > \delta \]  

(3.3)

Although Hauer's method (1992) is slightly different from Higle's, the basic assumptions are the same, i.e., (1) at any site, when the accident rate is known, the actual number of accidents follows a Poisson distribution, and (2) the probability distribution of the regional accident rate is the Gamma distribution. He proposed an EB approach with the combination of multivariate regression method to estimate hazard of an entity. We briefly describe the approach as in the following. A multivariate regression model is used to estimate expected mean value, \( E(m) \), of hazard in the reference population as a function of the independent variables. This is based on the belief that \( E(m) \), which depends on the independent variables in some systematic way, can be captured by a model. Those independent variables are traits. A residual could be calculated for each observation as a result of regression analysis. Next, we can estimate the variance \( \text{VAR}(m) \) in reference
population using:

\[ \text{VAR}(m) = \text{VAR}(x) - E(m) \]  \hspace{1cm} (3.4)

Where, \( \text{VAR}(x) \) is the squared residual.

A good estimator of the hazard \( m \) for a specific entity is given by

\[ m = \alpha E(m) + (1 - \alpha)x \]

with \( \alpha = \frac{E(m)}{E(m) + \text{VAR}(m)} \) \hspace{1cm} (3.5)

where, \( x \) = recorded accidents

However, both these approaches (Hauer's and Higle et al's) use only accident records from different sources to model road hazards. Another possible approach is to modify the hazard measure of an entity, i.e., instead of using accident records only, include also other indicators of hazard such as traffic conflicts.

3.4.2 Bayesian Method with Information from Traffic Conflicts

It is believed that information from other sources could be useful to improve the estimation of hazard. If we treat conflicts as an intermediate stage between accident and safety, the hazard index may include traffic conflicts as part of the measure. Here, we use traffic conflict as a measure of accident potential, rather than as "accident surrogate". For this discussion, an accident surrogate may be described
thusly as Glauz et al (1085) state: "accidents are so rare, statistically, that one must often wait for years, and for many accidents to happen, before enough data are available to enable rational decisions". Therefore, "if a surrogate measure such as traffic conflicts could be used, decisions might be made much more quickly". In this study, however, it is assumed that accident data are generally available. The difficulty of using accident data is due to random variations inherent in accident occurrence. To reduce the randomness, traffic conflicts are employed as another source of information to enrich the accident data. This is a process of information gain implied in the EB method. On the other hand, the more important reason of inclusion of traffic conflicts is that we really want to treat it as an indicator of accident potential. Accident potential is an unrealized hazard event (e.g., driver may take evasive action to avoid an incipient crash), while the accident is a realized hazard event. Nevertheless, the hazardous elements largely exist in the unrealized hazard event. It is, therefore, proposed that to measure the full degree of hazard unrealized hazard events should be considered in conjunction with accidents.

However, to use traffic conflict information in the analysis, it may be necessary to convert traffic conflict into accident expectation in order to conform with the unit of accident records. This can be realized by the use of accident/conflict ratios.

The general form of Bayes' rule can then be expressed in the representation of posterior distribution:
posterior \propto likelihood \times prior \quad (3.6)

Where, the prior is the initial estimate of the occurrence of hazard events before new information is collected. The likelihood distribution is the representation of information gained from another source. Here, we may specify the site accident records as prior, and site traffic conflict as likelihood. The posterior represents the modified assessment of hazard events.

In fact, Glauz et al (1985) examined this possibility. After calibrating the accident/conflict ratios, they estimated the expected number of accidents from traffic conflict rate. Then they developed a so-called "minimum variance predictions" method,

\[ \hat{A}_m = \left[ \frac{\hat{A}_0}{\text{Var}(\hat{A}_0)} + \frac{\hat{A}_a}{\text{Var}(\hat{A}_a)} \right] \text{Var}(\hat{A}_m) \]

where

\[ \text{Var}(\hat{A}_m) = \frac{1}{\frac{1}{\text{Var}(\hat{A}_0)} + \frac{1}{\text{Var}(\hat{A}_a)}} \]

where, \( \hat{A}_m \) = expected accident rate with minimum variance,

\( \hat{A}_0 \) = expected accident rate based on traffic conflict rate,

\( \text{Var}(\hat{A}_0) \) = variance of \( \hat{A}_0 \),

\( \hat{A}_a \) = expected accident rate based on accident data,

\( \text{Var}(\hat{A}_a) \) = variance of \( \hat{A}_a \).

Actually, this is a Bayesian posterior estimator (Freund 1984).
However, it must be pointed out that this expression is based on the presupposition that the frequency densities of both traffic conflicts and accidents are normally distributed. However, this may not be true. In some cases, accidents may follow a Poisson distribution at a site. Furthermore, a more general distribution, Gamma distribution, may be employed to represent site accidents. In Bayesian method, if we use Gamma as the prior distribution, the conjugate likelihood for a Gamma distribution is a Poisson distribution.

Therefore, equation (3.6) can be expressed specifically as

\[
\frac{(\text{Gamma})_{\text{posterior}}}{(\text{Gamma})_{\text{prior}}} \propto \frac{(\text{Gamma})_{\text{prior}} \times (\text{Poisson})_{\text{likelihood}}}{m^a e^{-bm}, m^b e^{-m}}
\]

(3.7)

Apparently, the parameters of posterior distribution are

\[
a_i = a + x \\
b_i = b + 1
\]

(3.8)

Thus, the revised mean and variance are

\[
\text{mean} = \mu = \frac{a_i + 1}{b_i} = \frac{a + x + 1}{b + 1}
\]

\[
\text{variance} = \sigma^2 = \frac{a_i + 1}{b_i^2} = \frac{a + x + 1}{(b + 1)^2}
\]

(3.9)

To illustrate the application of this method, we take advantage of the data obtained in a traffic conflict study (Brown, 1994). Traffic conflict data at ten intersections in Vancouver are available. Previous
five years' accident data are also collected for the analysis. These are shown in Table 3.8.

The estimated yearly accident numbers based on traffic conflict information are determined by using mean accident-conflict ratio, \( \pi \) (=2.95*10^-4), computed in Brown's study (1994). This estimation will be used as the likelihood in the Bayesian method.

After determining parameters \( a \) and \( b \) for prior (Gamma) distribution based on five year's accident records, we can calculate new parameters for posterior distribution by using equation (3.8). Then, the revised estimation of accident number could be computed using equation (3.9). Table 3.9 shows this result and the actual accident record in the year when the traffic conflict data were collected. To make a comparison, accident sample mean, i.e., average accident numbers based on previous five years records are also included.

Several observations follow. First, Bayesian method provides closer estimations to actual accidents for six locations. Second, the mean value of these ten points given by Bayesian method is also closer to actual mean than that from the average of five years' accidents.

Notice that to simplify the illustration, we employed total number of traffic conflicts at each location. The analysis based on traffic conflict types would give detail analysis and might provide better results.
<table>
<thead>
<tr>
<th>Intersections</th>
<th>5-years</th>
<th>Conflicts*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Accidents</td>
<td>(TC)</td>
</tr>
<tr>
<td>Oak/SW Marine</td>
<td>57</td>
<td>119</td>
</tr>
<tr>
<td>Granville/Drake</td>
<td>130</td>
<td>121</td>
</tr>
<tr>
<td>Seymour/Drake</td>
<td>58</td>
<td>94</td>
</tr>
<tr>
<td>Dunbar/SW Marine</td>
<td>25</td>
<td>93</td>
</tr>
<tr>
<td>Nanaimo/Hastings</td>
<td>129</td>
<td>45</td>
</tr>
<tr>
<td>Semlin/2nd Ave.</td>
<td>12</td>
<td>6</td>
</tr>
<tr>
<td>Gladstone/27th Ave.</td>
<td>14</td>
<td>1</td>
</tr>
<tr>
<td>Heather/12th Ave.</td>
<td>84</td>
<td>200</td>
</tr>
<tr>
<td>Main/10th Ave.</td>
<td>52</td>
<td>80</td>
</tr>
<tr>
<td>Blenheim/41st Ave.</td>
<td>52</td>
<td>40</td>
</tr>
</tbody>
</table>

* Conflicts were observed for a two-day period.

Table 3.8 Traffic Conflict Data, Vancouber, BC

There are several other possibilities in defining prior and likelihood distributions. One is using regional accident sample to determine parameters of the Gamma distribution, rather than using site accident records. In this case, we can usually get a larger sample size and probably a better estimation of parameters. Another possibility is using estimations from traffic conflict information across the region to determine parameters of the Gamma distribution. In this case, the traffic conflict is assumed to be Gamma distributed, while site accidents are Poisson distributed. Although the verification of these distributions were not conducted, the exercise show some interesting
Table 3.10 indicates that sample mean has only one best estimation. Three Bayesian estimations have two, three, and four best ones respectively. The estimation based on regional accidents as prior (Gamma) distribution produces the best mean value of the ten intersections, while the calculation based on site accidents as prior (Gamma) distribution gives the closest estimations to actual accidents. The sample mean method gives the poorest estimation.

On the other hand, the error sum of squares (SSE) given in Table...
3.10 indicates that using regional TC as prior (Gamma) distribution has the smallest SSE and sample mean method provides the second smallest SSE; while using regional accidents as prior (Gamma) distribution gives the largest SSE.

The results show a promising method to estimate hazards, though the sample is not large enough to provide further detailed statistical analysis. However, since traffic conflicts are relatively frequent events at road accesses, particularly at intersections as defined here, it seems reasonable to assume that further study, with sufficient conflict data, may lead to a new method of measuring and defining road safety.
<table>
<thead>
<tr>
<th>Intersections</th>
<th>Sample Mean</th>
<th>Regional Accident as Gamma</th>
<th>Regional TC as Gamma</th>
<th>Site Accidents as Gamma</th>
<th>Actual Accidents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oak/SW Marine</td>
<td>11.4*</td>
<td>19.1</td>
<td>12.4</td>
<td>14.8</td>
<td>11</td>
</tr>
<tr>
<td>Gastown/Drake</td>
<td>26.0</td>
<td>19.4</td>
<td>26.4*</td>
<td>23.6</td>
<td>36</td>
</tr>
<tr>
<td>Seymour/Drake</td>
<td>11.6*</td>
<td>15.6</td>
<td>12.6</td>
<td>14.3*</td>
<td>14</td>
</tr>
<tr>
<td>Dunbar/SW Marine</td>
<td>5.0</td>
<td>15.5</td>
<td>6.3</td>
<td>7.0*</td>
<td>8</td>
</tr>
<tr>
<td>Nanaimo/Hastings</td>
<td>25.8*</td>
<td>8.8</td>
<td>26.5</td>
<td>19.0</td>
<td>29</td>
</tr>
<tr>
<td>Semlin/2nd Ave.</td>
<td>2.4</td>
<td>3.4</td>
<td>3.8</td>
<td>2.2*</td>
<td>0</td>
</tr>
<tr>
<td>Gladstone/27th Ave.</td>
<td>2.8</td>
<td>2.7</td>
<td>4.2</td>
<td>2.2*</td>
<td>0</td>
</tr>
<tr>
<td>Heather/12th Ave.</td>
<td>16.8*</td>
<td>30.3</td>
<td>17.6</td>
<td>27.0</td>
<td>19</td>
</tr>
<tr>
<td>Main/10th Ave.</td>
<td>10.4</td>
<td>13.7*</td>
<td>11.5</td>
<td>10.6</td>
<td>15</td>
</tr>
<tr>
<td>Blenheim/41st Ave.</td>
<td>10.4</td>
<td>8.1*</td>
<td>11.5</td>
<td>8.9</td>
<td>8</td>
</tr>
<tr>
<td>Mean</td>
<td>12.26</td>
<td>13.68*</td>
<td>13.24</td>
<td>12.96</td>
<td>14</td>
</tr>
</tbody>
</table>

*SSE* \[= \sum_{i=1}^{n} (Y_i - \hat{Y}_i)^2\], where, \(Y_i\) represents actual accidents and \(\hat{Y}_i\) represents the estimated values.

* The value is the closest to the actual accidents in each row.

† This column is the same as the "Bayesian Estimation" column in Table 3.9.

Table 3.10 Bayesian Estimation Results (2)
Chapter 4

The Effect of Access on Traffic Operations

In this chapter a model of travel speed is obtained on the basis of access and other geometric information for two-lane highways.

