TRANSPORT CANADA TRANSPORTS CANADA

SYNTHESIS OF SAFETY

FOR TRAFFIC OPERATIONS

FINAL REPORT

MARCH 2003

INTUS

SYNTHESIS OF SAFETY FOR TRAFFIC OPERATIONS

Final Report

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1. Report No.	2. PO Number	3. ISBN No. N/A
4. Title and Subtitle Synthesis of Safety for Traffic Operations		5. Report Date March 2003
		6. INTUS Code
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9. Performing Organizations Name and Address Intus Road Safety Engineering Inc. 4261 Price Court Burlington, ON L7M 4X3 Canada		10. E-mail Intus@intus.ca
		11. Contract No. T8056-010057/001/SS
12. Sponsoring Agency Name and Address Transport Canada, Road Systems Tower C, Place de Ville 8th Floor 330 Sparks Street Ottawa, ON K1A 0N5		13. Type of Report Final Report
		14. Sponsoring Agency Project Manager Leanna M. Belluz Road Research Engineer
15. Supplementary Note(s)		

16. Abstract

This Synthesis presents the best available evidence respecting the safety impacts of traffic operations and control strategies for Canadian practitioners. Only research and studies that report on crash occurrence, crash severity, or crash surrogates with a proven correlation to crashes are included. Each study is critically reviewed to determine the accuracy of the results, and the particular situation in which they are applicable. The Synthesis is not all-inclusive and will be outdated by ongoing research in the road safety field. The practitioner should keep abreast of recent developments in the traffic operations-safety knowledge base.

Overall the Synthesis should be used by the road safety professional to practice evidence-based road safety, and in pursuing the Canadian vision of making Canada's roads the safest in the world.

17. Keywords	18. Distribution Statement		22. Distribution Agency
Traffic operations, safety, crash, accident, collision, traffic control device			Transport Canada, Road Systems Tower C, Place de Ville 8th Floor 330 Sparks Street Ottawa, ON K1A 0N5
19. Security Code	20. Price	21. No. of Pages xvi, 207	

DISCLAIMER

REJET DE RESPONSABILITÉ

ACKNOWLEDGEMENTS

This Synthesis was prepared in response to a recognized need for the explicit consideration of and better understanding of road safety in the Canadian traffic operations community. It was initiated and funded by Transport Canada.

Transport Canada and the author wish to express their appreciation to the Technical Advisory Team who helped to shape the contents of the Synthesis, and who provided invaluable feedback on the material presented to them. We would also like to thank the Canadian road safety professionals who were forthright in providing documentation, or assistance when contacted by the author. In particular, the author would like to thank Mr. Sany Zein who provided the author with many research reports containing Canadian research conducted by his firm.

We would also like to thank Mr. Andrew Beal from the Ministry of Transportation, Ontario and Mr. Rino Beaulieu from the New Brunswick Department of Transportation who reviewed the Synthesis while in draft form, and provided valuable comments.

The author would also like to thank Leanna Belluz the Project Manager from Transport Canada whose patience and commitment to quality were welcomed.

EXECUTIVE SUMMARY

Traffic operations practitioners are continually making decisions that impact on the safety performance of the transportation network. In order to make the best possible decisions the practitioner must be aware of the best available evidence on safety. The trouble is that the road safety knowledge base is expanding and it is difficult for the practitioner to keep abreast of the conventional wisdom. Moreover, once found, critically appraising, and determining the usefulness of safety-operations research is a daunting task. Practitioners are in need of a document that synthesizes the safety impacts of various traffic operations and control strategies for their day-to-day use. This Synthesis is intended to serve that purpose. It contains information on the safety impacts of traffic operations and control strategies that are most urgent/useful to practitioners, and attempts to highlight the conditions in which the impacts are likely to be realized.

The overarching goals of this Synthesis are to promote *evidence-based road safety* (EBRS) among the Canadian transportation sector, and to help Canada achieve its objective of making Canadian roads the safest in the world. EBRS is the conscientious and judicious use of current best evidence in providing road safety for individuals, facilities, and transportation systems.

Mindful of the above goals, the Synthesis was developed with the following objectives:

- *Focus on traffic operations and control strategies;*

- *The target audience is practitioners and other transportation professionals that make decisions and recommendations respecting traffic operations and control strategies;*

- *Include research and studies that report on crash occurrence, crash severity, or crash surrogates with a proven correlation to crashes; and*

- *As much as possible synthesize Canadian research using Canadian datasets.*

This last objective proved to be overly optimistic. At the start of the project it was believed that practitioners had a vast storehouse of traffic safety research that was unpublished. As it turns out, this is not the case. While certainly some information is unpublished and residing in government files, it appears that mainly due to human and financial resource limitations, practitioners are not undertaking evaluations.

In order to identify the issues and information that would be most valuable to Canadian practitioners, a technical advisory team comprised of provincial and municipal transportation engineering professionals from across Canada was assembled and consulted. The subject matter of this Synthesis was suggested by the advisory team. The main topics presented herein are:

- *Intersection control including signalization, all-way stop control, intersection control beacons, signal timing, and traffic signal design and operation;*
- *Traffic signs;*
- *Pavement markings;*
- *Pedestrian safety;*
- *Bicycle safety;*
- *Legislation and enforcement;*
- *Turn lanes; and*
- *Traffic calming.*

Literature synthesized in this document was gained through the following means:

- *Conventional literature searches of known databases;*
- *Internet search using appropriate key words; and*
- *Personal contact with Canadian academics and road safety practitioners.*

This Synthesis attempts to provide the reader with pertinent information concerning the parameters of the research, and the limitations of the study so that a critical appraisal is possible. In this way the practitioner is better able to judge whether the safety impacts identified are applicable to their particular situation. The key elements of the critical appraisal included a review of how the sites were selected for treatment, the treatment used¸ the study methodology employed, and the results.

In many instances the documented research relating safety to traffic operations and control is not fully reported. That is to say, the results are available, but all of the information necessary to critically appraise the findings is not always available. The primary shortcoming of reported research is the lack of information on site selection procedures. It is well known that evaluation of crash countermeasures that have been implemented at sites selected because of a high incidence of crashes will overestimate the countermeasure effectiveness. A failure to report the site selection process/criteria will leave the practitioner with a deficiency in information.

It is generally accepted that the appropriate metric for road safety is motor vehicle crash (MVC) occurrence and severity. Therefore, only research that assesses the impacts of a particular treatment on MVC occurrence and/or severity are included in the Synthesis. Research that used MVC surrogates are also included if the surrogate has been demonstrated to correlate well with MVCs. This action expands the available literature and remains true to the goal of promoting evidence-based road safety. In the end, operating speed, and traffic conflicts were the two surrogates that have definitive links to MVC occurrence or severity, and are included in the Synthesis.

As the road safety knowledge-base is continually growing, it is important that the practitioner have an understanding of how to appraise new evidence and integrate the findings with those contained herein. To this end, appendices have been provided on evidence-based road safety, critically reviewing literature, and the proper use of safety performance functions. In hopes that practitioners will be encouraged to conduct and document their own research, a further appendix is provided on conducting and authoring research.

Finally, readers should exercise caution in applying the results contained in this Synthesis to situations under examination. Apart from the critical review for methodological flaws in the research, readers must also consider the limitations of transferring research conducted in one jurisdiction to another jurisdiction. Specially, differences in crash reporting, crash severity classifications, driver populations, the vehicle fleet, road system legislation, and design standards may limit the applicability of the reported results to the jurisdiction in which the research was performed.

SOMMAIRE

Les praticiens de la circulation doivent constamment prendre des décisions qui ont un impact sur la performance du réseau de transport au niveau de la sécurité. Afin de pouvoir prendre les meilleures décisions possibles, un praticien doit avoir connaissance de la meilleure information disponible sur la sécurité. La difficulté réside dans le fait que la base de connaissances sur la sécurité routière se développe constamment, et qu'il est difficile pour le praticien d'en avoir une totale connaissance. De plus, après l'identification de diverses recherches sur la sécurité, il demeure difficile d'en faire une évaluation critique et d'en déterminer l'utilité. Le praticien a souvent besoin d'un document qui fait la synthèse des effets sur la sécurité de diverses stratégies et initiatives de gestion et contrôle de la circulation, pour son utilisation quotidienne. C'est l'objet de la présente synthèse. Elle contient de l'information sur les effets sur la sécurité de stratégies de gestion et contrôle de la circulation qui sont particulièrement urgentes/utiles pour les praticiens, et elle tente d'identifier les situations dans lesquelles ces effets se manifesteront probablement.

L'un des objectifs de cette synthèse est de promouvoir la *sécurité routière sur la base des observations (SRBO)* dans le secteur canadien de la gestion du transport, et d'aider le Canada à réaliser son objectif de faire du réseau routier canadien le plus sûr au monde. Le principe SRBO conduit à une utilisation judicieuse et consciencieuse des meilleures preuves/observations actuellement disponibles pour l'optimisation de la sécurité routière pour chacun et pour les systèmes de transport.

Compte tenu des objectifs généraux ci-dessus, la synthèse a été élaborée sur la base des critères suivants :

- *Concentration sur les stratégies de contrôle et gestion de la circulation;*
- *Auditoire cible constitué de praticiens et autres professionnels des transports qui prennent des décisions et formulent des recommandations à l'égard des stratégies de contrôle et gestion de la circulation;*
- *Prise en compte des études et travaux de recherche sur collisions et sévérité des collisions, ou portant sur des simulations des collisions lorsqu'une corrélation avec les collisions est démontrée; et*
- *Prise en compte optimale des travaux de recherche canadiens et des données recueillies au Canada.*

Ce dernier objectif s'est révélé être particulièrement optimiste. Au début du projet, on pensait que les praticiens avaient accès à une grande quantité d'études non publiées sur la sécurité routière, mais ce n'était en fait pas le cas. Tandis qu'on peut certainement trouver dans les dossiers gouvernementaux une certaine quantité d'information non publiée, il semble que – essentiellement du fait de la limitation des ressources humaines et financières – les praticiens n'en entreprennent pas l'évaluation.

Pour l'identification des questions et de l'information qui seraient particulièrement utiles pour les praticiens canadiens, une équipe de consultation technique rassemblant des professionnels de l'ingénierie des transports des provinces et de municipalités de tout le Canada a été constituée et consultée. C'est cette équipe consultative qui a suggéré le sujet de cette synthèse. Les principaux sujets présentés ici sont les suivants :

- *Contrôle des intersections, ceci incluant signalisation, obligation d'arrêt sur chaque voie, signaux lumineux de contrôle des intersections, minutage des signaux, et conception et exploitation de la signalisation routière;*
- *Signalisation routière;*
- *Marquage sur la chaussée;*
- *Sécurité des piétons;*
- *Sécurité des cyclistes;*
- *Législation et application de la législation;*
- *Voies réservées pour changement de direction; et*
- *Ralentissement de la circulation.*

Les documents publiés utilisés pour la production de cette synthèse ont été identifiés par les moyens suivants :

- *Recherche conventionnelle dans les bases de données connues d'articles publiés;*
- *Recherche sur le réseau Internet sur la base des mots-clé appropriés; et*
- *Contacts personnels avec des spécialistes canadiens de la sécurité routière (praticiens et universitaires).*

Dans cette synthèse, on a cherché à présenter au lecteur l'information pertinente concernant les paramètres de la recherche et les limitations de l'étude, afin qu'une évaluation critique soit possible. De cette manière, le praticien est mieux en mesure de déterminer si les effets identifiés sur la sécurité devraient être pris en compte dans chaque situation particulière. Les éléments-clés de l'évaluation critique sont les modes de sélection des sites pour traitement, le mode de traitement utilisé, la méthodologie employée dans l'étude, et les résultats.