4.1 Introduction

Some previous efforts have been made to determine the effects of access on traffic operations. However, most studies either did not intend to establish quantitative relationships, or were unsuccessful in establishing the relationship. One partial exception is the new Highway Capacity Manual (HCM), 1992, which does deal with the access/traffic operations relationship on multilane highways but not for two lane highways. This chapter examines the relationship between access and traffic operations in road sections on two-lane highways by means of the photolog data base. Since speed is used as a conditionary measure of level of service for two-lane highways in capacity calculations, and since it is relatively easier to obtain speed information than percent time delay along a road section, travel speed will be used in this research as a univariate proxy measure for traffic operations performance. In this research highway speed is empirically tested against categories of access and several other traffic and geometric road characteristics.

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1 The object of this study is a highway section, not a highway network. For a highway section, access points are an important influence factor. The number of access points in one section may have some effects on the rest of a network, but that is another aspect of the impact, not the focus of this study.
Glennon et al (1975) developed techniques to counteract vehicle delay due to access points, and although it was recognized that for the mainstream through traffic, delay is caused by turning vehicles, no quantitative relationship was established. Research conducted by Reilly et al (1989) for the Revised HCM (1992) found that the number of access points on multi-lane rural and suburban highways has an important influence on free-flow speed, in that for every 10 access points per mile, travel speed will be reduced by 2.5 mph. The access point is defined as a composite variable made up of the separate influences of different characteristics of access. However, the HCM (1992) has not yet been updated to include the effect of access points on traffic operations for two lane highways. One study conducted in the UK (Brocklebank, 1992) on two-lane rural highways found that access variables are generally difficult to fit to models of traffic operations. A composite variable used which synthesized intersections, lay-bys, and accesses was not significant in the UK study. Non-residential accesses and lay-bys as separate independent access variables were found either to be not significant or only occasionally significant. For example, the intersection variable was rather unstable with a coefficient value -1.8 (km/h). One possible explanation for the result is that there are so many other variables considered in the model (a total of 31 independent variables), the effect of access variables may be subsumed. In any case the results do not fully conform to our a priori expectations, and further study of two lane highways is needed. In addition, two-lane highways in urban and suburban areas should also be investigated because the higher traffic and more roadside development on these roads will undoubtedly intensify the influence of access points.
Traditionally, field data are used to quantify traffic flow relationships. For example, Reilly et al (1989) collected field data on multilane, mainly divided, highways in several states of the USA. Video cameras were used to collect traffic operation data (speed and flow), with the information incorporated into a comprehensive data base including geometric characteristics, roadside features, and access points. In all, 45 sites were selected, and hundreds of observation points were coded in the data base. Regression analyses were then performed to establish traffic flow relationships. In Brocklebank’s study (1992) a license plate matching technique was employed to record traffic information. The data base is comprehensive, including 31 flow and geometric variables with access variables among them. Because the cost of such data collection is very high, it impedes comprehensive studies of access and other constraint factors in traffic operations efficiency; and because of cost is not a feasible alternative for this present study.

To reduce data collection costs, simulation models are sometimes employed to estimate traffic impacts. Venigalla et al (1992) used the TRAF-NETSIM simulation program to compare operational effects of non traversable medians and two-way left-turn lanes. No real data collection was conducted. A number of scenarios with varying conditions were studied, with appropriate assumptions made to simplify simulation analysis. Driveway density was also considered in the study, but the main focus was on comparing alternative median designs for multilane highways. Driveway density was treated as a secondary, category scaled variable. The Colorado Demonstration Project (1985) utilized TRANSYT 7F
to evaluate the effect of access points on congestion for a five-mile road section, but again, the access variable was measured by a category scale: controlled or uncontrolled access.

One disadvantage of using a simulation model for this problem is that there are some assumptions that may not be verified. For instance, it is not clear whether delay (speed reduction) is solely caused by turning movements (this is generally adopted in simulation programs) or by the turning movement and the mere existence of access points. Revision of the HCM (1992) contends that access points may reduce travel speed because drivers adjust their travel speed when they see the existence of access points (even with no turning movements involved). Apparently, this psychological response cannot be simulated without more empirical data support. The simulation program replicates the mechanism of speed reduction caused by turning movements by using kinematic equations. Therefore, some actual data is needed to reveal any applicable correlation between access and speed. To do this and overcome the problem of the high cost of data collection, a search was made to see if any existing data sources could be used, and indicated that only small amounts of travel speed data for two-lane highways existed in the province of British Columbia. Nevertheless, it was found that in the photolog data source much relevant ancillary information was available, and consequently it was decided to use the photolog as a data base with further analysis of the speed variable to make it useful as a proxy for operations performance.
4.2 The Photolog Data

Besides geometric information (such as grade and horizontal curvature), the instantaneous speed of the test vehicle and the cumulative time are also recorded in the photolog data. While the photolog provides only the test vehicle's speed and travel time, if the travel distance is relatively great, or, in other words, if successive observation points are relatively distant, the test vehicle's travel speed can be used as a surrogate for average travel speed in the access and speed relationship. Two arguments are presented here to support this contention. First, we may treat the method as a type of moving vehicle technique, which is widely employed to collect travel time data. In this process, the subject moving vehicle is placed in the traffic stream to simulate an average vehicle in the traffic stream. (This is what our photolog test vehicle driver was asked to do.) Therefore, the travel time recorded on a road section by the photolog vehicle is used as the mean travel time for all traffic on the section. On the basis of this mean travel time, the travel speed can be computed by section length. Since section length is well defined, if the travel time obtained from the test vehicle can be accepted as the mean travel time as many studies do (see, e.g., Nutakor, 1992), it follows that the derived travel speed can be used as the average travel speed. In the standard moving vehicle method, the test vehicle is required to run several times in a study section, while the data base for this study gives only one run for each section. To allow for this, we recognize that the sample in this study does not consist of a number of runs in a road section, but rather consists of hundreds of sections. A single observation on one section may cause sampling error, one of the major sources of error in most
regression problems. Sampling error arises from random variation of the observations around their expected value. However, sampling error is inherent to the regression model and is normally represented as part of the random error term $\varepsilon_i$. Furthermore, from a traffic engineering point of view, the variance of travel speed in a homogeneous section is usually much smaller than that by section. Which means that the range of random variation of speed observations is small and ensures the approximate representation of the magnitude of expected speed values.

The second argument for the use of section travel speed as a proxy for average speed pertains to the fact that the focus of this study is on the general relative impact of access density on the average travel speed. What we are really interested in is the variation of travel speed rather than the absolute value of average travel speed on a section. It is assumed that the percentage speed change is nearly the same for most experienced drivers when they confront a hazard or other reason for reducing speed. Which means that experienced drivers reduce speed proportional to their original travel speed. Although this is not verified in this study, it seems to be a reasonable approximation based on engineering judgment. Therefore, the speed change of the test vehicle would approximate the general impact of access on traffic operations (speed).

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2 Others are specification error and measurement error, see e.g., Hawkins and Weber, 1980.

3 The use of the test vehicle data instead of traffic survey data is due mainly to practical limitations of the research. The very significant cost of collecting actual travel speed data on the one hand and the increasingly pressing need to understand the problem on the other has prompted this approach to the research.
The study data consist of 255 sections (sites) on 14 two-lane routes photologged in 1990. The total lengths of selected road sections are 625 kilometres, with an average length of 2.43 kilometres.

Seven types of road access were originally identified: (1) four-way unsignalized intersection; (2) three-way unsignalized intersection; (3) commercial driveway; (4) industrial driveway; (5) residential driveway; (6) agricultural driveway; and (7) roadside pullout. However, preliminary model testing indicated that the above classification resulted in some insignificant access variables. Three composite variables for access were then created as: (1) unsignalized intersection; (2) business access (commercial and industrial driveways); (3) and private driveway (residential and agricultural driveways). Roadside pullout was retained in its original form. This classification follows a functional definition of access and presumably resembles access volume categories. (Note that the term "intersection" is defined here as the general area where two or more public roads join or cross. This definition will distinguish the intersection from other access types, such as private access, where one private driveway is connected to one public road.)

Traffic volume is a key variable in any study of traffic flow, but instantaneous traffic flow rates were not available in the photolog data. Therefore average annual daily traffic (AADT), obtained from published traffic count station reports, was used to study the speed-flow relationship. Since it is convenient to use hourly volume instead of AADT, the AADT was converted to hourly flow rate based on the date.
and time of the photolog data. The peak hour adjustment factor was first computed. Since the actual time when the test vehicle was traveling was not always in the peak period, further adjustment was made to estimate the flow rate at the time of the test vehicle run over the section. Access traffic volumes, or turning traffic, of each access point were not available in any case, thus only major road traffic was included. The cost of collecting these access volumes could be much higher than that of main road traffic, considering the number of access points in a road section and number of sections in the study. However, it is believed that the grouping of accesses by functional characteristics as outlined here helps alleviate the necessity for access volumes since each access function can by and large be associated with crude variations in volume ranges. On the other hand, the priority movement privilege of main road traffic reduces the effect of conflict movement from access traffic. Traffic controls such as stop signs are utilized extensively on arterial roads to ensure largely uninterrupted movement of main traffic. It does not matter how many vehicles in the queue at an access point are waiting for an opportunity to cross or merge into the main traffic. Only the first vehicle has the potential to interfere with main traffic flow operations. The difference between higher access traffic and lower access traffic volume at a minor road is that the former case usually means more delay for the minor road access traffic. Although turning lanes might not be provided for main road traffic at all access points, it is, however, presumed that they are accommodated at access points where turning traffic from the main road constitutes a significant portion of through traffic. This should be true in a well-developed region. Since this study is not a microscopic operational analysis of delay at individual access point, but rather a macroscopic
analysis of the effect on traffic in road sections, some degree of aggregation of information would seem permissible and necessary, (i.e., the use of access types to represent the magnitude by classes of access traffic volume).

Besides, as indicated earlier, it is not clear yet if the speed reduction is actually caused by access traffic or the mere presence of access points, and the result of psychological response. This hypothesis, as well as the use of access types to account for access traffic volume, can reasonably be justified if the correlation between travel speed and access density which is classified by types can be statistically established.

Travel speed is defined as the section length divided by over the road travel time. The instantaneous speed recorded in the photolog data was not suitable for calculating the travel time, since it measures the speed of the test vehicle at each individual point in the section. It does not give the actual travel time in a road section. However, the real time of the survey was registered, and the section length divided by the time interval used for traveling through that section gives the travel speed of the test vehicle in that section. It is presumed that the relationship between the test vehicle's travel speed under different traffic conditions and the corresponding flow rate will resemble the relationship between average travel speed (space mean speed) and flow. Figure 4.1 shows the test vehicle's travel speed and flow relationship. It appears to have the same shape and similar magnitudes as a standard, known, speed-flow curve. This relationship is explored in detail in the next section.
Figure 4.1  Travel Speed-Flow Rate Relationship
Several road characteristic factors were also available from the photolog. Some of them were included in this analysis, such as grade, traverse slope, horizontal curvature, and the direction of curvature. The latter measure indicates whether the curvature is left or right extended, and therefore the frequency of change in direction per kilometre within the study segment can be calculated. The intention here is to determine the amount of a driver's attention needed to traverse a given segment. Grades were taken as absolute values only, and no direction of grade was considered, largely because Troutbeck (1976) indicated that car speeds depend largely on the magnitude of the grade, rather than the direction. Horizontal curvature is defined as the central angle of a 100 metre arc. Average grade and horizontal curvature were obtained for each road section. These two variables measure average vertical and horizontal displacement, respectively, which are used as road alignment variables in other studies (Duncan, 1974; Brewer et al, 1980; McLean, 1989; and Brocklebank, 1992) to study travel speed and geometry relationships. Two other factors not included in this study are lane and shoulder widths simply because this information was not available in digitized form in the photolog. However, the review of the study sections indicated that the lane and shoulder widths of the selected sites are largely fixed (lane width = 3.6m, and shoulder width = 2.0m). Therefore, it is assumed that any influence on speed is independent of access points, and no effect on possible influences of access.
All variables employed in the analysis are defined in Table 4.1.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
<th>Range</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPD</td>
<td>Travel speed (km/h)</td>
<td>11.02-96.00</td>
<td>74.02</td>
</tr>
<tr>
<td>UNS</td>
<td>Unsignalized intersection density (no./km)</td>
<td>0 - 8</td>
<td>1.01</td>
</tr>
<tr>
<td>BSN</td>
<td>Business access density (no./km)</td>
<td>0 - 20</td>
<td>1.00</td>
</tr>
<tr>
<td>PWY</td>
<td>Private driveway density (no./km)</td>
<td>0 - 35</td>
<td>3.80</td>
</tr>
<tr>
<td>RSP</td>
<td>Roadside Pullout (no./km)</td>
<td>0 - 1.5</td>
<td>0.15</td>
</tr>
<tr>
<td>VOL</td>
<td>Two-way flow rate (veh/h)</td>
<td>51 - 2,500</td>
<td>664</td>
</tr>
<tr>
<td>GRD</td>
<td>Grade (percent)</td>
<td>0.19 - 6.3</td>
<td>1.53</td>
</tr>
<tr>
<td>TS</td>
<td>Traverse slope (percent)</td>
<td>0.87 - 4.93</td>
<td>2.87</td>
</tr>
<tr>
<td>FCD</td>
<td>Frequency of change in the direction of</td>
<td>0.83 - 25</td>
<td>10.38</td>
</tr>
<tr>
<td></td>
<td>curvature (no. of changes/km)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HC</td>
<td>Horizontal curvature (degree)</td>
<td>0.3 - 14.9</td>
<td>3.96</td>
</tr>
</tbody>
</table>

Table 4.1 Variable Definition

4.3 Data Analysis

To verify a speed-flow relationship using test vehicle speed, a linear regression analysis was conducted with a resulting $R^2$ of 0.23, and F test of 75.44, a significant correlation. To attempt to increase the $R^2$ the data were disaggregated into two horizontal curvature (HC) categories: HC less than 5 degrees, and HC greater than and equal to 5 degrees. In the first group, the $R^2$ was 0.38 and F test was 113.64, which indicates a notable improvement. While in the second group, $R^2$ is 0.07 and F test is 0.024. The plots of observation points of these two
groups are shown in Figures 4.2 and 4.3. Clearly most road sections with higher traffic volume are designed with lower horizontal curvature as shown in Figure 4.2 with a large number of sections showing both high speed and higher volumes with the horizontal curvature less than 5°. On the other hand, road sections with greater horizontal curvature usually accommodate lower traffic volume as shown in Figure 4.3. However, there is no apparent speed reduction for the few sections in the higher horizontal curvature group. Specifically, it seems that horizontal curvature greater than 5° does not greatly affect speed. This unexpected result may be explained by the fact that at low traffic volumes, drivers more likely travel at their own desirable speed. Another possible reason is that the number of observation points, 68, may be relatively small.

Another test was made based on speed limit disaggregation. Speed limits were categorized in two groups: speed limits less than and equal to 70 km/h, and speed limits greater than 70 km/h. The plots of these two groups are shown in Figures 4.4 and 4.5. In the first group, $R^2$ is 0.34 and F test is 0.66, so it is insignificant. The second group provides a better result: $R^2$ is 0.34 and F test is 102.81. It can be seen that in the low speed limit group travel speed varies over a wide range at low traffic volume (the left portion of Figure 4.4), which indicates, as expected, that drivers more or less ignore the speed limit, and whenever the volume is low he or she may simply drive faster. However, at the higher speed limit (the second group) the actual travel speed is more consistently higher except when the traffic volume is very high. Again, perhaps the poor correlation in the low speed limit group can also be attributed to relatively few observations, 53, in that group.
Figure 4.2 Speed-Volume on Road Sections with HC < 5 degree
Figure 4.3  Speed-Volume on Road Sections with HC >= 5 degree
Figure 4.4  Speed-Volume on Road Sections with Speed Limit $\leq 70$ km/h
Figure 4.5  Speed-Volume on Road Sections with Speed Limit >= 80 km/h
Since the disaggregation of data does not markedly improve the results for all groups, it was therefore decided to use the aggregated data to examine the relationship between access and speed. It is believed that the statistical test for aggregated data has shown that the speed and volume relationship is reasonably linear, and furthermore, the use of test vehicle speed is an appropriate indirect measure of average travel speed.

Consequently, a backward stepwise multiple linear regression analysis was carried out. The results of this initial model are shown in Table 4.2. It can be seen that most independent variables except roadside pullout and traverse slope were satisfactorily entered into the model at significant level 0.01. All variables appear with the expected signs. The F-test is also satisfactory at 0.01 level of significance, indicating that all independent variables' coefficients are collectively significantly different from zero. Note that the ratio of the coefficients between access variables (in the order of intersection, business access, and private driveway) is approximately 1:2:4. The insignificant effect of roadside pullout on speed agrees with the effect of lay-bys in Brocklebank's study (1992). This might be due to infrequent use of roadside pullouts in most cases; and secondly, the poor design standards of the pullouts make it not easily perceived by drivers so that their behaviors are not influenced by the mere existence of these pullouts, without conflicting traffic.
<table>
<thead>
<tr>
<th>Independent Variable</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>t-Test</th>
<th>Level of Significance</th>
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</tr>
<tr>
<td>BSN</td>
<td>-0.89</td>
<td>0.29</td>
<td>-3.07</td>
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</tr>
<tr>
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</tr>
<tr>
<td>VOL</td>
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<td>0.0013</td>
<td>-7.25</td>
<td>0.000</td>
</tr>
<tr>
<td>GRD</td>
<td>-1.71</td>
<td>0.56</td>
<td>-3.06</td>
<td>0.002</td>
</tr>
<tr>
<td>FCD</td>
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<td>0.20</td>
<td>-5.92</td>
<td>0.000</td>
</tr>
<tr>
<td>HC</td>
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<td>0.22</td>
<td>-9.99</td>
<td>0.000</td>
</tr>
<tr>
<td>R²</td>
<td></td>
<td>0.59</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SE</td>
<td></td>
<td>9.63</td>
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<td></td>
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<td>F-Test</td>
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<tr>
<td>(P)</td>
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<td></td>
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<td>0.000</td>
</tr>
</tbody>
</table>

Table 4.2 Initial Model for SPD

The correlation analyses showed no significant multicollinearity among the independent variables, thus producing relatively unbiased estimates. Consequently, the effect of each variable on the accident measure provide some insight. See Table 4.3.

However, an examination of residuals in the regression model indicated relatively poorer results in the lower speed ranges, as Figure 4.6 shows. To solve this problem, a quadratic transformation was made for dependent variable SPD. Table 4.4 and Figure 4.7 show the new
<table>
<thead>
<tr>
<th>Independent Variable</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>t-Test</th>
<th>Level of Significance</th>
</tr>
</thead>
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<td>312.58</td>
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<tr>
<td>UNS</td>
<td>-232.06</td>
<td>59.72</td>
<td>-3.89</td>
<td>0.000</td>
</tr>
<tr>
<td>BSN</td>
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<td>36.37</td>
<td>-3.23</td>
<td>0.001</td>
</tr>
<tr>
<td>PWY</td>
<td>-56.64</td>
<td>18.18</td>
<td>-3.12</td>
<td>0.002</td>
</tr>
<tr>
<td>VOL</td>
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<td>0.16</td>
<td>-8.12</td>
<td>0.000</td>
</tr>
<tr>
<td>GRD</td>
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<td>69.76</td>
<td>-2.11</td>
<td>0.036</td>
</tr>
<tr>
<td>FCD</td>
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<td>25.28</td>
<td>-5.76</td>
<td>0.000</td>
</tr>
<tr>
<td>HC</td>
<td>-293.42</td>
<td>27.42</td>
<td>-10.70</td>
<td>0.000</td>
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</tbody>
</table>

Table 4.3 Correlation Matrix for Coefficient Estimates

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<th>Independent Variable</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>t-Test</th>
<th>Level of Significance</th>
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<td>312.58</td>
<td>32.12</td>
<td>0.000</td>
</tr>
<tr>
<td>UNS</td>
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<td>59.72</td>
<td>-3.89</td>
<td>0.000</td>
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<tr>
<td>BSN</td>
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<td>36.37</td>
<td>-3.23</td>
<td>0.001</td>
</tr>
<tr>
<td>PWY</td>
<td>-56.64</td>
<td>18.18</td>
<td>-3.12</td>
<td>0.002</td>
</tr>
<tr>
<td>VOL</td>
<td>-1.28</td>
<td>0.16</td>
<td>-8.12</td>
<td>0.000</td>
</tr>
<tr>
<td>GRD</td>
<td>-147.12</td>
<td>69.76</td>
<td>-2.11</td>
<td>0.036</td>
</tr>
<tr>
<td>FCD</td>
<td>-145.57</td>
<td>25.28</td>
<td>-5.76</td>
<td>0.000</td>
</tr>
<tr>
<td>HC</td>
<td>-293.42</td>
<td>27.42</td>
<td>-10.70</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Table 4.4 Modified Model for Transformed Variable SPD²
Figure 4.6 Residual Analysis of the Initial Model
Figure 4.7  Residual Analysis of the Modified Model
regression results and residuals plot respectively. The quadratic transformation appears to give better (more consistent) results across the speed range of the study using the original set of independent variables. Furthermore, the ratio of the coefficients between access variables (in the order of intersection, business access, and private driveway) is also approximately 1:2:4. Thus the degree of inconsistency of the initial model would appear to be relatively minor, and the comparative weight of each independent variable has not been changed greatly by the transformation. Only the influence of grade is measurably weaker in the transformed model as shown by the 't' statistic in Table 4.4.

In practice, therefore, unsignalized intersections, business access, and private driveways will reduce travel speed by approximately 1.6, 0.8, and 0.4 km/h, respectively. Compared with previous studies, the values of these results are larger reductions than those in the multilane highway situation as reported by Reilley et al (1989), which showed a speed reduction of 0.25 mph for each access point per mile (i.e., 0.65 km/h for each access point per km), as expected. This may be explained as follows. First, two-lane highways are not divided, and since left-turn movements exist at most access points, these have a much greater influence on traffic operations than right turn movements in the absence of a left turn lane. Second, multilane highways provide more opportunities for following vehicles to pass the vehicles slowing down to make turning movements or to avoid right turning vehicles. Third, in terms of behavioral response to the existence of access points, vehicles on two-lane highways may be affected more than vehicles on multilane highways because the vehicles in the center lanes may be less influenced...
by access points than those vehicles in the curb lanes. Compared with Brocklebank's result (1992), the intersection effect in our initial model (1.6 km/h) is marginally smaller than his (1.8 km/h). However, that effect in our modified model will be enhanced with the increases of intersection density and flow rate. This will be discussed further in the next section.

As for geometric factors (grade, horizontal curvature, and frequency of change in the direction of horizontal curvature), all indicated inverse proportional relationships with speed, i.e., the increase of variable value (absolute value) will reduce travel speed. This generally conforms to Duncan's (1974), Brewer et al's (1980), and Brocklebank's (1992) results for grade and curvature, although their definitions of these two variables are quite different. The effect of the frequency of change in the direction of horizontal curvature has not been found in previous studies, perhaps due to lack of information. However, the negative effect of this variable on travel speed seems to be intuitively correct, as larger values of this variable reflect more complex driving circumstances.

4.4 Results and Discussion

The specific influence of each access type on travel speed was normalized by using a modified model. The normalization was done by varying access variables while holding all other independent variables at their mean values. A secondary independent variable (flow rate) was also employed. For example, to study the effect of intersection density
on travel speed, several flow rate levels were considered while all other independent variables are held at their mean values.

Figure 4.8 through Figure 4.10 show the variations of travel speed with changes in intersection density, business access density, and private driveway density from the modified model. All figures show a decreasing trend line of travel speed with the increase of access densities as determined by the negative coefficients in the model. Note that the estimated relationship curves are for specific degrees of highway curvature. This points to the non-linear characteristics in the access-speed relationship, in that the curves are wider apart at higher access density than at lower access density, which indicates that the effect of access density on speed is intensified by the increase of flow rate.