Dans de nombreux cas, il n'est pas totalement fait rapport des recherches documentées qui font une relation entre la sécurité et le contrôle et la gestion de la circulation; c'est-à-dire, que certains résultats sont disponibles, tandis que la totalité de l'information nécessaire pour une évaluation critique des observations n'est pas toujours disponible. La principale déficience des travaux de recherche dont il est fait rapport réside dans le manque d'information sur les procédures de sélection des sites. Il est notoire que l'évaluation des mesures mises en oeuvre pour la réduction des collisions, sur des sites sélectionnés sur la base du grand nombre de collisions observées sur ces sites, a tendance à surestimer l'efficacité des mesures de prévention. Le non-rapport des critères et procédures de sélection des sites constitue une importante déficience en ce qui concerne l'information du praticien.

Il est généralement accepté que le nombre de collisions de véhicules à moteur (CVM) et la gravité des collisions sont les facteurs de quantification appropriés pour l'évaluation de la sécurité routière. Par conséquent nous n'avons inclus dans la synthèse que les travaux de recherche dans lesquels on a évalué les effets d'une mesure particulière de réduction du nombre, et/ou de la gravité des CVM.

Les travaux de recherche dans lesquels on a utilisé un substitut du nombre de CVM ont également été inclus, si la corrélation entre le substitut et le nombre de CVM pouvait être démontrée. Cette approche développe le volume de documentation utilisable et demeure parfaitement légitime face à l'objectif de promotion de la sécurité routière basée sur les observations. En dernier ressort, la vitesse de circulation et les conflits de circulation sont les deux substituts de quantification qui manifestent une claire corrélation avec le nombre ou la sévérité des CVM, et les études basées sur ces facteurs ont été incluses dans la synthèse.

Alors que la base de connaissance sur la sécurité routière se développe constamment, il est important que le praticien sache comment évaluer de nouvelles observations et les intégrer avec les résultats déjà connus. À cet effet, nous avons inclus des annexes sur la sécurité routière, basées sur des observations, l'étude critique de la documentation spécialisée et l'utilisation adéquate des fonctions de performance dans le domaine de la sécurité. Dans l'espoir que les praticiens soient incités à exécuter leurs propres recherches et à faire rapport adéquatement, une autre annexe a été incluse sur l'exécution des projets de recherche et la présentation des rapports.

Finalement, le lecteur devrait être circonspect en ce qui concerne l'application des résultats présentés dans cette synthèse à chaque situation spécifique. En plus d'une analyse critique pour l'identification de déficiences méthodologiques dans les travaux de recherche, le lecteur doit également prendre en compte les limitations affectant la possibilité de transfert des résultats de recherche, d'une juridiction à une autre. Spécifiquement, des facteurs comme les variations sur le mode de rapport des collisions, la classification de la sévérité des collisions, la population de conducteurs, le nombre de véhicules en circulation, la législation routière et les normes de conception, peuvent limiter l'application des résultats identifiés dans un rapport à la juridiction dans laquelle l'étude a été exécutée.

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Chapter 1: Introduction

CHAPTER 1: INTRODUCTION

The provision of safe roads is a primary objective for the transportation professional in designing and operating the highway system. Despite safety being of paramount importance, understanding the safety impacts of different traffic operations and control strategies is still one of the most challenging, and least understood, aspects of transportation engineering. Among other things, measuring safety impacts is complicated by the random, infrequent nature of motor vehicle crashes, and the less than desirable conditions in which evaluations must take place. Therefore, it is not surprising how little quality research the transportation community has performed respecting the safety consequences of different traffic control and operations strategies, and how much practitioners rely on evidence that is anecdotal.

Historically, most of what the practitioner learned about safety impacts came from conducting studies (which tend to be infrequent and often hastily conducted), through information presented at conferences, or by reviewing literature and trade journals. In this way the information respecting the safety impacts of various traffic operations and control strategies has been fractured. Over the past two decades the transportation profession has devoted a considerable amount of attention to road safety, and a resulting increase in information has resulted. Not only has the volume of research increased, but the quality of the research has also improved.

Finding, critically appraising, and determining the usefulness of safety-operations research is a daunting task. Practitioners are in need of a document that synthesizes the safety impacts of various traffic operations and control strategies for their day-to-day use. This Synthesis is intended to serve that purpose. It contains information on the safety impacts of traffic operations and control strategies that are most urgent/useful to practitioners, and attempts to highlight the conditions in which the impacts are likely to be realized.

Synthesis Objectives and Scope of Work

The overarching goals of this synthesis are to promote *evidence-based road safety* (EBRS) among the Canadian transportation sector, and to help Canada achieve its objective of making Canadian roads the safest in the world (Canadian Council of Motor Transport Administrators, 2000).

EBRS is defined as:

> *The conscientious and judicious use of current best evidence in providing road safety for individuals, facilities, and transportation systems.*

Before proceeding, a few important points should be highlighted about the above definition:

- *Judicious use* – Safety is one of several competing objectives in designing and operating road systems. In some instances, environmental protection, economic considerations, or some other competing objective may warrant a trade-off in safety. It is not sufficient to simply seek out the safest alternative and force it's implementation without due regard for the other system objectives. On the other hand, if safety is to be compromised for the sake of other objectives, then the practitioner should be able to reasonably quantify the safety consequences.

- *Current best evidence* – Firstly, the term 'current' is meant to remind practitioners that the science of road safety is continuously evolving. While documents such as this are helpful in determining the conventional wisdom, the literature is static and at some point it may be outdated by new research. Secondly, the term evidence should serve to remind the practitioner that decisions must be rooted in sound knowledge and not anecdotes and folklore.

Further information on evidence-based road safety is found in Appendix A.

Mindful of the above goals, the Synthesis was developed with the following objectives:

- *Focus on traffic operations and control strategies;*

- *The target audience is practitioners and other transportation professionals that make decisions and recommendations respecting traffic operations and control strategies;*

- *Include research and studies that report on crash occurrence, crash severity, or crash surrogates with a proven correlation to crashes; and*

- *As much as possible synthesize Canadian research using Canadian datasets.*

This last objective proved to be overly optimistic. At the start of the project it was believed that practitioners had a vast storehouse of traffic safety research that was unpublished. As it turns out, this is not the case. While certainly some information is unpublished and residing in government files, it appears that mainly due to human and financial resource limitations, practitioners are not undertaking evaluations.

Use of This Synthesis

This synthesis documents a collection of studies and research projects on the safety impacts of traffic operations and control strategies. It is not intended to be used as a standard, guideline, or procedural document. It is a technical document that provides an unbiased and impartial review of the conventional wisdom on traffic operations and control as it relates to safety. It is a reference document intended to promote better safety decisions among road authorities.

This Synthesis does not purport to be an all-inclusive document. Research on the safety implications of traffic operations and control is growing exponentially, and it was not possible to locate, review, and include all of the available research in this document. Practitioners are encouraged to seek out additional knowledge as required.

Synthesis Organization

This report is organized into twelve chapters and four appendices. The first two chapters are background and introductory material for the reader. Chapters 3 to 11 are the results of the literature search for the topics suggested by the Technical Advisory Team. The last chapter is a summary of the information uncovered by the literature search. The appendices are supporting information, provided to assist the practitioner with implementing EBRS.

Methods and Metrics

The key to evidence-based decision-making is finding the "best evidence" available, and integrating it into the decision-making process.

Methods

The wide variety of traffic control devices, and traffic operational issues that are presented to the practitioner made it impossible to synthesize all of the literature on safety as it relates to traffic operations and control. Therefore, in order to identify the issues and information that would be most valuable to Canadian practitioners, a technical advisory team comprised of provincial and municipal transportation engineering professionals from across Canada was assembled and consulted. The subject matter of this Synthesis was suggested by the advisory team.

Literature regarding the safety impacts of operations and control strategies that were identified by the advisory team was gained through the following means:

- *Conventional literature searches of known databases;*
- *Internet search using appropriate key words; and*
- *Personal contact with Canadian academics and road safety practitioners.*

Documents and publications uncovered by the literature search, and that are of interest to practitioners have been synthesized herein. The databases searched included the United States Department of Transportation National Transportation Library (including TRIS), the Transportation Association of Canada's Library, the McMaster University H.G. Thode Library of Science and Engineering, the University of Waterloo Library, and the Ontario Ministry of Transportation Research Library Online Catalogue.

Since safety evaluations and safety research is a delicate business, blind reliance on documented studies is considered poor practice. The practitioner must critically appraise the work and determine its validity and usefulness for his/her particular situation. To this end, this Synthesis attempts to provide the reader with pertinent information concerning the parameters of the research, and the limitations of the study so that a critical appraisal is possible.

The key elements of the critical appraisal included a review of how the sites were selected for treatment, the treatment used¸ the study methodology employed, and the results. A critical appraisal tool was developed for the review of literature and is included in Appendix B. This tool may be used by the practitioner to critically appraise other research and supplement the information contained herein.

During the preparation of the synthesis it was discovered that a significant proportion of the documented research relating safety to traffic operations and control is not fully reported. That is to say, the results are available, but all of the information necessary to critically appraise the findings is not always available. For instance, many articles do not provide information on site selection procedures. It is well known that evaluation of crash countermeasures that have been implemented at sites selected because of a high incidence of crashes will overestimate the countermeasure effectiveness. A failure to report the site selection process/criteria will leave the practitioner with a deficiency in information.

In an effort to encourage better reporting of road safety research, the Synthesis includes a section on the conduct and reporting of research (see Appendix C).

METRICS

It is generally accepted that the appropriate metric for road safety is motor vehicle crash (MVC) occurrence and severity. MVCs are system failures that are caused by one or more of the three elements of the highway system: the environment (the road and its ancillary devices), the road user, and the vehicle.

Because MVCs are rare and random events using them as the sole measure of effectiveness is limiting. Many evaluation studies, particularly research into the safety of vulnerable road users where crashes are very infrequent, use crash surrogates to determine effectiveness. The most common surrogates are traffic conflicts, violations, road user behaviour, and speed. The use of surrogates is a reasonable action.

The body of knowledge that is formed by research into the impacts of traffic operations and control strategies on crash surrogates is formidable and important. Therefore, the original intent of including only research that used crash frequency and severity as the primary endpoints was abandoned in favour of including research that measured the impacts on crash surrogates.

However, in order to be true to the goal of promoting evidence-based road safety, research that used crash surrogates is included only if there is a reasonable expectation that the crash surrogate is correlated with crash occurrence or severity.

For instance, operating speed is often used as a measure of the "safety" of a situation. Measures that reduce actual travel speed are seen to be improving system safety. Since the collected wisdom on speed is that lowering speeds reduces crash severity, evaluations that measure the impacts of strategies on speed are included in this Synthesis. Alternatively, crash surrogates such as "stop sign violations" particularly on local streets, to the best of our knowledge, have no definitive correlation with crashes and are not included in this Synthesis.