Another representation of the relationship is shown in Table 4.5, where speed reduction factors were calculated for each access type. Again, all other variables are kept at their mean values while only access density and flows are changing. For each pair of access and flow combination, the speed reduction factor is calculated as the ratio of the current speed and the base speed. The speed at access density equivalent to zero and flow rate equivalent to 500 vph is chosen as the base value for each access type. The purpose of this representation of the relationship is for the convenience of the application of the results. An example is presented below to show how Table 4.5 could be used in practice.
Figure 4.8 Estimated Relationship between Unsignalized Intersection and Travel Speed by Traffic Volume
Figure 4.9 Estimated Relationship between Business Access and Travel Speed by Traffic Volume
Figure 4.10 Estimated Relationship between Private Driveway and Travel Speed by Traffic Volume
<table>
<thead>
<tr>
<th>Accesses</th>
<th>Density (no./km)</th>
<th>Flow Rate (veh/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>500</td>
<td>1,000</td>
</tr>
<tr>
<td>Unsig. Intersection</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1.00</td>
<td>0.95</td>
</tr>
<tr>
<td>2</td>
<td>0.96</td>
<td>0.91</td>
</tr>
<tr>
<td>4</td>
<td>0.92</td>
<td>0.86</td>
</tr>
<tr>
<td>6</td>
<td>0.88</td>
<td>0.82</td>
</tr>
<tr>
<td>8</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>Business Access</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1.00</td>
<td>0.95</td>
</tr>
<tr>
<td>5</td>
<td>0.95</td>
<td>0.89</td>
</tr>
<tr>
<td>10</td>
<td>0.90</td>
<td>0.84</td>
</tr>
<tr>
<td>15</td>
<td>0.84</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>0.78</td>
<td>0.71</td>
</tr>
<tr>
<td>Private Driveway</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1.00</td>
<td>0.95</td>
</tr>
<tr>
<td>7</td>
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</tr>
<tr>
<td>14</td>
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<td>0.88</td>
</tr>
<tr>
<td>21</td>
<td>0.90</td>
<td>0.84</td>
</tr>
<tr>
<td>28</td>
<td>0.86</td>
<td>0.80</td>
</tr>
<tr>
<td>35</td>
<td>0.82</td>
<td>0.76</td>
</tr>
</tbody>
</table>

* The speed reduction factor is calculated as the ratio between each speed and the base speed. The speed at no access points and 500 vph flow rate is chosen as the base speed.
Assume we know a two-lane road section with the conditions as follows: level terrain, no passing lanes, 1000 vph, one unsignalized intersection per km, five business accesses per km, and seven private driveways per km. To determine the effect of these accesses on average speed, first, find the average speed for the base condition: no accesses, 500 vph, which can be obtained from HCM (1985) as approximately 91.7 km/h. Then, get an adjustment factor from Table 4.5: 0.93 * 0.89 * 0.91 = 0.75, which means the speed will be reduced by 26 percent. The estimated average speed is therefore equal to: 91.7 * 0.75 = 68.8 km/h.

Finally, the estimated speed-flow relationship by access density is shown in Figure 4.11, using flow as the primary variable and access as the secondary variable. Access density levels were defined by arbitrarily assigning densities of unsignalized intersections, business accesses, and private driveways in a group. The figure clearly indicates that, with higher flow rate and access density, travel speed drops rapidly.

4.5 Summary Conclusion

To estimate the effect of access on travel speed on two-lane highways, the photolog data source was employed. It is shown that the test vehicle's travel speed can be used as a reasonable proxy for average travel speed if the test vehicle's speeds are reasonably high. This was illustrated by plotting the speed-flow relationship for the test vehicle. It is shown that the shape and the magnitude of the speed-
Figure 4.11 Estimated Relationship between Speed and Traffic Volume by Access Points
flow curve for the test vehicle resembles the known general speed-flow relationship well for two-lane highways (Figure 4.1).

Access points were classified into four types: unsignalized intersection, business access, private driveway, and roadside pullout. Except the last one, all others are significant in the model. The regression analysis indicated that non-linear relationships may exist between these accesses and travel speed. The higher the access density, the more speed reduction will be induced. As intuitively expected, the magnitude of the influence on speed is in the order of intersection, business access, and private driveway, with a ratio of approximately 1:2:4. If all other independent variables remain at mean values, one unsignalized intersection will approximately reduce travel speed by 1.6 km/h, one business access will reduce the speed by 0.8 km/h, and one private driveway will reduce the speed by 0.4 km/h. To more precisely describe the effect of access on speed reduction, Table 4.5 should be used, with the following caveat: it is presumed that the influence of access on speed is due to the combined effect of turning movements at access points and the mere existence of the access points. However, no attempt was made to differentiate between these two factors in the study.

One possible use of the findings of this chapter is in the estimation of road user cost variation with access density. With the change of average travel speed, as a result of change in access density, delay costs could be estimated. By including other relevant costs, such as accident costs and out-of-way travel costs for access traffic, an
optimal access density might be achieved to minimize the total road user costs in an access control program.
Chapter 5
A Framework for Planning Optimal Number of Access Points

The study of the optimal number of access points is necessary for highway planning and design in terms of access density, and for highway control in terms of access permit regulations. The aim of this chapter is to establish a procedure for defining the optimal number of access points with considerations of not only traffic safety but also traffic delay. The framework developed here can not be all inclusive because of data limitations; particularly those data related to side road delay and non-user environmental costs and benefits. The procedure is strategic, directed to questions related to highway planning, and is concerned with a relatively wide range of road sections instead of the minimum geometric distance between two adjacent access points.

5.1 Model Construction

5.1.1 General Discussion

Before presenting decision variables, independent variables, the objective function and constraints, the basic form of model objective is discussed. This discussion will provide a base and a scope to elaborate on variables, objective function, and constraints of the model.

The model objective here is to minimize the total social costs, which include the traffic accident cost and delay cost induced by access points. Specifically, the delay costs consist of two parts: main traffic
flow delay, and minor traffic flow delay. Thus the basic form of model objective is:

\[
\text{Minimize } Z = c_1 f_{\text{acd}} + c_2 f_{\text{spd}} + c_3 f_{\text{dly}} \tag{5.1}
\]

where,

- \( f_{\text{acd}} \) represents the access-accident function,
- \( f_{\text{spd}} \) represents access-speed function for main traffic flow,
- \( f_{\text{dly}} \) represents delay function for minor traffic flow at access points and on side road,
- \( c_1, c_2, \) and \( c_3 \) represent cost coefficients.

Note that \( f_{\text{acd}} \) is obtained from Table 3.2, and \( f_{\text{spd}} \) is obtained from Table 4.2. Only \( f_{\text{dly}} \) is to be derived.

In model construction, at this stage, only two-lane highways are targeted for study. However, the analysis method presented here might be applied to other road types in the future.

It is assumed that the roadside developments on both sides of the main road are homogeneous. This assumption ensures trip generations and attractions are evenly distributed in the area abutting the main road. The assumption is necessary because of data limitations, but in any case could appear valid for residential areas.
5.1.2 Decision Variables

Decision variables to solve for are access variables illustrated in previous chapters. These access variables are in the form of number of access points for each access type. Five access types are discussed for two-lane highways: signalized intersection $n_1$, unsignalized intersection $n_2$, business access $n_3$, private access $n_4$, and roadside pullout $n_5$. However, signalizing an intersection is basically a traffic control issue rather than a planning issue. The need for signalizing an intersection increases with the increase of side road traffic. The intersection must already have been planned and constructed. On the other hand, the effect of signalized intersection on traffic operation is related to signal coordination in a network, while this study only concentrates on road sections. Therefore, the type of signalized intersections is excluded from the present study.

Unsignalized intersection is the most basic access type on a road. Study of this basic access type would provide some essential insight understanding of the problem.

Other low level access types, i.e., business access, private access, and roadside pullout, do not contribute to side road traffic delay. This is because the side road traffic is so low at these access points that no queue is usually built up. Therefore, no side road traffic delay is involved at these access points. It can be seen from Chapter 3 and Chapter 4 that the effect of these access points on $f_{acd}$ and $f_{spd}$ is to increase the accidents and reduce the travel speed in the
whole density range. Which means that accident cost and travel time cost will be increased as the numbers of these access points increase anywhere in the practical access density range. Therefore, to minimize the total cost these access densities should be minimized to zero. However, this is not practical. It is therefore not necessary to include these access points in the optimization model. The provision of these accesses should be based on the spacing of unsignalized intersection and other considerations, e.g., minimum access spacing in terms of geometric design criterion.

There is another problem with business and private access provisions. Especially for business sites, access to the main road is very important. Presumably the access provision (number, type, geometric design) may affect the sales of a business and the land value of this location. Conceptually, it may be argued that the more access points and the higher standard of access design, the higher the value of the adjacent land lot. If the price change of the land could also be regarded as a kind of social cost, then to include business access in the model we may consider the change of land value as a result of access provision. The general function is

\[ P = f_{\text{land}}(n_3) \]  

(5.2)

where,

P is average road side land price in a section,
Then this component could be incorporated in our general model. However, a conversion must be made to convert this price into cost. Conceptually, the lower the price, the higher the cost. However, it is not easy to define this function and requires much more information about land prices. Due to this lack of the information at this time, the function was not defined in this study.

5.1.3 Independent Variables

Independent variables are those with known values. They are inputs of an optimization model. Based on the discussion in section 5.1.1, these independent variables are: average annual daily traffic $A_q$ on the main road, average annual daily traffic $A_q$ on side roads, peak hour traffic $Q$ on the main road, peak hour traffic $q$ on side roads, two-lane highway section length $L$ in km, speed limit $S_1$ on the main road and $S_2$ on side roads, and geometric variables $X_{10}$, $X_{12}$, $X_{13}$ as defined in Chapter 3 as grade, frequency of changing direction of curvature, and horizontal curvature, respectively. Besides, cost coefficients are also inputs of the model.

5.1.4 The Objective Function

For the basic form of the model objective in section 5.1.1, note that units of $f_{\text{acd}}$ (access-accident function), $f_{\text{spd}}$ (access-speed function for main traffic flow), and $f_{\text{dly}}$ (delay function for minor traffic flow at access points and on parallel minor road) are quite
different. $f_{acd}$ gives number of accidents per kilometre per day (accident rate), $f_{spd}$ gives kilometre per hour (travel speed), and $f_{dly}$ gives hour per vehicle (delay time). If we have cost coefficients $c_1 = \$/accident, and $c_2 = c_3 = \$/veh-hr, we may rewrite our objective function equation (5.1) as

$$\text{Minimize } Z = c_1 f_{acd} L + c_2 \left( \frac{L}{f_{spd}} - \frac{L}{f_0} \right) A_2 + c_3 f_{dly} A_3$$

(5.3)

$c_i$, $f_{acd}$, $f_{spd}$, $f_{dly}$, $L$, $A_0$, $A_q$ are prescribed. $f_0$ is $f_{spd}$ with unsignalized intersection points $n_2 = 0$, km/h. The inclusion of this factor is to calculate increased travel time due to access points. Therefore, $f_0$ represents a base speed in the situation of no access points in a road section.

As indicated earlier, $f_{dly}$ is unknown yet. In defining $f_{dly}$, two components of delay will be considered for minor traffic flow. The first one is the stop delay at intersections, and the second is the delay due to slower speed of traffic on parallel minor road. In the second part, $L/4n_2$ is the average travel distance on minor road and is derived in Appendix A. The travel time is the distance divided by speed, and the time saving is the difference between the two. It can be expressed as

$$f_{dly} = f_0 + \frac{L}{4n_2} \left( \frac{1}{S_1} - \frac{1}{S_2} \right)$$

(5.4)

where,

$S_1$, $S_2$, $L$, and $n_2$ are prescribed

$f_a$ is average delay of minor traffic at an access point.
Note that $f_a$ is derived in Appendix B and is a nonlinear function of $n_2$.

### 5.1.5 The Constraints

Several constraints are formulated as follows. They define the solution space for the problem.

The accident rate should be less than or equal to some critical value, $ACD$, specified by the authorities to meet safety requirement and policy. On the other hand, it cannot take negative value.

$$0 \leq f_{acd} \leq ACD$$ \hspace{1cm} (5.5)

The stop delay for minor traffic flow at access points only should be less than or equal to some critical value, $t_c$, to maintain a certain level of service (e.g., LOS D), i.e.,

$$0 \leq f_a \leq t_c$$ \hspace{1cm} (5.6)

A desirable level of service on the main road should be maintained, i.e., traffic speed on the main road should be no less than a specified value, $S_i$, which is the average travel speed defined in Highway Capacity Manual (1985) for level of service of $i$.

$$f_{spd} \geq S_i$$ \hspace{1cm} (5.7)
Theoretically, design speed or speed limit is another constraint, i.e., estimated average main traffic travel speed should be lower than design speed, DSP,

\[ f_{spd} \leq DSP \]

However, travel speed should not be constrained by access provisions; rather it should be constrained by speed limit regulation and necessary enforcement. In other words, access points should not be used as a tool to apply speed limit regulation. Therefore, this constraint is excluded from the model.