Chapter 2 includes a description of crash surrogates and provides references to the research that demonstrates their correlation with crashes.

With respect to terminology, the terms "safety impact" and "safety effect" are more appropriate than "safety benefits", as a modification may not always bring about the anticipated improvement in safety. The term "crash countermeasure" is also avoided, as traffic operations and control strategies are not always safety-driven. For instance, traffic signals are most often introduced to better control right-of-way between conflicting traffic flows, and to reduce delay. Traffic signals are not typically implemented as a crash countermeasure. Nonetheless, the decision to implement signals will certainly have an impact on the safety of the intersection.

The current science of road safety employs two different methods to measure the safety impacts of a traffic operations/control choice.

Method 1: Using Safety Performance Functions

The safety of a facility is best represented by the expected number of crashes, as derived from a well-developed crash prediction model, also known as a safety performance function (SPF). By developing and comparing SPFs for two different conditions, which vary only by the traffic operations strategy of interest, one can predict the safety impacts of the traffic operations strategy. For instance, consider the following SPFs:

$N = 0.0012ADT^{0.8116}$ for an arterial under two-way operation [1.1]
$N = 0.0009ADT^{0.8010}$ for an arterial under one-way operation [1.2]

where: N = crashes per kilometre per year
ADT = Average Daily Traffic

If the arterial is currently operating as a one-way street with 26,000 vehicles per day, the expected annual number of crashes is 3.1 crashes/km. If it is proposed to convert the street to two-way operation, and it is expected that traffic volumes will not change as a result, the expected annual number of crashes would be 4.6 crashes/km. By employing the SPFs, one can determine the safety consequences of a particular action.

This method of assessing safety impacts is essentially the same as employing a cross-sectional study design. The safety performance of two or more facilities that are considered to be similar in all aspects except one are compared and the difference in performance is assumed to be caused by the differing aspect.

The above example is somewhat simplistic, and the proper use of SPFs requires slightly more effort than "plugging in the numbers" and examining the output. A more in depth discussion on calibrating and using SPFs is provided in Appendix D.

Method 2: Developing Crash Modification Factors

Observational before-after studies that are properly conducted can yield information on the expected number of crashes that can be translated into crash modification factors (CMFs). CMFs are multipliers that indicate the residual number of crashes that are to be associated with a particular operation or control strategy. If the CMF is less than one, there is a safety benefit; if the CMF is greater than one, there will be an increase in crashes. A traffic operations strategy that is expected to reduce crashes by 23% would have a CMF of 0.77.

CMFs are usually developed from before-after studies. Again the potential for developing misleading results is high since observational studies of rare and random events (crashes) requires some skill and knowledge of statistics and a good understanding of the conditions and events that could obfuscate or confound the results. Hauer (1997) is an excellent reference respecting observational before-after studies in road safety.

Study Methodologies

The evaluation of the safety impacts of a traffic operations or control decision using crash data (most often crash frequency) can take many forms with inherent strengths and weaknesses. These methodologies are the basis for the development of CMFs and play a major role in their reliability. The most popular methods are discussed below:

- *Naïve before-after study: A simple comparison of the number of crashes (or crash rates) before and after treatment. The CMF is calculated by dividing the number of "after" crashes by the number of "before" crashes. Most safety studies are observational, are conducted in the field, and all conditions are seldom constant. Therefore, the naïve before-after study does not account for any changes in the crash record that may have resulted from changes other than the treatment (i.e., unrelated effects). Furthermore, in the instance that treatment sites have been selected due to an aberrant crash record, this study methodology does not account for any regression-to-the-mean effects*[1].

- *Before-after Study with a Control Group: The control group (i.e., a group of untreated sites) is introduced to account for the unrelated effects. It is assumed that any change in the crash record in the control group would have also occurred in the treatment group, if no treatment were applied. The CMF is therefore calculated by the following equation:*

$$\text{CMF} = \frac{A/C}{B/D} \qquad [1.3]$$

where: A = "Before" crashes in the control group
B = "Before" crashes in the treatment group
C = "After" crashes in the control group
D = "After" crashes in the treatment group

This is an improvement on the naïve before-after study, but still does not account for possible regression-to-the-mean bias. Also, one must be careful to amass an

[1] See Appendix C for a brief description of the regression-to-the-mean effect.

adequate size control group, otherwise random variations in the control group data could jeopardize the validity of the results.

- *Cross-sectional study: This study methodology involves the study of two different groups of sites that vary only by the feature of interest. For instance, the crash records of two-lane rural arterials with an 80 km/h speed limit would be compared to the crash record of two-lane rural arterials with a 60 km/h speed limit, to determine the safety impacts of a reduction in the speed limit from 80 km/h to 60 km/h. The most obvious shortcoming of this study methodology is the ability to match the two groups of sites on all other features that will impact on crash occurrence and severity.*

- *Empirical Bayes (EB) Techniques: The EB technique is actually an observational before-after study that uses advanced mathematics to minimize biases introduced by unrelated effects, and regression-to-the-mean. The procedure is based on the premise that the true safety record of a site is some combination of the actual crash frequency at the site itself, tempered by the mean crash record for a collection of sites with similar characteristics. These two measures of site safety are combined by considering the amount of site data available, and the reliability of the group mean. The EB technique most often takes the form of using SPFs (as discussed previously), combined with site-specific crash data in the before and after periods to determine the safety impacts. Appendix D provides a more detail on how this method is used.*

The foregoing is a very brief description of the more popular study methodologies found in the literature and is not exhaustive or complete. For more information and discussion on each of these techniques the reader is referred to Hauer (1997), and Hamilton Associates (1997).

Caveats and Cautions

Caution should always be exercised when attempting to apply the results of a study undertaken in one location to another analogous location. This is particularly true when attempting to transfer results from one jurisdiction to another. The most common pitfalls are as follows:

Reporting methods vary between jurisdictions

The definition of an “intersection crash” is a prime example of varying methods. In some jurisdictions/research, intersection crashes are defined as all crashes that occur with 30 metres of the intersection, and all crashes that are recorded as intersection-related. In other research the limit is extended within 250 metres of the intersection.

Severity classifications differ

All of the Canadian provinces and territories classify fatal crashes as those crashes that result in the death of an involved individual within 30 days of the crash, except for Quebec who use an eight-day threshold.

Driver populations

Inexperienced and novice drivers are known to be over-represented in certain types of crashes. A variation in the application of graduated drivers licensing (GDL) across the provinces and territories will necessarily impact the safety performance between provinces. For instance, novice drivers are over-represented in night-time crashes. In comparing these crash statistics from Alberta, which does not have GDL, to Nova Scotia, which has GDL, there may be a difference.

The fleet may differ

Areas and facilities that accommodate industries that rely heavily on trucking of goods (e.g., logging, and the steel industry) will have a greater percentage of truck traffic. Larger vehicles are known to be associated with fewer crashes, but also tend to produce more severe crashes. Therefore, the application of results gathered from elsewhere need to recognize and accommodate differences in the fleet.

Supporting legislation

Again variation across provincial and territorial boundaries may limit the transferability of research results. For example, British Columbia law indicates that motorists must yield the right-of-way to pedestrians at marked and unmarked crosswalks; this may impact the safety performance of crosswalks. Ontario legislation imposes no such duty on the motorist to yield to pedestrians except at signalized locations, and locations with Pedestrian Crossovers.

Design standards

Although consistency and standardization of traffic operations and control strategies is a major initiative of the transportation profession, there is some room for variation between jurisdictions. For example, the use of backboards on, and the position of traffic signal heads can vary between road authorities. SPFs for signalized intersections that were developed in a jurisdiction that does not routinely use backboards, and tends to post mount the signal head are not directly applicable to a jurisdiction whose minimum standard requires backboards and mast-arm mounted signal heads.

In concluding this section on methods and metrics it suffices to state that:

- *Crash occurrence and severity are the primary metrics of safety;*
- *Crash surrogates that are definitively linked to either occurrence or severity are reasonable alternative metrics;*
- *The typical methods of determining the safety impacts of a particular traffic operations or control strategy is to use either properly developed safety performance functions (SPFs) or crash modification factors (CMFs);*
- *Different study methodologies have different inherent strengths and weaknesses and are key factors in determining the reliability of the study results; and*
- *Transferability and validity of SPFs and CMFs need to be assessed by the practitioner.*

Chapter 2:

Crash Surrogates

CHAPTER 2: CRASH SURROGATES

The safety of a facility is measured by crash frequency and severity. As crashes are random and rare events, sometimes requiring years before an adequate sample size of "after" data can be amalgamated to detect an effect, it is commonplace for road safety researchers to use crash surrogates. Surrogates are substitute measures that are representative of crashes or crash severity but that occur with greater frequency. Typical surrogates found in the literature include, compliance with traffic laws, speed of travel, traffic conflicts, lane positioning, and other forms of driver behaviour and performance.

Under EBRS, surrogates only have value in measuring safety if there is a strong correlation with crash occurrence or severity. It is not enough to rely on a likely etiology. The following examples illustrate this point:

- *Many studies of horizontal curves use lateral lane positioning as a surrogate measure of safety. The notion that being in the centre of the lane is the "safest" position is frequently the hypothesis. It is well-known that when traversing isolated horizontal curves motorists will move from the outside of the lane at the beginning of curve, to the inside of the lane at the apex of the curve (Stimpson et al, 1977). This phenomenon is known as "curve lengthening", shows a great variance in lateral lane position, and as long as there are no shoulder or centreline encroachments, probably is desirable from a safety perspective.*

- *A pedestrian crosses the road at a signalized intersection during the "don't walk" indication. This action may be used as a measure of safety under the hypothesis that crossing without the right-of-way places the pedestrian in a potential conflict with motor vehicle traffic, and therefore decreases safety. The pedestrian crash statistics certainly lend support to that hypothesis. Pedestrian crashes are frequently the result of pedestrians crossing without the right-of-way. However, if a pedestrian crosses against the light when the visibility is adequate, and (s)he has determined that the way is clear, is this action indicative of lower safety? Unless this behaviour can be correlated with crash occurrence, it is nothing more than a measure of noncompliance.*

The above examples are reflective of the following sentiment from the Guarding Automobile Drivers through Guidance Education and Technology (GADGET) Final Report:

> *A documented effect of some road measure on some driver behaviour is not always easily interpreted in terms of safety. In addition, accident data or convincing theoretical arguments regarding relationships of the given behavioural observation to safety is needed. [Austrian Road Safety Board, 1999]*

Much research has been conducted using crash surrogates that are intended to measure the safety effects of a particular treatment. Much less research has been conducted to determine if a correlation exists between the surrogate and crashes.

In this Synthesis, surrogates are behaviour-based, reflecting road users behaviour and performance; surrogates are not environmental, reflecting the road or surrounding conditions. The road and its ancillary devices are the variables that engineers manipulate to increase safety. If one intends to measure safety by some other means than crashes, one cannot manipulate the road to see the effect it has on the road. Engineers modify roads, which presumably have an impact on road user behaviour or performance, which in turn leads to reduced crash occurrence and/or severity.

Again it should be noted that the focus of this Synthesis is on safety. Metrics such as traffic violations, although they may not be correlated with crashes, may be helpful to traffic operations engineers in other ways. For example, failing to yield to pedestrians is a concern that is often brought to the attention of municipal traffic operations staff. These violations have not been correlated with crash occurrence, and are not necessarily reflective of crashes. Nonetheless, since the violations are of concern to the public, they are at the very least a quality of life issue. The practitioner should not propagate the misconception that violations are indicative of crashes, but may want to address the concern as a quality of life issue.

The following safety surrogates and their recommendations for use have been found in the literature.

Stop Sign Compliance

In a study involving 2,830 observations at 66 intersections, Mounce (1981) found there to be no correlation between stop sign violation rates and crashes. In the absence of any definitive research that links this crash surrogate to crash occurrence, it will not be considered in this report.

Traffic Conflicts

There is a considerable amount of discrepancy in reported use of traffic conflicts as crash surrogates. Some of the discrepancy comes from variation in how conflicts are defined, and the methods of measurement. The prevalent definition and method of measurement used in British Columbia, and Ontario is outlined in the second edition of the Traffic Conflict Procedure Manual [Hamilton Associates, 1989]. In a study to determine the correlation between conflicts and crashes, it was found that a correlation existed only between the most severe conflicts and crashes. Conflicts that were less severe, were not correlated with crashes, and are likely more indicative of "normal driving". Nonetheless, there appears to be some correlation between conflicts and crashes, and therefore they are suitable crash surrogates.

In a separate study conducted by Salman and Al-Maita (1995) traffic conflicts and crashes were correlated at three-leg unsignalized intersection in Jordan. The researchers used the traffic conflict technique as developed by the Federal Highway Administration (Parker and Zeeger, 1989). Eighteen sites were studied and were selected because of the availability of crash data, and because all of the sites had low pedestrian volumes, approaches that were two-way streets, no visibility restrictions, no turn restrictions, no parking restrictions, and no appreciable approach grades.

Crash data spanned a three year period, and crashes that occurred during wet weather, at night, on the weekend, and those that involved a pedestrian were excluded from the analysis. Traffic conflict studies were conducted between 07:00 hrs and 18:00 hrs on weekdays during the summer.

A linear regression of crashes and conflicts yielded a statistically significant correlation as follows:

$$N = 0.744 + 0.0116X \quad [2.1]$$

where: N = Annual number of crashes
X = Mean hourly conflict count

Together the Hamilton Associates and the Salman and Al-Maita provide reasonably sound evidence that traffic conflicts and collisions are correlated.

Speed

While the link between speed and crash occurrence is somewhat undecided, there is nonetheless a strong correlation between speed and crash severity [IBI Group, 1997]. While the debate over speed and crash occurrence continues, it is enough to know that speed is correlated with severity, therefore it will be considered as a suitable crash surrogate.

While conflicts and speed are determined to be suitable crash surrogates, their direct application to safety management can be troublesome. The statistical correlation between the surrogate and the crash occurrence/severity needs to be established for the surrogates to be used in a meaningful quantitative manner. A linear correlation would permit the pseudo-CMF for the surrogate to be used directly as the CMF. However, non-linear relationships would not permit direct application of the surrogate results.

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Chapter 3: Intersection Control

CHAPTER 3: INTERSECTION CONTROL

SIGNALIZATION

Agent (1988)

Agent (1988) in a comprehensive study of 65 rural, high-speed intersections in Kentucky, examined the effects of intersection control on crashes. The sites were selected to provide a variety of traffic volumes, roadway geometrics, and traffic control. The 65 locations were comprised of 47 signalized sites, 15 minor street stop-controlled sites, and 3 all-direction stop-controlled intersections. Sixteen of the 18 unsignalized locations were supplemented with intersection control beacons. Other forms of traffic control at the study sites varied from site-to-site, ex., advance warning, signal phasing, stop lines, transverse rumble strips, etc.

Using a cross-sectional analysis of the study sites, Agent reported the crash rates shown in Table 3.1. The number of intersections exceeds 65, because some of the sites had changes in intersection control during the study period.

TABLE 3.1: Crash Rates at Rural High-Speed Intersections in Kentucky

Control Type	No. of Sites	Crashes	Crash rate (per MV)	CMF*
Stop Sign	27	338	1.1	---
Stop + Beacon	37	541	1.2	1.09
Traffic Signal	46	1290	1.2	1.09

* - Assuming a stop sign as the base condition.

Using a naïve before-after study of crash frequency, Agent also examined the effects of changing intersection control on safety (Table 3.2).

TABLE 3.2: The Effect of Signalization of Crashes in Kentucky

Intersection Control		No. Sites	Crash rate		CMF
Before	After		Before	After	
Stop sign	Stop+Beacon	11	1.1	1.0	0.91
Stop sign	Traffic Signal	16	1.3	1.8	1.38
Stop+Beacon	Traffic Signal	20	1.4	1.1	0.79

Finally, Agent provides information on the severity of crashes at all locations, as shown in Table 3.3.

TABLE 3.3: Crash Severity for Kentucky Intersections

Crash Severity	Proportion of Crashes (%)			
	Stop Sign	Stop + Beacon	Signal	State wide
Fatal	1.5	2.6	0.9	0.2
Injury	37.2	39.6	34.1	23.6
PDO	61.3	57.8	65.0	76.2

It is difficult to extract any meaningful conclusions from this research. The results of the cross-sectional analysis and the before-after study provide conflicting evidence – in the cross-sectional study adding a beacon to an unsignalized location should produce an increase in crashes; in the before-after study there is a decrease. Furthermore, replacing a stop+beacon with a signal exhibits no change in crash rate in the cross-sectional study, but a fairly substantial decrease in crash rate in the before-after study.

Lalani (1991)

The City of San Buenaventura, California as part of a Comprehensive Safety Program undertook signalization of several intersections as a safety improvement (Lalani, 1991). Sites were selected because they had three or more crashes in a one-year period, the high crash locations. The results from a naïve before-after analysis using crash frequency from four of the signal installations are shown in Table 3.4. The before and after periods were one year.

TABLE 3.4: Safety Impacts of Signalization in San Buenaventura, California

Location	Crashes		CMF
	Before	After	
A	5	2	0.40
B	8	1	0.13
C	17	0	0.00
D	4	3	0.75
Totals	34	6	0.18

Lalani does not account for exposure in the safety analysis but reports that traffic volumes in the city increase at an average rate of 6% per annum.

Poch and Mannering (1996)

In an attempt to determine the interactions between intersection approach characteristics and crash occurrence, Poch and Mannering (1996) developed crash prediction models for 63 intersections in Bellevue, Washington. The intersections were located in urban settings, and were targeted for operational improvements. The models that were developed were for intersection approaches; therefore the crashes at each intersection were assigned to a specific approach.

The models were developed using negative binomial regression, which is appropriate for modelling sparse, non-negative integers such as motor vehicle crashes. A total of 64 variables (intersection characteristics) were analysed. Four separate models were developed for all, rear-end, angle, and approach-turn crashes. The model for all crashes is shown in Equation 3.1.

$$\begin{aligned}N_{all} = {} & 0.244 \exp(0.251V_{lt} + 0.0902V_{rt} + 0.0523V_o + 0.153L) * \\ & \exp(-0.753TC - 0.325TS + 0.27P_2 + 0.61P_8 - 0.468PL) * \\ & \exp(-0.336LS - 1.093AL + 1.123SD + 0.201TL - 0.899LA) * \\ & \exp(0.0203SL - 0.0075SL_o) \qquad [3.1]\end{aligned}$$

where: N_{all} = Annual number of crashes on the intersection approach
V_{lt} = Average daily left-turn volume in thousands
V_{rt} = Average daily right-turn volume in thousands
V_o = Average daily total opposing traffic volume in thousands
L = Number of through, combined through-right, and right-turn lanes
TC = Traffic control on approach (1 if no control; 0 otherwise)
TS = Signal control on approach (1 is signalized; 0 otherwise)
P_2 = Two-phase signal (1 is two-phase; 0 otherwise)
P_8 = Eight-phase signal (1 if eight-phase; 0 otherwise)
PL = Protected left-turn (1 if protected; 0 otherwise)
LS = Local street approach (1 if local street; 0 otherwise)
AL = All approaches are local streets (1 if all local; 0 otherwise)
SD = Sight-distance restrictions (1 is restricted; 0 otherwise)
TL = 1 if there is a combined through-left lane and two or more lanes on the approach; 0 otherwise
LA = 1 if left-turns are not aligned and the approach does not have a single lane, protected left, or stop control; 0 otherwise
SL = Approach speed limit (km/h)
SL_o = Opposing approach speed limit (km/h)

The model provides some noteworthy results with respect to traffic operations and control strategies. Firstly, in using an uncontrolled approach as the base condition, it can be estimated that a change in intersection control would be associated with an increase in crash frequency on that approach according to the CMFs shown in Table 3.5.

TABLE 3.5: CMFs for a Change in Approach Control

Approach Control		CMF
Uncontrolled		1.00
Stop-control		2.12
Traffic signal	Two-phase	2.01
	Eight-phase	1.77

The above results are consistent with intuition. In applying the above CMFs to a two-way stop controlled intersection with four approaches, signalization would decrease crash occurrence on the two stop-controlled approaches, and increase crashes on the two uncontrolled approaches. Furthermore, the results indicate that modifying signal operation from two-phases to eight-phases is a safety benefit.

The Poch and Mannering analysis selected intersections that were identified for operational improvements. This limits the applicability of the model and the CMFs to intersections that are considered to be operationally deficient and requiring remedial work. In this instance, remedial work consisted of intersection reconstruction that improves the intersection, modifying intersection control, modifying the signal timing/phasing at signalized locations, and/or providing channelization.

Tople (1998)

Tople (1998) in an evaluation of hazard elimination and safety projects included a review of the safety benefits of signalization at nine locations in South Dakota. The evaluation was a naïve before-after study of crash frequency and crash severity. The impact on crash severity was determined through a comparison of equivalent property damage only crashes, using monetary conversations deemed appropriate by the investigation team. Three years of before and three years of after crash data were used in the analysis.

The results of the analysis are presented in Table 3.6.

TABLE 3.6: Safety Impacts of Signalization in South Dakota

Improvement Type	No. of Sites	AADT Range	Crashes			EPDO Crashes*		
			Before	After	CMF	Before	After	CMF
Signal	9	5960 - 20995	188	139	0.74	2313	2307.5	1.00

* EPDO crashes were calculated as (1300*F)+(90*I)+(18*N)+(9.5*P)+PDO

where: F = fatal crash
I = incapacitating injury crash
N = non-incapacitating injury crash
P = possible injury crash
PDO = Property damage only crash

The Tople analysis possesses many potentially serious flaws. Most importantly, the sites were selected for treatment as part of a safety program. This means that the crash record was likely abnormally high, and there is a great potential for regression to the mean artefacts. This shortcoming is likely offset somewhat by a failure to account for changes in exposure. Traffic volumes were not controlled for, but typically volumes tend to increase which would lead to a higher "after" count of crashes. In the end, the South Dakota results are based on a limited number of sites and weak analyses.