The spacing should meet geometric spacing criterion (minimum spacing). This criterion is guaranteed by the specified length that should be long enough for vehicles to stop before reaching the next intersection when they pass the prior intersection at normal travel speed. Hence, we here use stop distance as the minimum spacing criterion, i.e.,

\[
\frac{L}{n_2} \geq 0.278(S_1)t_r + 0.139\frac{S_1^2}{a}
\]  

(5.8)

where,

- \( n_2, L, S_1 \) are prescribed,
- \( t_r \) represents response time in seconds,
- \( a \) represents deceleration rate that is a function of speed,
  
  \[ = \mu(S_1), \text{ km/h/sec}, \]
Furthermore, the number of access points must be a nonnegative integer. It is self-explanatory to specify the decision variable as an integer.

\[ n_2 \geq 0 \]

\( n_2 \) is integer \hspace{1cm} (5.9)

The objective function and some constraints are clearly nonlinear functions of the decision variables. Therefore, this is an integer nonlinear programming problem.

5.2 Solving Integer Nonlinear Programming

5.2.1 General Discussion

There are relatively few methods for obtaining integer solutions to nonlinear programming problems. Most textbooks confined their attention to either integer linear programs, or nonlinear programs, see, e.g., Wismer and Chattergy (1978), Zangwill (1969). The difficulty in considering nonlinear integer programming is that, as Simmons (1975) stated, "integer programming is too rich and too intricate a subject to be profitably treated as a species of mathematical programming, its current solution methods and recent research have little in common with those continuous-variable problems". One method that was suggested to solve the nonlinear integer problem is using dynamic programming, see, e.g., Cooper and Steinberg (1970), and Pfaffenberger and Walker (1976).
However, since there is only one decision variable in the current model, there is no advantage of applying dynamic programming. The advantage of dynamic programming is that it treats one subproblem with one variable each time for a problem with \( n \) variables. On the other hand, the difficulty of using dynamic programming technique is that the constraint is specified as linear function in this technique; while in our model it includes nonlinear functions in constraints. Therefore, even if the model is expanded to include several decision variables, e.g., number of business access points \( n_3 \), in the future, this method is still not applicable.

5.2.2 Piecewise Integer Programming

It was indicated that non-linear problems can sometimes be treated as piecewise integer programming problems with advantage (Cooper and Steinberg, 1974; and Williams, 1985). It is therefore pursued to see if the method can also be used for solving an integer nonlinear problem. The procedure is that we first try to solve the nonlinear problem using the method, while ignoring the integer requirement. Then we modify the procedure to include the integer requirement.

The prerequisite of piecewise integer programming is that the problem can be expressed in a separable programming form. The separable functions ensure that they can be approximated to by piecewise linear functions. An example of a piecewise linear approximation to \( f(x) \) is the function \( \hat{f}(x) \), illustrated with dashed lines in the sketch (Figure 5.1).
We can see that a piecewise linear function is a set of connected line segments. This is also referred to as a "polygonal line", and the approximation \( \hat{f}(x) \) as a "polygonal approximation". The widths of the intervals need not be equal.

![Diagram](image)

**Figure 5.1** A Polygonal Approximation of a Nonlinear function

(From Cooper and Steinberg, 1974)

We now try to approximate \( f(x) \) by three connected line segments as shown in Figure 5.1. At the endpoints of each subinterval \( \hat{f}(x) = f(x) \). Thus, in the interval \( a_1 \) and \( a_2 \)

\[
\frac{\hat{f}(x) - f(a_1)}{x - a_1} = \frac{f(a_2) - f(a_1)}{a_2 - a_1} \quad a_1 \leq x \leq a_2
\] (5.10)
Then

\[ f(x) = f(a_1) + \frac{f(a_2) - f(a_1)}{a_2 - a_1} (x - a_1) \quad a_1 \leq x \leq a_2 \]  

(5.11)

Assume a value \( \lambda \) in the range of 0 and 1, for \( x \) in the interval \( a_1 \) and \( a_2 \), we have an expression as

\[ x = \lambda a_1 + (1 - \lambda) a_2 \]  

(5.12)

If we denote this value of \( \lambda \) by \( \lambda_1 \), and let \( \lambda_2 = 1 - \lambda_1 \), then the above equation becomes

\[ x = \lambda_1 a_1 + \lambda_2 a_2 \]  

where,

\[ \lambda_1 + \lambda_2 = 1, \quad \lambda_1, \lambda_2 \geq 0 \]  

(5.13)

Subtracting \( a_1 \) from both sides of equation (5.13), we have

\[ x - a_1 = \lambda_1 a_1 + \lambda_2 a_2 - a_1 \]
\[ = (\lambda_1 - 1) a_1 + \lambda_2 a_2 \]
\[ = -\lambda_2 a_1 + \lambda_2 a_2 \]
\[ = \lambda_2 (a_2 - a_1) \]  

(5.14)

Then equation (5.11) becomes
\[ \hat{f}(x) = f(a_1) + \frac{f(a_2) - f(a_1)}{a_2 - a_1} \lambda_2(a_2 - a_1) \]
\[ = f(a_1) + \lambda_2 f(a_2) - \lambda_2 f(a_1) \]
\[ = f(a_1)(1 - \lambda_2) + \lambda_2 f(a_2) \]
\[ = \lambda_1 f(a_1) + \lambda_2 f(a_2) \]  
(5.15)

Similarly, we can get \( \hat{f}(x) \) for other two subintervals, \( a_2 \leq x \leq a_3 \), and \( a_3 \leq x \leq a_4 \):

\[
\begin{align*}
\hat{f}(x) &= \lambda_3 f(a_3) + \lambda_2 f(a_3), \quad a_2 \leq x \leq a_3 \\
&\quad x = \lambda_3 a_3 + \lambda_2 a_3 \\
&\quad \lambda_2 + \lambda_3 = 1,
\end{align*}
\]
(5.16)

\[
\begin{align*}
\hat{f}(x) &= \lambda_4 f(a_4) + \lambda_3 f(a_4), \quad a_3 \leq x \leq a_4 \\
&\quad x = \lambda_4 a_4 + \lambda_3 a_4 \\
&\quad \lambda_3 + \lambda_4 = 1,
\end{align*}
\]
(5.17)

All \( \lambda \geq 0. \)

To consider \( x \) in the whole range, we now have

\[
\begin{align*}
\hat{f}(x) &= \lambda_1 f(a_1) + \lambda_2 f(a_2) + \lambda_3 f(a_3) + \lambda_4 f(a_4), \quad a_1 \leq x \leq a_4 \\
&\quad x = \lambda_1 a_1 + \lambda_2 a_2 + \lambda_3 a_3 + \lambda_4 a_4 \\
&\quad \lambda_1 + \lambda_2 + \lambda_3 + \lambda_4 = 1
\end{align*}
\]
(5.18)

Again, all \( \lambda \geq 0. \)

Note that the function now becomes linear in the new variable \( \lambda_k \).

To ensure that \( x \) is only in the interval of adjacent \( a_i \), i.e., in the interval \([a_{i-1}, a_i]\) or \([a_i, a_{i+1}]\), we shall stipulate that at most only two adjacent \( \lambda_k \) can be positive and all others must be zero. This
stipulation also ensures equation (5.18) is equivalent to equations (5.15) to (5.17).

To incorporate this additional requirement on \( \lambda_k \) within an integer programming framework, new variables \( y_k, k = 1, 2, 3 \) are introduced in this case. Each \( y_k \) is constrained to be 0 or 1 as below

\[
\begin{align*}
\lambda_1 & \leq y_1 \\
\lambda_1 & \leq y_1 + y_2 \\
\lambda_2 & \leq y_2 + y_3 \\
\lambda_3 & \leq y_3 \\
\end{align*}
\]

\[
y_1 + y_2 + y_3 = 1, \\
y_k = 0 \text{ or } 1, \quad k = 1, 2, 3
\]

Equations (5.18) through (5.21) represent the polygonal approximation \( \hat{f}(x) \) of a nonlinear function \( f(x) \).

Up to now we excluded constraints from the discussion. If nonlinear functions also appear in the constraints the same transformation approach will be applied. That is, they can be replaced by linear terms and a piecewise linear approximation made to the nonlinear function. Suppose we have a problem with nonlinear constraints as

Minimize \( z = f(x) \)

Subject to \( g_i(x) \leq b_i \) \quad i = 1, \ldots, m

\( x \geq 0 \)
Then the general representation of this model for \( \hat{f}(x) \) with \( P \) connected line segments (i.e., for \( a_k, \ k=1,2, \ldots, P+1 \)) is:

Minimize \[ z = \sum_{k=1}^{P+1} \lambda_k f(a_k) \]

Subject to
\[
\begin{align*}
\sum_{k=i}^{P+1} \lambda_k g_i(a_k) &\leq b_i, \quad i = 1,...,m \\
\lambda &\leq y, \\
\lambda_k &\leq y_{k-1} + y_k, \quad k = 1,...,P \\
\lambda_{P+1} &\leq y_P, \\
\sum_{k=1}^{P+1} y_k & = 1 \\
\sum_{k=1}^{P+1} \lambda_k & = 1 \\
\lambda_k &\geq 0 \\
y_k & = 0 \text{ or } 1, \quad k = 1,...,P
\end{align*}
\]  

(5.23)

where

\[ \sum_{k=1}^{P+1} \lambda_k a_k = x \]  

(5.24)

Be aware that equation (5.24) is not used in the programming. Once the programming is solved, the optimal values of the \( \lambda_k \) may be inserted to (5.24) to obtain the optimal values of the original variable \( x \).

Up to now the nonlinear issue has been solved. Next, we are going to include integer requirement in the model. To do this we modify the model (5.23) by stipulating that only one element in \( \lambda_k \) be positive, precisely the value of 1, and all others be 0. This requirement ensures
that only one line segment will be selected. In fact, this is a logic constraint. \( \lambda_k \) is a set of ordered variables within which only one variable must be non-zero. It is called SOS1 set, means special ordered set of type 1 (Williams, 1985).

Furthermore, we stipulate that the end point of the line segment should be an integer, i.e., \( a_k \) be chosen as an integer as well. We can see that the optimal solution will be \( a_k \) if \( \lambda_k \) is determined to be 1 by the model. This is because \( f(a_k) \) is exactly equal to \( f(a_k) = f(x) \) when, \( x = a_k \). In this sense, we turn piecewise programming into "pointwise" programming. Ideally, \( a_k \) should be assigned as 1, 2, ..., m. If the value range of \( x \) is not very wide there is no problem; otherwise, it may result in enormous computation. Fortunately, the range of our decision variable is relatively narrow. Due to the geometric spacing constraint, the access density would not be very high. The maximum density may be just less than 10/km for a speed limit of about 40 or 50 km per hour. Then, for a road section of \( L \) km, the reasonable range of number of access points is not very wide. It is doable. However, for other problems with a wide range of decision variable, a wider interval might be selected first, e.g., 5, 10, 15, ...; then, after we find the optimal interval we can repeat the optimization procedure in this interval only to find the optimal integer value. This kind of strategy was also suggested by Hadley (1964).

In essence, the requirement on \( a_k \) is just one of the methods used for solving general integer programming problems: i.e., enumeration
method. This method enumerates all possible integer solutions. Therefore, equation (5.23) can be modified as

\[
\text{Minimize } z = \sum_{k=1}^{p+1} \lambda_k f(a_k)
\]

Subject to \[
\begin{align*}
\sum_{k=1}^{p+1} \lambda_k g_i(a_k) & \leq b_i, \quad i = 1, \ldots, m \\
\sum_{k=1}^{p+1} \lambda_k &= 1 \\
\lambda_k &= 0 \text{ or } 1, \\
a_k &= \text{integer}
\end{align*}
\]

where \[
\sum_{k=1}^{p+1} \lambda_k a_k = x
\]

Another key question is if the solution is global optimal or locally optimal. Since this procedure only produces an approximate function for analysis, it generally guarantees no more than a local optimum. Nevertheless, one strategy is to solve the problem a number of times using different \(a_k\) to obtain different local optima. There may be some chance to obtain a global optimum, or at least one close to the global optimum (Williams, 1985). However, it is indicated that only way to be sure of obtaining a global optimum when a problem is not known to be convex is to resort to integer programming (Cooper and Steinberg, 1970; Hadley, 1964; and Williams, 1985). Otherwise, if a problem is known convex a global optimum would be obtained.
5.3 Model Application

The issue of separable function does not exist when there is only one decision variable in the model. However, if we could include other decision variables, e.g., number of business access points, in the model, this approach is still applicable as long as we can separate the variables. This is most probably because it is presumed that there are no interactions between decision variables.