Ministry of Transportation for Ontario (1998)

The Ministry of Transportation for Ontario (MTO, 1998) undertook a comprehensive study of the crash data to determine the safety performance of their facilities, including intersections. In the conduct of the study, SPFs, disaggregated by severity, were developed for signalized intersections owned by the province of Ontario. The form of the SPF is shown in Equation 3.2, the estimates of the parameters are shown in Table 3.7.

$$N = a\ AADT^{b} \qquad [3.2]$$

where: N = annual number of crashes
AADT = Average annual daily traffic of the main road
a, b = parameters are shown in Table 3.10

TABLE 3.7: SPFs for Ontario Signalized Intersections

Intersection Type	Collision Type	a	b	AADT Range
Signalized, 4 approaches	Fatal Injury PDO	0.0002283 0.0103469 0.0169214	0.54866	1,090 to 34,280
Signalized, 3 approaches	Fatal Injury PDO	0.0000853 0.0038654 0.0063216	0.54925	4,600 to 28,460

A SPF for unsignalized intersections in Ontario was not developed by the MTO. This limits the usefulness of the SPF for signalized intersections. Nonetheless, the SPF is helpful as a measure of safety.

Sayed and Rodriguez (1999)

Sayed and Rodriguez (1999) developed SPFs for unsignalized intersections in British Columbia using crash and traffic volume data from urban intersections in the Greater

Vancouver Regional District, and Vancouver Island. The dataset included 186 intersections with three-legs and 233 intersections with four legs. Three years of data were used in the model development, sites were selected on the basis of available data. Only one-way (at T-intersections) and two-way (at cross intersections) stop controlled intersections were included in the analysis. Crashes that were within 30 metres of the intersection, or were coded as "intersection-related" were defined as intersection crashes.

The range of data used in the analysis are shown in Table 3.8.

TABLE 3.8: Data Ranges Used in Developing SPFs for Urban, Unsignalized Intersections

Variable	Statistic		
	Minimum	Maximum	Mean
Major Road ADT	500	47,800	13,343
Minor Road ADT	100	11,000	1,735
Crashes/year	0.3	11	1.72

SPFs were developed using generalized linear modeling assuming a negative binomial distribution. The model form is shown in Equation 3.3, the parameters are as shown in Table 3.9.

$$N = a(AADT_{major}/1000)^{b1}\ (AADT_{minor}/1000)^{b2} \qquad [3.3]$$

where: N = Crashes per 3 years
$AADT_{major}$ = Average annual daily traffic of the major road
$AADT_{minor}$ = Average annual daily traffic of the minor road
a, b1, b2 = constants as shown in Table 3.7

TABLE 3.9: SPFs for Urban, Unsignalized Intersections in BC

No. Approaches	a	b_1	b_2
3	0.9333	0.4531	0.5806
4	1.5406	0.4489	0.6475

Vogt (1999)

Vogt (1999) developed crash models for rural signalized and stop-controlled intersections using combined data from Michigan and California. Eighty-four stop-controlled intersections with three approaches, 72 stop-controlled intersections with four approaches, and 49 signalized intersections with four approaches were included in the analysis. The stop-controlled intersections had four lanes on the main road, and two lanes on the minor road; the signalized intersections had two-lanes on all approaches. Generalized linear regression using a negative binomial distribution was used to model all crashes within 250 feet of the intersection on the main road, and with 100 and 250 feet of the intersection on the side road, in California and Michigan, respectively.

The resultant models are as shown in Equations 3.4 to 3.5. For four lane main roads, with stop-controlled two-lane minor roads and three approaches:

$$N = 0.000000192\ ADT_m^{1.433}\ ADT_s^{0.269}\ \exp(-0.0612M + 0.0560D) \qquad [3.4]$$

where:
- N = Number of crashes per year
- ADT_m = Average two-way major road traffic per day
- ADT_s = Average two-way side street traffic per day
- M = Median width on the major road (metres)
- D = Number of driveways on the major road within 76 metres of the intersection centre

For four lane main roads, with stop-controlled two-lane minor roads and four approaches:

$$N = 0.0000777\ ADT_m^{0.850}\ ADT_s^{0.329}\ \exp(0.110PL - 0.484L) \qquad [3.5]$$

where:
- N = Number of crashes per year
- ADT_m = Average two-way major road traffic per day
- ADT_s = Average two-way side street traffic per day
- PL = Proportion of peak hour traffic approaching on the major road that is turning left (%)
- L = 0 if major road has no left-turn lane; 1 if at least one left-turn lane.

For the signalized intersection of two-lane roads with four approaches:

$$N = 0.000955\ ADT_m^{0.620}\ ADT_s^{0.395}\ \exp(-0.0142PL_s + 0.0315T) * \exp(-0.675L_T + 0.130V) \qquad [3.6]$$

where:
- N = Number of crashes per year
- ADT_m = Average two-way major road traffic per day
- ADT_s = Average two-way side street traffic per day
- PL_s = Proportion of peak hour traffic approaching on the side street that is turning left (%)
- T = Proportion of peak hour traffic approaching the intersection that consists of trucks (%)
- L_T = 0 if the major road does not have a protected left turn; 1 if the major road has at least one protected turn phase
- $V = 0.5 * (V_m + V_s)$

V_m = the sum of the absolute percent grade change per 100 feet for each vertical curve along the major road, any portion of which is within 800 feet of the intersection centre, divided by the number of such curves

V_s = the sum of the absolute percent grade change per 100 feet for each vertical curve along the side street, any portion of which is within 800 feet of the intersection centre, divided by the number of such curves

The above models by themselves cannot be used to determine the safety impacts of a change in intersection control. Nonetheless, they provide good estimates of the long-term safety of the three intersection types. The SPFs for the unsignalized locations may be combined with the crash record at an existing intersection to better predict the long-term safety of the location. The SPF for the signalized intersection may be used similarly, or to predict the safety performance of an unsignalized intersection if it is signalized.

To combine the SPF with the crash record of a site, the "overdispersion parameter" (k) must be know, they are shown in Table 3.10. For a description on how to combine SPF estimates with crash records see Appendix D.

TABLE 3.10: Overdispersion Parameters from Vogt (1999)

Equation	k
3.2	0.389
3.3	0.458
3.4	0.116

Transport Research Laboratory (2000)

The Transport Research Laboratory (2000) in the United Kingdom has examined the safety effects of signal installation as part of a program to monitor the safety impacts of various actions undertaken by local road authorities. The Transport Research Laboratory (TRL) maintains the information about local road safety improvements in the Monitoring Of Local Authority Safety SchemES (MOLASSES) database. This database has been active since 1991 and contains information on a variety of local road safety improvements.

Data is voluntarily inputted into the database by local road authorities. The data collected include

- *Average daily traffic*
- *Speed limit*
- *Setting (i.e., urban or rural)*
- *Location (type of intersection, type road section, or area-wide)*

- *Pedestrian flow*
- *Safety problem being addressed (target crashes)*
- *Description of the treatment*
- *Number of crashes by severity in the before and after periods*

The manner in which MOLASSES sites are selected for treatment is unknown. Therefore, regression-to-the-mean effects may cause efficacy estimates to be inflated. In addition, the estimates are based on before-after studies of crash frequency. There is a failure to account for changes in exposure or other potential confounding factors.

Limited information is available from the MOLASSES database without sending a tailored request to the TRL. Nonetheless, the safety impacts of new signals in urban areas, and in rural areas are as shown in Table 3.11.

TABLE 3.11: Safety Effects of Signal Installation in the United Kingdom

Setting	Number of Installations	Number of Crashes		CMF
		Before	After	
Urban	26	323	144	0.45
Rural	8	93	20	0.22

Bauer and Harwood (2000)

Bauer and Harwood (2000) developed SPFs for urban, four-leg intersections using three years of crash and infrastructure data from California. Lognormal and loglinear regression was performed on the dataset described in Table 3.12.

TABLE 3.12: Intersection Characteristics for SPFs from California

Characteristic		Stop-controlled	Signal-controlled
No. of intersections		1342	1306
ADT	Major	1100 – 79000	2400 – 79000
	Minor	100 – 16940	101 – 48000
Mean No. of Crashes	All	7.4	23.4
	Fatal+Injury	3.3	9.6

The regression analysis yielded the SPFs in Equations 3.7 and 3.8.

Stop-controlled

$$N = 0.009429\, ADT_{main}^{0.620}\, ADT_{side}^{0.281}\, e^{-0.941X1} e^{-0.097X2} e^{0.401X3} e^{0.120X4} e^{-0.437X5} e^{-0.384X6}\, e^{-0.160X7}\, e^{-0.153X8}\, e^{-0.229X9} \qquad [3.7]$$

where: X1 = 0 if main road left-turns are permitted; 1 otherwise
X2 = Average lane width on main road (metres)
X3 = 1 if the number of lanes on main road is 3 or less; 0 otherwise
X4 = 1 if the number of lanes on main road is 4 or 5; 0 otherwise
X5 = 1 if no access control on main road; 0 otherwise
X6 = 1 if right-turn is NOT free flow from main road; 0 otherwise
X7 = 1 if no illumination; 0 otherwise
X8 = 1 if the main road is a minor arterial; 0 otherwise
X9 = 1 if the main road is a major collector; 0 otherwise

Signal-controlled

$$N = 0.032452\, ADT_{main}^{0.503}\, ADT_{side}^{0.224}\, e^{0.063X1} e^{0.622X2} e^{-0.200X3} e^{-0.310X4} e^{-0.130X5} e^{-0.053X6}\, e^{-0.115X7}\, e^{-0.225X8}\, e^{-0.130X9} \quad [3.8]$$

where: X1 = 1 if pre-timed signal; 0 otherwise
X2 = 1 if fully-actuated signal; 0 otherwise
X3 = 0 if two-phase signal; 1 otherwise
X4 = 1 if no access control on main road; 0 otherwise
X5 = 1 if 3 or less lanes on the side road; 0 otherwise
X6 = Average lane width on main road (metres)
X7 = 0 if no free flow right turn from main road; 1 otherwise
X8 = 1 if 3 or less lanes on the main road; 0 otherwise
X9 = 1 if 4 or 5 lanes on the main road; 0 otherwise

These models can be used to determine the impacts of signalization (or the removal of a signal). Caution should be exercised in relying solely on the prediction models. First of all, it is a better estimate of the long-term safety of a facility if the results of the prediction model can be combined with the actual safety performance (i.e., crash history) of the site. Secondly, there are some aspects of the above models that seem to be counterintuitive. For instance, in the model for stop-controlled intersections, controls on access, and the provision of illumination are seen to be detrimental to safety (CMFs of 1.55 and 1.17, respectively). The conventional wisdom disagrees with these findings and casts some doubt on the usefulness of these equations.