The verification of convexity of a function is sometimes tedious and cannot be easily solved. However, graphic representation of a function when there is only one variable (or may be two) can greatly reduce the computation. To do this we assume some input values for parameters and plot our objective function of equation (5.9) in figure 5.2. It can be seen clearly that the function is convex in this range. Since constraints are very complex functions, it is not pursued to prove that they are a convex set. Instead, the characteristics of decision variable (i.e., integer and value range) limit the range of feasible values as indicated earlier. Which ensures that a full enumeration approach is applicable and will guarantee a global optimal solution.
Substitution of equations (5.4), (5.5), (5.6), (5.7), and (5.8) into equation (5.25) yields

Minimize \[ z = \sum_{k=1}^{P+1} \lambda_k f(a_k) \]

Subject to \[
\sum_{k=1}^{P+1} \lambda_k f_{scd}(a_k) \leq ACD \tag{5.27}
\]
\[
\sum_{k=1}^{P+1} \lambda_k f_s(a_k) \leq I,
\]
\[
f_{spd} \geq S,
\]
\[
\sum_{k=1}^{P+1} \lambda_k g_s(a_k) \leq L,
\]
\[
\sum_{k=1}^{P+1} \lambda_k = 1
\]
\[
\lambda_k = 0 \text{ or } 1,
\]
\[
a_k = 1, \ldots, 10
\]

where
\[
f(a_k) = c_1 f_{acd}(.)L + c_2 \left( \frac{L}{f_{spd}(.)} - \frac{L}{f_0(.)} \right) A_q + c_3 f_{dly}(.) A_q \tag{5.28}
\]

\[
(f_{acd} \text{ and } f_{spd} \text{ are obtained from chapter 3 and 4 respectively.})
\]

\[
f_s(a_k) = \frac{1}{C_{sh} - q/n_2} \tag{5.29}
\]

\[
(C_{sh} \text{ is capacity of the shared lane as defined in Appendix B.})
\]

\[
g_i(a_k) = \left\{ 0.278 S_i t_r + 0.139 \frac{S_i^2}{a} \right\} n \tag{5.30}
\]

\[
n = \sum_{k=1}^{P+1} \lambda_k a_k \tag{5.31}
\]

It makes common sense that a graphing method will most often provide the most efficient solution for solving mathematical programming problems when decision variables are less than three. Since there is only one variable retained in our model at this stage, the easiest way of obtaining optimal solution is graphing the model. This was attempted and the results obtained from piecewise programming are identical to the graphing method. For example, as shown earlier, figure 5.2 indicates that the optimal value is 3, while the same result was achieved from analytical procedure.

We found that the RHS coefficient (right hand side value) of all inequality constraints are inactive at their upper bounds. Which means the optimal solution will be valid for any very large values of accident rates, delay time, and very small value of stop distance (since this value is in the denominator). However, the lower bounds are all active.
Which means if we specify a very low accident rate and short waiting time the optimal solution will be changed. Generally, this agrees with our expectation.

An important aspect of optimization is dealing with uncertainty. A brief discussion only will be presented here. Basically, three types of uncertainty could be involved in mathematical programming. The first type makes parameters random with known probability distributions. The second type makes RHS coefficients of constraints random with known probability distribution. The third type makes goals and constraints nondeterministic. The first and the third types of uncertainty are not significant in our model. In transportation planning, cost coefficients are relatively stable because time value and unit accident cost are well established. Some references were found, e.g., Bruzelius (1979) for time cost, and Miller et al (1991) for accident cost. Objective and constraint functions are deterministic rather than random for they could be sharply defined in our model. Regarding type two uncertainty, we observed that the RHS coefficient for minor traffic delay time at access points could show some degree of variation. Thus, if we know the distribution of waiting time of minor traffic at access points, we may employ chance constraint programming to transform the original constraint to conventional linearized form. However, sensitivity analysis indicates that our model is quite robust. For the constraint of concern, the optimal solution is valid for RHS coefficient in the range of about 10 seconds to infinity. The lower bound reaches the level of service B for signalized intersection delay. This range appears to be quite satisfactory, and supports the validity of the model.
To illustrate the application of the model, the computation was made with variations of principal input values, main road and side road daily traffic volumes. Other parameters were assigned fixed mean values. The results are shown in Table 5.1. Next, we may compare this with the current practice. Table 5.2 shows some two-lane highway sections of length about four kilometres in the database. It can be seen that the optimal number of unsignalized intersections determined by the model is somewhat in the practical range. However, due to the lack of minor road traffic information it cannot be determined explicitly whether the existing condition is optimal or not.

There are limitations of the model. First, other types of access points, for example, business access, have not been included, as discussed earlier. Second, accident cost on minor roads have not been considered. Limitation on the number of access points to main road will influence traffic on minor roads. This can be explained in that the increase traffic volume and relatively slow travel speed might have a positive impact on increasing accidents. Therefore, the other aspect of improving the model is to take accidents occurred on minor roads into consideration.
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<td>3</td>
</tr>
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<td>6000</td>
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Table 5.1 Optimal Number of Intersections on a 4-km Section of Two-lane Highway
<table>
<thead>
<tr>
<th>Section Length (km)</th>
<th>Main Road Traffic (AADT)</th>
<th>No. of Unsignalized Intersections</th>
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<td>3.9</td>
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<td>3</td>
</tr>
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</table>

Table 5.2 Actual Number of Unsignalized Intersections for Some Two-lane Highway Sections of Length about 4-km

(Source: Photolog database)
In chapter 5, we developed a model to determine the optimal number of access points on a road section. In future design, we may use the model to plan access density for any two-lane road sections based on traffic volume, speed, safety and delay considerations. Once an access density is decided upon, the operational design associated with access density can be initiated. One important part is design for speed. The design for speed ensures safe and efficient traffic flow on a road section, with different speeds should be specified for sections with different access densities. Speed zone design is widely practiced for optimum road performance and safety as given by ITE (1976), Parker (1985). Usually, the 85-percentile speed as determined by spot speed studies is the principal factor used by traffic engineers to determine speed limits for a speed zone. However, access is an important factor to be considered and not well documented, although one reference published in Traffic Engineering (1961) outlined the number of access points for roadside businesses (B) per unit distance as the density index to define the speed zone: 90 km/h for three B per kilometre, 80 km/h for 4 B per km, and 70 km/h for 5 B per kilometre. Furthermore, we could also make use of some results obtained in chapter 3 (i.e., figures 3.7, 3.8, 3.10, and 3.12). For example, we can check the impact of business access density on the estimated accident rate for each speed limit and we may then select a suitable speed limit on the basis of a pre-specified acceptable accident rate.
In addition to the need to develop design guidelines for speed zones based on access density it is generally the case that some aspects of speed zone design, for instance speed transition and speed sign design, have not yet been clarified, particularly as a means to mitigate existing high levels of access density. Therefore, in this chapter, section one deals with speed transition zones, and particularly the minimum length of speed transition zone. Section two concentrates on speed-change sign design. Section three outlines a procedure of speed transition zone design.

6.1 Speed Transition Zone

The review of some studies (e.g., Parker, 1985) indicated that the principal guideline used in designing transition zones (also called "buffer zones") is based on the Uniform Vehicle Code's (National Committee of Uniform Traffic Laws and Ordinances, 1968, and 1979) recommendation that no more than six alterations per mile be used with not more than 10 mph (16 km/h) differences between zones. However, it does not present the criteria to determine under what condition that a transition zone is needed and what is the minimum length of a transition zone.

Three principles are proposed here for transition zone setup: safety, efficiency, and convenience. Safety: all road users' safety, drivers and pedestrians, must be considered. Efficiency, roadway speed potential (higher speed limit) should be fully used to reduce travel time. Convenience, driving process should be performed without excessive
control limits.

According to the above principles, two general criteria are suggested here. First, a transition zone should be provided when the roadway section is a transitional stretch of highway between low access density section and high access density section, which represents the first principle. This statement is explained graphically in Figure 6.1.

The second criterion of setting up transition zone is that the difference between speed limits of adjoining road sections should be \( \geq 40 \text{ km/h} \). This is based on the recommendation by National Committee of Uniform Traffic Laws and Ordinances (1968 and 1979) that speed change between zones is no more than 10 mph (16 km/h, rounded to 20 km/h). It represents above principle two and three (efficiency and convenience). This means that (1) if a higher speed is feasible in terms of a safety concern, keep the speed as high as possible; and (2) do not set up too many signs and limits on the road. A particular case is when the difference is 30 km/h, where generally, it is not necessary to insert a transition zone in between adjoining sections. However, engineering judgment is also needed to evaluate any practical situation. On the other hand, probably it is better to reset the speed limits to get the difference as 20 km/h, or 40 km/h.
To determine the minimum length of the transition zone, first we should examine the length of transition section of access density, $D_b$, as indicated in Figure 6.2. Generally, the summation of the minimum length $L_t$ and deceleration distances, $D_d'$ and $D_d''$ (the two deceleration distances may not be equal), should be equal to or greater than $D_b$.

Second, we will consider the length of "speed maintaining distance", which is the minimum length of roadway that drivers do not have to change speed, i.e., they can travel at a constant speed for a certain distance. This distance could be different at different speed as proposed in Reference (Traffic Engineering, 1961): varying from 0.2 mile (0.322 km) to 0.5 mile (0.805 km) for a speed range from 20 mph (32 km/h) to 70 mph (112 km/h), see Table 6.1. It may be more convenient to adopt a "relieving time" defined as a time period that relieves drivers from the change of traffic control. They are free from making "forced" changes in speed. As shown in Table 6.1, the travel time for different minimum speed zone length varies between 24 seconds and 36 seconds. On the basis of this information, a period of 30 seconds is suggested here.
as the "relieving time".

Table 6.1 Minimum Zone Length and Corresponding Travel Time

<table>
<thead>
<tr>
<th>Speed (mph/km/h)</th>
<th>Min. Length of Zone (miles/km)</th>
<th>Corresponding Travel Time (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20/32.2</td>
<td>0.2/0.322</td>
<td>36</td>
</tr>
<tr>
<td>30/48.3</td>
<td>0.2/0.322</td>
<td>24</td>
</tr>
<tr>
<td>40/64.4</td>
<td>0.3/0.483</td>
<td>27</td>
</tr>
<tr>
<td>50/80.5</td>
<td>0.5/0.805</td>
<td>36</td>
</tr>
<tr>
<td>60/96.6</td>
<td>0.5/0.805</td>
<td>30</td>
</tr>
<tr>
<td>70/112.7</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Source: Traffic Engineering, 1961

We can then calculate the minimum transition zone length for each transition speed by using the following equation.
\[ d = 0.278 \mu t + 0.139at^2 \] (6.1)

where, 
- \( d \) = minimum transition zone length, \( L_t \), in metres
- \( \mu \) = transition zone speed in km/h
- \( a \) = acceleration or deceleration rate in km/h/sec
- \( t \) = "relieving time" in seconds

The acceleration rate is zero in this case. The results are shown in Table 6.2.

<table>
<thead>
<tr>
<th>Transition Zone Speed (km/h)</th>
<th>Minimum Zone Length, ( L_t ), (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>830</td>
</tr>
<tr>
<td>90</td>
<td>750</td>
</tr>
<tr>
<td>80</td>
<td>670</td>
</tr>
<tr>
<td>70</td>
<td>580</td>
</tr>
<tr>
<td>60</td>
<td>500</td>
</tr>
</tbody>
</table>

Table 6.2 Minimum Transition Zone Length

6.2 Location of R-3 Sign

"R-3 MAXIMUM (Speed) ... AHEAD" sign may be used to give advance notice of a speed zone with a lower limit. The manual of Standard Traffic Signs (B.C. Ministry of Transportation and Highways, Traffic Branch, 1988) states that "the advance signs shall be erected from 350
to 750 feet ahead of the transition point". However, it does not give sufficient detail to select a specific value. The following analysis applies kinematic principles to calculate appropriate advance placement distance for the R-3 sign.

The basic equation used to calculate distance traveled given deceleration rates and elapsed time is

\[ d = 0.278 \mu_b t_r + 0.139 \frac{\mu_b^2 - \mu_e^2}{a} \]  

(6.2)

where, \( d \) = travel distance
\( t_r \) = response time in sec
\( \mu_b \) = initial speed in km/h
\( \mu_e \) = final speed in km/h
\( a \) = deceleration rate

Drivers' response time to the Speed Limit sign has not been found in literature and an average value of 1.5 second is then assumed for \( t_r \), which is the value of brake reaction time of 90 percent of drivers (ITE, 1982). We define the response time as the period from the time that drivers pass by the sign to the time that drivers take action to slow down. We assume that detecting, reading, and comprehending the sign will occur before drivers passing the sign, and is neglected in this analysis. Figure 6.3 illustrates the parameters for locating the warning sign. Where, R-4 sign indicates the effective speed limit.