Harwood et al (2000)

Harwood et al (2000) also developed SPFs for cross intersections of two-lane roads in rural settings. The models are as follows:

Stop-control

$$N = \exp(-9.34 + 0.60 \ln ADT_{main} + 0.61 \ln ADT_{side} + 0.13ND - 0.0054SKEW) \quad [3.9]$$

where:
N = annual number of crashes
ADT_{main} = Average daily traffic on the main road
ADT_{side} = Average daily traffic on the minor road
ND = number of driveways on the major road legs within 76 metres of the intersection
SKEW = intersection angle (degrees) expressed as one-half of the angle to the right minus one-half of the angle to the left for the angles between the major road leg in the direction of increasing stations and the right and left legs, respectively.[2]

Signal Control

$$N = \exp(-5.46 + 0.60 \ln ADT_{main} + 0.20 \ln ADT_{side} - 0.40PL - 0.018LT + 0.11V + 0.026T + 0.041ND) \quad [3.10]$$

where:
N = annual number of crashes
ADT_{main} = Average daily traffic on the main road
ADT_{side} = Average daily traffic on the minor road
PL = 0 if no protected left-turn phasing on major road; 1 otherwise
LT = Proportion of minor road traffic turning left during AM and PM peak combined (%)
V = grade rate for all vertical curves within 76 metres of the intersection along the main and minor roads
T = Proportion of trucks entering in the AM and PM peak hours combined (%)
ND = number of driveways on the major road legs within 76 metres of the intersection

The stop-control SPF was developed using data from 324 intersections in Minnesota, with five years of crash data. The signal controlled SPF was developed from 18 intersections in California, and 31 in Michigan, each with three years of crash data available.

[2] In most instances SKEW is computed as the absolute value of the angle of intersection minus 90 degrees.

Region of Durham (2001)

The Region of Durham, Ontario has recently developed SPFs for their facilities and produced SPFs with forms as shown in Equations 3.11 to 3.13, and parameters shown in Table 3.13 and 3.14 for signalized and unsignalized intersections, respectively.

$$N = a\ AADT_{major}^{b1}\ AADT_{minor}^{b2} \qquad [3.11]$$

$$N = a\ (AADT_{major} + AADT_{minor})^{b1} (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{b2} \qquad [3.12]$$

$$N = a\ (AADT_{major} + AADT_{minor})^{b1} \qquad [3.13]$$

where: $AADT_{major}$ = Total entering AADT on major road
$AADT_{minor}$ = Total entering AADT on minor road

Three different equations are presented because these forms best fit the data.

TABLE 3.13: Region of Durham – Signalized Intersections

Type	Environment	Equation	a		b_1	b_2
			Fatal+Injury	PDO		
3-Leg	CBD	3.8	7.71E-2	1.44E-1	0.304	0.157
	Suburban	3.8	8.22E-2	1.39E-1	0.304	0.157
	Rural/Rural Centre	3.8	3.47E-2	6.62E-2	0.304	0.157
	Semi-urban	3.8	7.40E-2	1.81E-1	0.304	0.157
4-Leg	CBD	3.7	1.44E-6	3.24E-6	1.111	0.373
	Suburban	3.9	7.11E-5	1.57E-4	0.997	------
	Rural/Rural Centre	3.9	1.04E-4	1.62E-4	0.977	------
	Semi-urban	3.7	1.26E-6	3.13E-6	1.111	0.373

TABLE 3.14: Region of Durham – Unsignalized Intersections

Type	Environment	Equation	a		b_1	b_2
			Fatal+Injury	PDO		
3-Leg	CBD	3.7	3.42E-6	7.98E-6	1.021	0.219
	Suburban	3.7	6.38E-7	1.56E-6	1.152	0.292
	Rural/Rural Centre	3.7	4.18E-5	9.03E-5	0.598	0.484
	Semi-urban	3.7	2.31E-6	5.39E-6	1.021	0.219
4-Leg	CBD	3.8	3.17E-3	1.20E-2	0.676	0.450
	Suburban	3.8	2.30E-3	4.96E-3	0.676	0.450
	Rural/Rural Centre	3.8	3.25E-3	5.16E-3	0.676	0.450
	Semi-urban	3.8	2.93E-3	6.03E-3	0.676	0.450

Region of Halton (2001)

The Region of Halton, Ontario has also developed SPFs for their signalized and unsignalized intersections, disaggregated by severity. These SPFs can be used to estimate the safety impacts of signalization through comparison. The form of the SPFs are as per Equation 3.14, the parameters are estimated in Table 3.15 and 3.16 for signalized and unsignalized intersections, respectively.

$$N = a\ TOTAL^{b1}\ RATIO^{b2} \qquad [3.14]$$

where:
$TOTAL = AADT_{main} + AADT_{minor}$
$RATIO = AADT_{minor} / TOTAL$
$AADT_{main}$ = Average daily traffic entering from the main road
$AADT_{minor}$ =Average daily traffic entering from the minor road

TABLE 3.15: Region of Halton – Signalized intersections

Type	Environment	a		b_1	b_2
		Fatal+Injury	PDO		
3-leg	All	7.0E-5	2.5E-4	0.934	0.165
4-leg	Urban/Suburban	8.1E-3	2.32E-2	0.591	0.688
	Rural	1.04E-3	3.16E-3	0.581	-0.940

TABLE 3.16: Region of Halton – Unsignalized intersections

Type	Environment	a		b_1	b_2
		Fatal+Injury	PDO		
3-leg	All	2.5E-3	7.32E-3	0.614	0.5253
4-leg	All	7.2E-4	1.64E-3	0.838	0.591

Thomas and Smith (2001)

Thomas and Smith (2001) undertook an examination of the safety impacts of signalization at 16 intersections in various municipalities in Iowa. The site selection process is not described; the study methodology is a naïve before-after analysis using crash frequency and severity. The crash frequency is comprised of three years of before and three years of after data categorized by severity, and several impact types. Some "outliers" were removed from the data set, as they were skewing the results. The results are shown in Table 3.17.

TABLE 3.17: CMFs for Signalization (Thomas and Smith)

Crash Type		Mean	# Sites	90% Confidence Intervals	
				Lower	Upper
Severity	Fatal	N/A	0	N/A	N/A
	Major	0.57	7	1.29	0.00
	Minor	0.92	16	1.42	0.43
	Possible	1.44	13	2.00	0.88
	PDO	0.60	14	0.71	0.48
Impact Type	Right-angle	0.25	15	0.34	0.16
	Rear-end	0.96	12	1.25	0.68
	Left-turn	1.27	12	1.82	0.71
	Other	0.70	15	0.92	0.48
Total		0.73	15	0.93	0.53

It is evident from the results that under a 90% level of confidence, safety benefits can be expected for PDO, right-angle, other, and total crashes. All other crash types have confidence intervals that straddle unity (one CMF on either side of “1”). This indicates that we are unsure if these crash types are positively or negatively impacted by signalization.

Exposure was not accounted for in the analysis, as this data was not readily available.

In the same study Thomas and Smith also evaluated the safety impacts of signalization in conjunction with the addition of turn lane(s). A similar methodology was used to evaluate 11 sites. The results are shown in Table 3.18.

TABLE 3.18: CMFs for Signalization Plus Turn Lane Construction (Thomas and Smith)

Crash Type		Mean	# Sites	90% Confidence Intervals	
				Lower	Upper
Severity	Fatal	0.00	3	N/A	N/A
	Major	0.00	9	N/A	N/A
	Minor	0.34	8	0.45	0.23
	Possible	0.73	11	1.13	0.34
	PDO	0.94	11	1.32	0.57
Impact Type	Right-angle	0.37	11	0.52	0.22
	Rear-end	1.44	11	2.02	0.86
	Left-turn	0.65	11	0.00	0.30
	Other	0.83	11	1.16	0.50
All crashes		0.80	11	1.12	0.49

Region of Waterloo (2001)

As part of an ongoing program the Region of Waterloo, Ontario routinely assesses the street system for locations with an elevated risk for motor vehicle crashes, and implements appropriate countermeasures. The Region of Waterloo (2001) reports that in 1998 three locations were changed from stop control to signal control with results as shown in Table 3.19.

TABLE 3.19: Safety Effects of Signalization in Waterloo, Ontario

Location	Crash Frequency		CMF
	Before	After	
A	7	5	0.71
B	8	3	0.38
C	9	4	0.44
Average	8	4	0.50

The Waterloo analysis is a naïve before-after study of crash frequency using one-year of before and one year of after data. Furthermore, the traffic signals were installed, at least in part, because these locations had an aberrant crash record. The results are very unreliable due to a failure to account regression-to-the-mean, the limited sample size, and the failure to account for exposure.

Pernia et al (2002)

Florida's investigation into the safety impacts of signalization included developing crash prediction models for 447 intersections that were signalized between 1990 and 1997 (Pernia et al, 2002). The models included crashes that occurred within three years of signalization (i.e., three years before, and three years after), and captured all crashes within 76 metres (250 feet) from the point of intersection along the major road. It is recognized that regression-to-the-mean bias may be present in the dataset; it was not addressed in the analysis.

The form of equation used in the analysis was as follows:

$$N = \exp(a + C_1X_1 + C_2X_2 + \ldots + C_nX_n) \qquad [3.15]$$

Where: N = Annual number of crashes
a = constant as shown in Table 3.16
X_n = Independent variables as shown in Table 3.16
C_n = ("value" multiplied by "coefficient") from Table 3.16

The predictor variables, which were selected based on data availability, and engineering judgement, are: traffic volume, land use, location (i.e., business or other), number of lanes, posted speed limit, presence of a median, and shoulder type. SPFs were developed for all, angle, rear-end, left-turn, and other crashes. The results of the analysis on all crashes are shown in Table 3.20.

The SPFs can be used to estimate the safety impacts of signalization at intersections with selected characteristics. However, by selecting intersections that had been signalized through routine, it limits the applicability of the models to those intersections that "warrant" signalization.

TABLE 3.20: Estimated Parameters for Florida SPFs (All Crashes)

Variable	Category	Value	Unsignalized		Signalized	
			Coefficient	CMF	Coefficient	CMF
Alpha			0.6827		0.5718	
ADT	< 15k	0	0.2777	1.32	0.4868	1.63
	15k - 30k	1				
	> 30k	2				
Land Use	Urban	1	0.1193	1.13	0.0949	1.10
	Rural	0				
Location	Business	1	0.1705	1.19	0.1728	1.19
	Other	0				
No. Lanes*	> 4	1	0.2614	1.30	0.2654	1.30
	≤ 4	0				
Posted Speed*	> 45 mph	1	-0.1695	0.84	NS+	NS+
	≤ 45 mph	0				
Median*	Yes	1	0.2752	1.32	0.1845	1.20
	No	0				
Shoulder Treatment	Paved	1	-0.1679	0.85	-0.1102	0.90
	Other	0				

* - ON THE MAJOR ROAD
+ - Not significant at the 80% level

Lyon and Persaud (2002)

In a study to develop SPFs for pedestrian crashes at intersections, Lyon and Persaud (2002) used 11 years of data from sites in Toronto, Ontario to develop SPFs for pedestrian crashes at signalized and unsignalized, three-leg intersections. The characteristics of the intersections are as shown in Table 3.21.