In determining proper deceleration rates, various references have
been searched. Since very little information is available we assume that drivers do not respond to the R-3 sign by applying brakes when roadway is level or uphill. Coast-down operation is expected for deceleration process. Some research indicates that deceleration rates without brakes are much greater at high speeds because the resistance to motion, particularly air resistance, is greater (ITE, 1982). For example, at speed of 70 mph (113 km/h), the deceleration rate is about 3.5 km/h/s; at speed of 50 mph (80.5 km/h), the deceleration rate is 1.7 mph/s (2.74 km/h/s).

The relation between deceleration rates and speeds is a polynomial function of speed and elapsed time in the coast-down operation as proposed in Lucas (1986):

\[
\mu = a_0 + a_1 t + a_2 t^2 + a_3 t^3 + a_4 t^4 + a_5 t^5 + a_6 t^6
\] (6.3)
A curve fitting technique is used to estimate the coefficients $a_i$. Thus the expression is readily differentiated to give the deceleration

$$\frac{d\mu}{dt} = a_1 + 2a_2t + 3a_3t^2 + 4a_4t^3 + 5a_5t^4 + 6a_6t^5$$

(6.4)

and may be evaluated at a number of discrete speeds throughout the vehicle speed range. The results of the evaluation for selected speeds are shown in Figure 6.4.

In addition to consider grade we assume that if grade is greater than or equal to $-1\%$, no breaks are applied; if grade is less than that, a minor factor, $0.2$ km/h/s, will be added to the deceleration rate, $a$, for each increment of one percent downgrade slope. The implied assumption is that reasonable breaking is expected when roadway is downhill. Equation (6.2) can then be modified as

![Figure 6.4. Deceleration-Speed Relation](image-url)
\[ d = 0.278 \mu_b t, + 0.139 \frac{\mu_b^2 - \mu_e^2}{a + gG} \] (6.5)

where, \( g \) = gradient in percent
\( G \) = gravity factor

Table 6.3 summarizes the distance traveled between two speed limits. The values can be used as the suggested distance between R-3 and R-4 (see Figure 6.3).

<table>
<thead>
<tr>
<th>( \mu_b ) **</th>
<th>( \mu_e ) ***</th>
<th>Gradient (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>110</td>
<td>90</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>190</td>
</tr>
<tr>
<td>100</td>
<td>80</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>180</td>
</tr>
<tr>
<td>90</td>
<td>70</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>160</td>
</tr>
<tr>
<td>80</td>
<td>60</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>150</td>
</tr>
<tr>
<td>70</td>
<td>50</td>
<td>100</td>
</tr>
</tbody>
</table>

Notes:
* The value is in metres.
** \( \mu_b \) = Initial Speed Limit (km/h)
*** \( \mu_e \) = Final Speed Limit (km/h)

Table 6.3 Distance Traveled Between Two Speed Limits*

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6.3 Outline of Transition Zone Design Procedure

In conclusion, on the basis of above discussions figure 6.5 presents a general procedure to set up a speed transition zone for sections with higher than critical access density, and explained below.

(1) Collect roadway characteristics and existing speed limit information.

(2) Review accident records with respect to frequency, severity, type, and cause. Particular attention should be given to those accidents in which unreasonable speed appears to have been a causative factor. If such a factor exists, set up a speed transition zone, and go to (7).

(3) Review road-side development and traffic condition. Specifically, this includes access density; parking, loading and other vehicle operations adjacent to travel lanes; turn movements and controls. If there is a transitional stretch of those activities between two adjoining speed zones, set up a speed transition zone. Otherwise, reduce speed directly to lower limit and go to step (9).

(4) Check the difference between two speed limits. Generally, if the difference is equal to or greater than 40 km/h, set up a speed transition zone, go to (7). If the difference is less than 40 km/h, go to (5).
Road Characteristics and Speed Limits

Yes (2) Review Accident Record

No

No

Yes

Review Access Density

Yes

Speed Difference $\geq 40 \text{ km/h}$ ?

No

No

Yes

Speed Difference $= 30 \text{ km/h}$ ?

Yes

Engineering Judgment

No

Determine Transition Zone Speed

Determine Transition Zone Length, $L_t$

Determine Placement Distance of R-3, $L_d$

Figure 6.5 Transition Zone Design Procedure to Mitigate for High Access Density
(5) If the speed difference is equal to 30 km/h, go to (6), if not, go to (9) for direction speed reduction (no transition zone).

(6) When the difference is equal to 30 km/h, the transition zone may not be necessary, but engineering judgment is needed to evaluate the practical situation. If a transition zone is warranted, go to (7), otherwise, go to (9).

(7) Determine transition zone speed. This speed should be set at the median value of two speed limits, and rounded to nearest 10 km/h.

(8) Determine transition zone length. Check minimum transition zone length suggested in Table 6.2.

(9) Determine the advance placement distance of R-3 sign by using Table 6.3.

Finally, figure 6.6 illustrates speed transition zone setup by using speed and distance diagram.

*If Dec Length Ld1 > 300 m, a second R-3 sign should be erected.

Figure 6.6 Speed Transition Zone Sketch
Chapter 7

IMPLICATIONS AND CONCLUSIONS

A classical concern in both traffic engineering and traffic planning has been the safety of intersections, the efficient operation of highways at intersections, and the tradeoffs required in planning road access.

The contribution of the research reported on here is the systematic study of road access, and to suggest practical applications of the result of the findings related to two main components of highway access: those of main road safety and delay. Besides statistical studies of the relationships between highway access, safety and travel speed, the research has developed an approach based on Bayesian analysis for a more complete understanding of these relationships. The hazard model developed in Section 3.4 holds promise not only for a more precise understanding of access safety problems but also for new definitions of traffic safety in general. The Bayesian updating approach developed here is one possible mean of improving our understanding of road safety.

In addition there has been to date no systematic definition of highway classification that could be tied to safety criteria. This research extends highway classification criteria to include relative safety levels based on the number of intersections, or other access types per kilometre. Similarly, highway capacity concepts have not, in
the past, included the friction due to access as an influence on levels of service, but conversely has treated capacity as a sub-optional process for highway (or street) sections between intersections, or conversely for intersections individually (except freeway terminals). This research suggests, by implication, the need to incorporate speed reduction due to access as a delay factor in levels of service.

Finally it is clear that much work must be accomplished to comprehensively plan highway systems to account for the tradeoff's between road way benefits and the respective benefits and costs to peripheral activities and to non-users who are affected by highways. This research, suggests a framework which, it is proposed could be used for the optimization of access points to minimize delay cost to all users. Further study is needed, along with adequate data, to bring in the benefits and costs. However, this research points to an analytical procedure based on classical optimization techniques which, it is felt, holds promise for future research on this subject.

7.1 Implication of the Research for Highway Classification

Functional hierarchy of a highway system is an important element in highway planning. One understands that the higher class of a road the less access points should be present along the road. This understanding is illustrated by Stover and Koepke in Figure 7.1. However, this
understanding of the relationship between access points and functional classification is basically conceptual. The dotted line represents a general trend only. It does not imply a determined quantitative relationship.

Figure 7.1 Access and Functional Classification

(Adapted from Stover and Koepke, 1988)

With the framework obtained in Chapter 5, it may be possible to begin to quantify the access provisions for each class of road. For example, using specified daily traffic volume in a functional classification system for a certain highway class (see, e.g., Highway Functional Classification Study, 1992) as an input, with other given or prespecified information such as speed limit, one can derive appropriate access points (unsignalized intersections) for a road section in that highway class. One such example is given in Table 5.1. With given traffic input and characteristics of a road section, which represent
the features of a specified class for the road section, the appropriate (optimal) access points can be determined. For example, in planning a four kilometre arterial road section, if main road daily traffic is 10,000 vehicles and side road daily traffic is less than 1,500 vehicles, the appropriate number of unsignalized intersections is only one intersection (see Table 5.1). If the daily traffic on side road is 4,000, then two unsignalized intersections should be planned for this arterial road section.

The appropriateness of the access provisions is validated by the fact that it is derived on the basis of minimizing the total cost. Although the minimum cost is not stated as an objective in a highway classification system, it is, however, an implied objective. As stated in Highway Functional Classification Study (1992), one of the objectives of the functional classification system is to provide "safe, efficient and economical operation for all highway users".

7.2 Implication of the Research for Highway Level of Service

In determining the level of service of a two-lane highway, some adjustment factors are considered in the Highway Capacity Manual for directional distribution, narrow lanes and restricted shoulder width, the operational effects of grades on passenger cars, and the presence of heavy vehicles in the upgrade traffic stream. However, there is no consideration at all for access points. The impact of access on traffic operation efficiency for two lane highways can be estimated using the results obtained in Chapter 4.
In Chapter 4, a quantitative relationship between access and traffic speed is developed. This relationship provides a basis to evaluate the level of effectiveness (service) of two-lane highways. Although percent time delay (%) is the primary measure of level of effectiveness for two lane highways, average travel speed is a secondary measure of level of effectiveness and is clearly specified in level-of-service criteria (see Highway Capacity Manual).

Messer (1983) summarized the specifications of the levels of service in three ways: speed-volume, percent time delayed-volume, and maximum service volumes. Levels of service are assigned in each case. Table 7.1 shows the classification of level of service corresponding to speed and volume. These relationships are for traffic in a one hour period and over flat terrain for ideal traffic conditions.

<table>
<thead>
<tr>
<th>Two-Way Volume, pcph</th>
<th>Average Speed, km/h*</th>
<th>Level of Service Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 400</td>
<td>≥ 92.52</td>
<td>A</td>
</tr>
<tr>
<td>≤ 700</td>
<td>≥ 88.5</td>
<td>B</td>
</tr>
<tr>
<td>≤ 1,100</td>
<td>≥ 84.47</td>
<td>C</td>
</tr>
<tr>
<td>≤ 1,700</td>
<td>≥ 80.45</td>
<td>D</td>
</tr>
<tr>
<td>≤ 2,800</td>
<td>≥ 72.41</td>
<td>E</td>
</tr>
</tbody>
</table>

* The conversion was made from the unit of miles per hour to kilometres per hour.

Source: Messer, 1983

Table 7.1 Level of Service Classification
Based on the above relationship between speed and level of service and the relationship between access density and speed that is determined in Chapter 4, the effect of access density on level of service can be easily obtained. From Chapter 4, we have found that one unsignalized intersection per kilometre will reduce travel speed by 1.6 km/h. It is assumed that there are no access points (unsignalized intersections) for ideal traffic conditions on a road section, and this can be taken as a base level for access so that level of service A should be no access points. As access density increases, the level of service will deteriorate. With the speed index shown in Table 7.1, we have the following table (Table 7.2) giving access density for each level of service below A.

<table>
<thead>
<tr>
<th>Access Density* (No. per km)</th>
<th>0</th>
<th>1-2</th>
<th>3-5</th>
<th>6-7</th>
<th>&gt;8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level of Service</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
</tr>
</tbody>
</table>

* Access density represents unsignalized intersection density.

Note: The traffic volume range for this table is 50 - 2500 veh/h.

Table 7.2 The Effect of Access Density on Level of Service

Note that other types of access points, such as business accesses, can be converted into unsignalized intersections in terms of their effects on travel speed. As indicated in Chapter 4, one unsignalized intersection is equivalent to two business accesses, and four private
accesses respectively. Thus level of service may be calculated for any combination of access types and roadway capacity (level of service 'E') be defined in terms of access density in combination with the well recognized factors influencing roadway capacity.

In both the functional classification system and traffic efficiency measures, traffic safety criterion is not directly included. For instance, in measuring level-of-service of highways only operation indexes such as speed, density, flow are considered. However, it is believed that the inclusion of safety criterion may be necessary in future. Safety is surely a part of service level of a highway. Excluding safety gives an incomplete picture of real "service level" of highways. Some further work is necessary to develop a new measure of "level of service" to include not only traffic operation measures but also safety criterion. Here, this concern is first raised as a result of reviewing the implications of present traffic safety and operation studies.

7.3 Summary Conclusions and Applications of Research

Two main functions of a highway are to serve through traffic flow on a regional basis and to provide accessibility to the local area. The desire for free flow, high speed and safety for through traffic usually conflicts with the basic needs of accessibility of local traffic. Functional classification of highways provides an appropriate way to tackle the problem. Full access control is the most effective method for serving through traffic, but is limited in usage. Partial access control is most widely applied. Therefore, to provide an appropriate level of
access control it is necessary to define the relationship between accesses and traffic safety and traffic operations and find the optimal access points for a road section. This research has generated insight into some refinements for functional road classification.

In this thesis we have created a data base of highway access, accident, and traffic speed for the province of British Columbia. The main use of the data base is to estimate the impact of accesses on highway safety and operations. The inclusion of access information is not available in any other data base in the province. Therefore, it can be used in some access related safety and traffic operation studies.

Empirical models of the relationship between access and accidents were developed. The preliminary accident analyses indicate that a simple correlation between accident rate and access density does not reflect the actual accident experience. The change in the structure and complexity of such relationships established decades ago may be due to temporal and/or spatial differences in the characteristics of the highway system, and this study demonstrates that the impact of access on accidents is still very significant. There also seems to be a significant interaction between the access density and some road characteristics, such as roadway horizontal curvature and speed limit. More specifically, the following observations can be derived from the results of the accident analyses:

1. Access points are significantly correlated with the occurrence of accidents for all highway classes. As expected, signalized and
unsignalized intersections appear to have the most important effects on all accident measures (e.g., one unsignalized intersection is equivalent to ten private accesses for two-lane rural highway).

2. For two-lane rural highways, speed limit has a major impact on the relationship between business access density and accident rate. The impact of business access on accident rate seems to intensify as the speed limit increases. The relative impact of business access on accident rate was calculated as one half of that of unsignalized intersections.

3. For two-lane rural highways, grade, horizontal curvature, and the frequency of change in curve direction contribute positively to accident occurrence, whereas traverse slope is negatively correlated with accidents. Furthermore, the presence of an auxiliary lane has a significant impact on the severity of accidents, perhaps because of overtaking at high speed.

4. For two-lane rural highways, horizontal curvature (measured by the average degree of curve) has a major impact on the relationship between accident occurrence and private access and roadside pullout densities. The impact of private access and roadside pullout on accident rate appears to increase as the average degree of curve increases.

5. In case of incomplete information about traffic volume for a road
section (especially in the lack of the access traffic volume data), accident frequency measures, both all-accident and severe-accident-only, are generally superior to accident rate measures. This is because we can assure that the measure of the dependent variable is correctly represented.

6. In general, it seems that the use of all types of accidents in the analysis, as opposed to using fatal and injury accidents only, result in statistically better prediction models. This is due to the inclusion of more accident information in the study road sections.

Furthermore, a conceptual hazard model was proposed to improve the understanding of highway safety. The model attempted to associate traffic conflicts with accident records by using Empirical Bayes methods. This is an information gain process. The information on safety should be obtained from not only accident records but also other sources, e.g., traffic conflict which contains abundant hazardous potential information. The results of the model show it is a promising method to estimate hazards. Therefore, it may be applied in access-safety studies.

In traffic operation analyses, the quantitative relationship between access and travel speed was developed. Although no average travel speed data was available, it was indicated that the test vehicle's travel speed could be used as a surrogate of average travel speed if the observations of the test vehicle's speed are reasonably
large. It was shown that unsignalized intersection, business access, and private driveway are significant in the speed model. Nonlinear relationships exist between these accesses and travel speed. The higher the access density, the more speed reduction would be induced. As intuitively expected, the magnitude of the influence on speed is in the order of intersection, business access, and private driveway, with a ratio of approximately 1:2:4. If all other independent variables remain at mean values, one unsignalized intersection will approximately reduce travel speed by 1.6 km/h, one business access will reduce the speed by 0.8 km/h, and one private driveway will reduce the speed by 0.4 km/h. However, Table 4.4 provides a more precise description of the effect of access on speed reduction.

The possible use of the findings of this study is in the estimation of the user costs. With the change of average travel speed, as a result of change in access density, the delay costs could be estimated. The other important application is in the evaluation of level of service for two-lane highways. The effect of access density on level of service is derived in Table 7.2. This finding enriches the understanding of the effectiveness of traffic operations on two-lane highways.

On the basis of the relationships between accesses and traffic safety and traffic speed derived in Chapters 3 and 4, an optimization model was developed to minimize the social costs. From a planning point of view, the optimization model is to provide a procedure to determine appropriate number of access points of a road section. In the model,
several considerations, such as permissible traffic delay, are incorporated into the constraints. The generalized model is an integer nonlinear function. A modified piecewise integer programming technique was developed to solve the integer nonlinear programming problem. The results of the model generally agree with the expectation. Therefore, the model could provide useful suggestions or guidelines on highway access planning. For example, given daily traffic volume on main road, daily traffic volume of side roads, speed limits for main and side road, and some geometric factors as prescribed in 5.1.3, an optimal number of unsignalized intersections can be derived using the model for a certain length of two-lane road section.

In the implementation of highway functional classification, the optimization model will provide guidelines in access provisions for each highway class (although only two-lane highways are considered at the present).

Access points in terms of access density are also an important factor in transportation design. It should be taken as a criterion in speed zone design (see Figure 6.1). Some related issues were discussed in Chapter 6. Specifically, the design criteria of speed transition zone are proposed as: first, a transition zone should be provided when the roadway section is a transitional stretch of highway between low access density section and high access density section, and second, the difference between speed limits of adjoining road sections should be \( \geq 40 \) km/h. The minimum length of speed transition zone is derived and shown in Table 6-2. The location of reducing speed warning sign is
studied. Finally, the design procedure of speed transition zone is proposed in Figure 6.5.

7.4 Future Research

There are several aspects that the thesis may be extended. The use of the travel speed of the test vehicle in traffic operation analysis is due to practical limitation of resources. Although it seems to be acceptable as explained in Chapter 4, the real data of average speed may need to be collected once resources are available to redefine the relationship between access and average travel speed. On the other hand, this will also verify whether the use of test vehicle speed in traffic operation analysis is practical. If it is so, the method employed in the thesis can be taken in other similar situations.

The optimization model for determining number of access points is a useful tool in highway planning and it may be expanded in several ways: (1) include other types of accesses, such as business access, as decision variables; (2) define land use cost as a function of number of access points, and incorporate this cost into the model; (3) consider environmental cost resulted from traffic delay.

The hazard model proposed in Chapter 3 points out a new direction in measuring road safety, particularly at access points. It needs to be finalized when more traffic conflict data are available.
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APPENDIX A

Average Travel Distance on parallel Minor Road

Several assumptions were made to determine the average travel distance on parallel minor road:

(1) Drivers will take advantage of higher speed limit on main road whenever they can.

(2) The origins of travellers are distributed evenly along the minor road.

(3) The characteristics of adjacent road sections are homogeneous with the one under study.

(4) The access points are parted at the same distance.

The first assumption ensures that drivers will turn on to the main road at the first access point they pass. The second assumption leads the conclusion that the average travel distance on the minor road to the closest access point will be one fourth of the length between two adjacent access points, see sketch below.

\[
\begin{align*}
\text{access 1} & \quad \quad \quad \quad \quad \text{access 2} \\
\leftarrow 1/4 & \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad 1/4 \\
1/2 & \quad \quad \quad \quad \quad \quad \quad \quad \quad 1/2 \\
\end{align*}
\]

the average travel distance to the closest access point = \(1/4\)

The third and fourth assumptions specify the location of access points.
in such a way as shown in the following sketch

```
section length = L

adjacent section

adjacent section
```

Apparently, if the number of access points in the section is n then the distance between adjacent access points is L/n. While the assumption two holds, the average travel distance is L/4n.
APPENDIX B

Minor Traffic Delay at Access Points and on Minor Road

Specifically, $f_a$ is prescribed as follows. According to the ITE handbook (1982), the average delay can be expressed as

$$f_a = \frac{1}{(a-b)} \quad (B-1)$$

where,

- $a$ = service rate (reserve capacity)
- $b$ = side-street arrival rate

Let the total minor traffic arrival rate on the one side in a road section be $q$, the arrival rate at an individual access point then is $q/n_2$, assuming that the traffic is evenly distributed in the section. This gives $b = q/n_2$.

Let $a = C_{SH}$, the capacity of shared lane. From the Highway Capacity Manual (1985),

$$C_{SH} = \frac{v_l + v_t + v_r}{[v_l/c_{mt}]+[v_t/c_{mt}]+[v_r/c_{mt}]} \quad (B-2)$$

where,

- $v_l$ = volume of flow rate of left-turn movement in shared lane, in pcph;
- $v_t$ = volume of flow rate of through movement in shared lane, in pcph;
- $v_r$ = volume of flow rate of right-turn movement in shared lane, in pcph;
\[ C_{r1} = \text{movement capacity of the left-turn movement in shared lane,} \]
\[ \quad \text{in pcph;} \]
\[ C_{mt} = \text{movement capacity of the left-turn movement in shared lane,} \]
\[ \quad \text{in pcph;} \]
\[ C_{mr} = \text{movement capacity of the left-turn movement in shared lane,} \]
\[ \quad \text{in pcph;} \]

Furthermore, we assume that the percentage of right turn traffic from side road is \( \alpha_r \), the percentage of through traffic is \( \alpha_t \), and the percentage of left turn traffic is \( \alpha_l \). Therefore, the right turn minor traffic, through minor traffic, and left turn minor traffic at an access point on one side of the main road are
\[ v_r = \alpha_r \frac{q}{n_2}, \quad v_t = \alpha_t \frac{q}{n_2}, \quad \text{and} \]
\[ v_l = \alpha_l \frac{q}{n_2} \]
respectively.

The potential movement capacity in passenger cars per hour as illustrated in the Highway Capacity Manual (1985) is based on the conflicting traffic volume, \( V_C \), in vehicles per hour, and the critical gap, \( T_C \), in seconds. This capacity is given in Figure 10-3 in the Highway Capacity Manual. To enter the optimization model, it is required to convert the relationship into an equation. Therefore, the curve was digitized taking the average critical gap = 6 second,

\[ C_{mi} = \exp(6.968 - 0.00135 \times \text{conflict traffic}) \]  \hspace{1cm} (B-3)

where,
\[ C_{mi} = \text{movement capacity.} \]

For right turn movement of minor traffic, the conflict traffic is the
through traffic on one direction of the main road, which is Q/2. For
through movement of minor traffic, the conflict traffic is the through
traffic on both directions of the main road, which is Q. For left turn
movement of minor traffic, the conflict traffic is the main road through
traffic plus through and right turn movements of minor traffic from the
opposite side of the main road, which is $Q + (\alpha_r + \alpha_t)q/n_2$.

Furthermore, it is indicated in HCM (1985) that "when traffic
becomes congested in a high-priority movement, it can impede lower
priority movements from utilizing gaps in the traffic stream, and reduce
the potential capacity of the movement." Therefore, impedance factor $P$
was proposed to modify the potential capacity of a movement. However,
right turns are usually not affected. To simplify the problem, average
values of $P$ were chosen as 0.68 for left turns from minor road and 0.8
for through traffic from minor road. The values are derived based on the
assumption that average capacities used by existing demand are 40
percent for left turns and 30 percent for through traffic respectively.

The equation (B-3) can be specifically expressed as

$$C_m = \exp \left[ 6.968 - 0.00135 \times \left( \frac{Q}{2} \right) \right]$$

$$C_{mr} = P_1 \exp \left[ 6.968 - 0.00135 \times (Q) \right]$$

$$C_{mi} = P_i \exp \left\{ 6.968 - 0.00135 \times \left[ Q + (\alpha_r + \alpha_t) \frac{q}{n_2} \right] \right\}$$

(B-4)

In summary, the average delay at an individual access point
\[ f_a = \frac{1}{C_{3H} - q/n_2} \]  \hspace{1cm} (B-5)

where,

\[ C_{3H} = \frac{q/n_2}{\exp[6.968 - 0.00135(Q/2)] + \frac{\alpha_r q/n_2}{\exp[6.968 - 0.00135(Q)]} + \frac{\alpha_s q/n_2}{\exp[6.968 - 0.00135(Q + (\alpha_r + \alpha_s)q/n_2)]} + \frac{1}{\exp[6.968 - 0.00135(Q/2)] + \frac{\alpha_r q/n_2}{\exp[6.968 - 0.00135(Q)]} + \frac{\alpha_s q/n_2}{\exp[6.968 - 0.00135(Q + (\alpha_r + \alpha_s)q/n_2)]}} \]