TABLE 3.21: Characteristics of Intersections used to Develop Pedestrian SPFs

Site	No. Sites	Crashes in 11 years		Entering AADT		Ped. Volume (8 hours)	
		Mean	Range	Mean	Range	Mean	Range
Signal	263	4.05	0 - 33	29285	2451-64684	1342	47 – 9811
Unsignal	122	1.30	0 – 10	30099	9352-54046	432	48 - 3131

The SPFs that provided the best fit assumed the form shown in Equation 3.16, and have parameter estimates as shown in Table 3.22.

$$N = ln(a)\ AADT^{b}\ PED^{c}\ (V_L/AADT)^{d} \qquad [3.16]$$

where: AADT = total entering volume (vehicles/day)
PED = 8 hour pedestrian count
V_L = total volume of traffic turning left (vehicles/day)
a, b, c, d = parameters as shown in Table 3.19

TABLE 3.22: Parameters for SPFs for Pedestrian Intersection Crashes

Intersection Type	Parameter				Overdispersion Factor (k)
	a	b	c	d	
Signal	-8.18	0.399	0.412	2.841	1.7
Stop	-9.82	---	0.662	0.531	3.7

All-way Stop

Lovell and Hauer (1986)

Lovell and Hauer (1986) have conducted the most thorough analysis on the safety effects of conversion from two-way to all-way stop control. Although the research is somewhat dated, it remains the best available effort at determining the effectiveness of all-way stop control. The study included a re-analysis of data from San Francisco, Philadelphia, Michigan, and Toronto using a before-after analysis and controlling for regression-to-the-mean through the use of likelihood functions.

The data were primarily from urban intersections, although the Michigan data were from rural locations. The results of the analysis are shown in Table 3.23. The combined results are impressive showing a safety benefit to all types of crashes.

TABLE 3.23: CMFs for Conversion to All-way Stop

Crash Type	SF	Philly	Michigan	Toronto	Combined
No. Sites	49	222	10	79	360
Right-angle	0.16	0.22	0.36	0.52	0.28
Rear-end	3.05	0.80	0.81	0.78	0.87
Left-turn	0.67	---	1.07	0.75	0.80
Pedestrian	0.34	0.60	---	0.58	0.61
Fixed object	---	1.30	---	---	---
Injury	0.26	0.26	0.38	0.37	0.29
Total	0.38	0.53	0.41	0.63	0.53

Laplante and Kropidlowski (1992)

Laplante and Kropidlowski (1992) studied the safety impacts of all-direction stop control on arterial roads in Chicago. Thirty intersections were studied; 16 of these locations met the warrants for all-direction stop control, the remainder were unwarranted. Traffic volumes on the arterial streets ranged from 3,000 to 23,000 vehicles per day. The side streets were local streets with daily volumes of less than 3,000 vehicles.

The study methodology is a before-after analysis of crash frequency using three years of before, and three years of after data. In addition, the researchers gathered an additional three years of data from 10 years after the initial installation to determine if the safety impacts varied over time. The results are shown in Table 3.24.

TABLE 3.24: CMFs for All-direction Stop Control in Chicago

All-way STOP Type	Number of Sites	Crash Type	Crash Frequency (crashes/year)			CMF	
			Before	After	10 years After	Immediate	10 year
Warranted	16	All	11.0	4.6	3.2	0.42	0.29
		Angle	6.8	1.5	1.4	0.22	0.21
Unwarranted	14	All	3.4	4.2	2.2	1.24	0.65
		Angle	1.4	0.8	0.8	0.57	0.57
Unwarranted ADT < 12,000	3	All	3.2	1.4	0.8	0.44	0.25
		Angle	1.0	0.5	0.3	0.50	0.30
Unwarranted ADT > 12,000	3	All	4.1	10.8	7.0	2.63	1.71
		Angle	1.7	2.2	2.2	1.29	1.29

The Laplante and Kropidlowski study results indicate that warranted all-direction stop control are effective at reducing angle and total crashes. The unwarranted all-direction stops indicate some safety benefits (in both the short and long-terms). However, upon further analysis by the researchers it appears that the safety benefits of unwarranted all-direction stop control are realized at sites where the arterial traffic was less than 12,000 vehicles per day. Whereas, at the unwarranted all-direction stops on the more heavily travelled arterials the angle and total crashes increased.

There is no mention in the study of site selection methodology. Nonetheless, the data appear to indicate a regression-to-the-mean bias. The warranted all-way stops have an average crash frequencies that is over three times that of the unwarranted locations. In addition, the sample sizes are relatively small and the lack of statistical testing draws into question the reliability of the results.

Harwood et al (2000)

Harwood et al (2000) using an expert panel, considered the evidence available concerning all-way stop control at the intersection of rural, two-lane highways and determined that the CMF for conversion from minor road stop-control to all-way stop control is 0.53. The authors caution that this formidable CMF may only be applicable to intersections that warrant, or are close to warranting an all-way stop. This CMF is identical to that developed by Lovell and Hauer in their 1986 study; it is likely that Harwood et al considered the 1986 study to be the best available evidence, and elected to adopt the results directly.

System Intersection Control

Main (1984)

With respect to unsignalized intersections on local and collector streets in residential neighbourhoods, Main (1984) investigated the safety effects of the system. Envisioned as a traffic management tool for grid street systems in the older part of the urban area, the City of Hamilton adjusted and implemented stop control within a neighbourhood to develop a regular pattern of stop control. The basic principles upon which the pattern were based, are that:

- all intersections with four approaches should be stop-controlled; and
- motorists should be required to stop on the local/collector grid system at two block intervals.

Revisions to the system were minimized by conforming as much as possible to the existing pattern of stop control. Variations from the two-block stop guideline were permitted if visibility obstructions or other conditions necessitated a change. The program included the replacement of some yield controlled intersections with stop control.

Nine residential areas where the stop-control strategy was implemented were studied. Collision frequency using a naïve before-after analysis was the study methodology. Three years of before, and three years of after data were used in the analysis, and the results show an impressive CMF of 0.76 (see Table 3.25).

TABLE 3.25: CMFs for Regular Patterns of Stop Control

Location	Collisions		Change	CMF
	Before	After		
1	155	91	-41.3	0.59
2	200	183	-8.5	0.91
3	198	129	-34.8	0.65
4	93	74	-20.4	0.80
5	57	62	+8.8	1.09
6	44	26	-40.9	0.59
7	27	26	-3.7	0.96
8	21	23	+9.5	1.10
9	26	12	-53.8	0.46
Total	821	626	-23.8	0.76

The main shortcoming of this analysis is the failure to account for exposure. The regular pattern of stop signs was admittedly implemented as a traffic management tool. One of the desired benefits was a reduction in traffic volume by rerouting "through" traffic to the surrounding arterial street system. Any success at rerouting traffic would bring about a subsequent decrease in collision frequency. Whether the risk of collision was subsequently reduced cannot be ascertained from the information contained in the documentation.

Laplante and Kropidlowski (1992)

In a similar study Laplante and Kropidlowski (1992) examined the safety impacts of a regular pattern of stop signs at nine low-volume intersections in a neighbourhood of Chicago. None of the intersections met the United States warrants for stop control but were converted from uncontrolled to two-way stop controlled intersections. Again, the pattern of stop sign placement was such that a motorist could travel no more than two blocks without encountering a stop sign.

A naïve before-after study using crash frequency was the methodology employed. Three year of before and three years of after data were available. The average crash frequency decreased from 21.3 crashes/intersection/year to 2.6 crashes/intersection/year (CMF = 0.12). The safety benefits were achieved despite traffic volumes in the study area increasing an average of 11%. The researchers re-examined the crash frequency 13 years after stop sign installation and found the crash frequency had increased to 3.3 crashes/intersection/year (CMF = 0.15). The traffic volumes at this time are not reported.

Laplante and Kropidlowski also were concerned about crash migration to the streets surrounding the treated neighbourhood. An examination of the crash frequency at the 10 stop controlled intersections on the four peripheral streets (arterials or collectors) decreased from 27.9 crashes/intersection/year to 19.7 crashes/intersection/year. Again traffic volume changes are not reported. However, the analysts make note that the total number of crashes in Chicago increased during the study period.

The results of the Laplante and Kropidlowski study are questionable based on a likely regression-to-the-mean bias. Site selection is not detailed in the documentation. However, a crash frequency of 21.3 crashes/year at a low-volume intersection is certainly considered abnormally high. It would appear that these intersections were selected based on their high crash frequency. Moreover, the sample size (nine intersections) is very small and a lack of statistical analysis does not provide the reader with any information on the variability of the results.

Intersection Control Beacons

Pant et al (1999)

Pant et al (1999) undertook a cross-section, and before-after study of six stop-controlled intersections and seven stop-controlled intersections that were supplemented with beacons. The sites were rural intersections located in Ohio. The study sites were selected because of the availability of complete traffic and crash data for four years but were matched on the following geometrics:

- The angle of intersection was approximately 90 degrees;
- All legs had a single approach lane;
- All intersections were located in a rural area and had no substantial development around the intersection; and
- The posted speed limit was 55 mph in the major direction.

The researchers speculate that the intersections that are supplemented with beacons were done so because of an abnormal crash record.

The results of the cross-section study are shown in Table 3.26.

TABLE 3.26: Mean Crash Rates at Stop- and Stop+Beacon Controlled Intersections

Intersection	No. of Sites	Sight Distance	Crash Rate (crashes/10,000 vehicles)			
			Fatal	Injury	PDO	Right Angle
Stop+Beacon	3	Adequate	0.43	3.71	3.00	3.86
Stop	4		0.48	3.25	2.67	4.05
Stop+Beacon	4	Inadequate	0.21	4.46	3.41	6.37
Stop	2		0.43	3.79	3.79	3.99

The data is sparse and definitive conclusions cannot be drawn. Nonetheless, it appears that the addition of beacons to stop controlled intersections in rural areas actually increases the overall and casualty crash rates.

A naïve before-after evaluation was conducted on the seven stop+beacon controlled intersections using two to three years of before and after data. No significant differences were found between the before and after crash frequencies (95% level of confidence).

Traffic Signal Design and Operation

Tople (1998)

Tople (1998) in an evaluation of hazard elimination and safety projects included a review of the safety benefits of traffic signal upgrading at five locations in South Dakota. The specific intervention implemented was not specified. The evaluation was a naïve before-after study of crash frequency and crash severity. The impact on crash severity was determined through a comparison of equivalent property damage only crashes, using monetary conversations deemed appropriate by the investigation team. Three years of before and three years of after crash data was used in the analysis. The actual treatment was not specified.

The results of the analysis are presented in Table 3.27.

The Tople analysis possesses many potentially serious flaws. Most importantly, the sites were selected for treatment as part of a safety program. This means that the crash record was likely abnormally high, and there is a great potential for regression to the mean artefacts. This shortcoming is likely offset somewhat by a failure to account for changes in exposure. Traffic volumes were not controlled for, but typically volumes tend to increase which would lead to a higher "after" count of crashes. In the end, the South Dakota results are based on a limited number of sites and weak analyses.

TABLE 3.27: Safety Impacts of Signal Upgrades in South Dakota

Improvement Type	No. of Sites	AADT Range	Crashes			EPDO Crashes*		
			Before	After	CMF	Before	After	CMF
Signal Upgrading	6	5085 - 28200	272	180	0.66	4673	2635.5	0.56

* EPDO crashes were calculated as (1300*F)+(90*I)+(18*N)+(9.5*P)+PDO
where: F = fatal crash
I = incapacitating injury crash
N = non-incapacitating injury crash
P = possible injury crash
PDO = Property damage only crash

Transport Research Laboratory (2000)

The TRL (2000) of the United Kingdom has collected information on the safety impacts of signal modifications through the MOLASSES database (see the section on "Signalization" for more information on MOLASSES). The definition of "signal modification" is not provided, so the results are generalized and are only adequate to provide cursory guidance on the magnitude of the potential for safety improvement. The results are shown in Table 3.28.

TABLE 3.28: Safety Effects of Signal Modifications in the United Kingdom

Setting	Number of Locations	Number of Crashes		CMF
		Before	After	
Urban	80	1130	697	0.62
Rural	10	135	66	0.49

Signal Conspicuity

Cottrell (1995)

The use of white strobe lights as a supplement to the red signal indication at six traffic signals in Virginia was evaluated by Cottrell (1995). The strobe light was a horizontal bar pattern that was placed concentric with the red lens of the signal lens. Three years of before, and three years of after crash data were used in a naïve before-after analysis of crash frequency.

The characteristics of the study intersections are displayed in Table 3.29. The results of the crash analysis are shown in Table 3.30.

TABLE 3.29: Study Site Characteristics for White Strobes to Supplement Red Signal Indications

Site	No. Strobes/ approach	No. Approaches	Major Road		Minor Road	
			Speed limit (mph)	ADT	Speed limit (mph)	ADT
1	1	4	45	11000	45	6300
2	1	4	55	21000	55	11000
3	2	4	40	9000	40	4400
4	2	3	45	14200	45	1100
5	2	4	55	9400	45	2500
6	1	4	45	11000	45	2400

TABLE 3.30: Crash Results for White Strobes to Supplement Red Signal Indications

Site	Rear-end			Angle			Total		
	Before	After	CMF	Before	After	CMF	Before	After	CMF
1	4	4	1.00	13	8	0.62	19	20	0.95
2	2	7	3.50	1	5	5.00	3	15	5.00
3	4	4	1.00	8	3	0.37	13	7	0.54
4	6	6	1.00	4	1	0.25	12	9	0.75
5	0	0	1.00	1	1	1.00	2	2	1.00
6	3	6	2.00	12	15	1.25	15	28	1.87
All	19	27	1.42	39	33	0.85	64	81	1.27

Although the results of the analysis indicate that the strobe lights are likely detrimental to overall intersection safety, no significant conclusions can be drawn because of several flaws in the study design. First of all, the sites were not selected at random, there is a likely regression-to-the-mean effect, which would suggest that the strobe lights were actually more detrimental than the results would indicate. Secondly, the lack of a comparison group means that other factors that may have influenced crash frequency were not accounted for. Thirdly, exposure was not accounted for, and increased traffic volumes would certainly play a role in increasing crash frequency. Lastly, the strobes can only be expected to affect crash occurrence while they are flashing (i.e., during the red phase). Reviewing all crashes, including those when the signal indication is green would confound the results.

Sayed et al (1998)

Sayed et al (1998) researched the safety impacts of a revised signal head configuration at 10 intersections in British Columbia. The existing standard signal head (and the "before" condition) is the traditional red, yellow, green signal arranged vertically, with 300mm, 200mm, and 200mm lenses, respectively. The standard installation also included a yellow backboard. The treatment was to increase the size of the yellow and green lenses to 300mm each, and to place a 50mm reflective border on the backboard.

The intersections under study were four-leg intersections with left-turn channelization, and three or four lanes on each approach. Primary signal heads were mast-arm mounted, while the secondary heads were post-mounted. The visibility was considered adequate for the posted speed limit, and the setting was an urban, commercial/retail area.

The study methodology was a before-after study with a control group, using Empirical Bayes techniques to account for regression-to-the-mean. Crash frequency and severity were the measures of effectiveness, with one year of before, and two years of after data being available. The results are shown in Table 3.31.

TABLE 3.31: Safety Impacts of Alternative Signal Head Design in BC

Site	CMF	
	All Crashes	Injury+Fatal
1	0.57	0.74
2	0.55	0.68
3	0.66	0.89
4	0.90	0.88
5	0.59	0.54
6	1.10	1.29
7	1.29	1.77
8	0.58	0.90
9	1.05	0.89
10	0.62	0.46
Average	0.79	0.91

Transportation Association of Canada (2001)

Phase 2 of this study (TAC, 2001) included an investigation of the safety impacts of diamond-grade[TM] yellow reflective tape on the signal head backboard. The tape was placed on the outside edge of the backboard and was 75 mm wide. Six intersections in British Columbia were outfitted with the new heads and studied for a one-year before, and a three-year after period. The safety analysis examined only night-time crashes. Results are as shown in Table 3.32.

[TM] Diamond-grade is a trademark of 3M Company.

TABLE 3.32: Night-time Crash Frequency at Locations with Modified Signal Heads in BC

Crash type	Before	After		
		Year 1	Year 2	Year 3
Angle	1	3	0	1
Left turn	3	1	1	0
Right turn	1	1	0	0
Rear end	7	2	3	0
Overtaking	0	1	0	0
Off road	0	1	1	1
Unknown	2	5	0	1
Total	14	14	5	3

Traffic volumes were not used in the analysis, but showed an average increase of 2% per year. The researchers note that the sample size is small, and the analysis methodology is suspect. Nonetheless, there are apparently overall benefits, particularly in reducing rear-end crashes.

Region of Waterloo (2001)

As part of an ongoing program the Region of Waterloo, Ontario routinely assesses the street system for locations with an elevated risk for motor vehicle crashes, and implements appropriate countermeasures. The Region of Waterloo (2001) reports that in 1998 two locations were provided with new signal heads and revised signal timings to improve safety. The results as shown in Table 3.33.

TABLE 3.33: Safety Effects of New Signal Heads in Waterloo, Ontario

Location	Crash Frequency		CMF
	Before	After	
A*	24	12	0.50
B	34	13	0.38
Average	29	12.5	0.43

* - Site A also had a right-turn lane added to one approach

The Waterloo analysis is a naïve before-after study of crash frequency using one-year of before and one year of after data. It is impossible to separate out the effects that may be attributed to the new signal heads, from the effects that may be attributed to the revised signal timing. Furthermore, the changes were made, at least in part, because these locations had an aberrant crash record. The results are very unreliable due to a failure to account regression-to-the-mean, the limited sample size, and the failure to account for exposure.

Signal Head Location

Bhesania (1991)

Bhesania (1991) examined the safety impacts of replacing post-mounted signal heads with mast-mounted signal heads at five locations in Kansas City, Missouri. The treatment also included the addition of a one-second all-red interval for both through phases. No information is provided on how the sites were selected for analysis. The study uses a naïve before-after analysis using collision frequency during 12 month before and after periods. The author notes that the intersection traffic volumes "remained fairly constant" during the before and after periods, and that no other treatments were implemented.

The results of Bhesania's analysis are shown in Table 3.34. It is impossible to separate out the effects of the change in signal head position and the effects of the all-red interval. The combined effect is a CMF of about 0.75.

TABLE 3.34: Safety Impacts of Signal Head Location in Kansas City

Crash type	Crash frequency		CMF
	Before	After	
Right-angle	65	24	0.37
Rear-end	37	30	0.81
Left-turn	37	50	1.35
Other	22	16	0.73
Total	161	120	0.75

Thomas and Smith (2001)

Thomas and Smith (2001) undertook an examination of the safety impacts of replacing pedestal-mounted signals with mast arm mounted signals at 33 intersections in Iowa. The site selection process is not described; the study methodology is a naïve before-after analysis using crash frequency and severity. The crash frequency is comprised of three years of before and three years of after data categorized by severity, and several impact types. The results are shown in Table 3.35, some outliers have been removed from the dataset.

The results indicate that under a 90% degree of confidence, safety benefits can be expected for total crashes

TABLE 3.35: CMFs for Replacing Pedestal-Mounted Signal Heads with Mast Arm-Mounted Signal Heads (Thomas and Smith)

Crash Type		Mean	# Sites	90% Confidence Intervals	
				Lower	Upper
Severity	Fatal	0.00	1	N/A	N/A
	Major	0.53	17	0.84	0.23
	Minor	0.87	30	1.11	0.62
	Possible	1.12	31	1.41	0.84
	PDO	0.60	32	0.68	0.52
Impact Type	Right-angle	0.28	31	0.35	0.21
	Rear-end	1.20	32	1.51	0.90
	Left-turn	1.02	24	1.23	0.81
	Other	0.73	31	0.82	0.64
Total		0.64	31	0.72	0.57

Additional Primary Signal Heads

Hamilton Associates (1998)

Hamilton Associates (1998) undertook an evaluation of the safety impacts of providing a second primary signal head at intersections in the Lower Mainland of British Columbia. The second primary head is mounted on the far right side of the intersection and is differentiated from a tertiary signal head by being located above the intersection. The study methodology included a cross-section study of crash rates, and a before-after study using Empirical Bayes

The cross-sectional study compared the crash rates at 63 signalized intersections that were matched on the following criteria:

- Urban area;
- Four intersection approaches; and
- Two or more through lanes on each approach

Forty-eight of the study intersections had one primary signal head per approach; 15 of the study intersections had two primary signal heads per approach. Crash rates were computed using available crash data, the mean periods were 3.8 years and 2.6 years for primary and two primary head intersections, respectively. The results of the cross-section analysis are shown in Table 3.36.

The differences in total and PDO crash rates for two types of intersections are statistically significant to a 90% confidence level.

TABLE 3.36: Safety Impacts of An Additional Primary Signal Head

Intersection Type	Crash Frequency (/intersection/yr)	Crash Rate (/MVE)		
		Total	Fatal & Injury	PDO
One Primary Head	23.1	1.30	0.44	0.86
Two Primary Heads	19.7	1.02	0.40	0.62
CMF		0.78	0.91	0.72

In the before-after analysis, eight intersections in Richmond, British Columbia that are outfitted with additional 30/20/20 primary signal heads were studied. One or two year before and after periods was employed depending on date of installation and availability of data. The researchers employed Empirical Bayes, and Multi-variate Empirical Bayes methods to assess the effects of the second primary signal head. The results are shown in Table 3.37.

TABLE 3.37: CMFs for Additional Primary Signal Heads

Statistical Method	CMF		
	Total Crashes	Fatal & Injury Crashes	PDO Crashes
EB	0.78	0.79	0.64
Multi-variate EB	0.72	0.83	0.69

The results of the cross-section and before-after studies are consistent and indicate a likely reduction in total crashes of 20 to 30%, and a reduction in casualty crashes of 10 to 20%.

Lens Size

Polanis (1998)

Polanis (1998) reviewed the safety impacts of replacing eight-inch traffic signal lenses with 12-inch lenses at 38 locations in Winston-Salem, NC. None of the locations warranted the larger lenses, as determined by the MUTCD; sites were selected on the basis of a pattern of crashes that could be remedied. The study uses a naïve before-after study of collision frequency with no accounting for exposure.

It is noted that 11 of the 38 sites had multiple interventions and therefore any safety impact cannot be attributed to the signal lens alone. The results of the Polanis study, minus the sites with multiple interventions, are presented in Table 3.38.

TABLE 3.38: Safety Impacts of 12 inch Signal Lenses

Site	Target Crashes			Total Crashes		
	Before	After	Change*	Before	After	Change*
1	4	0	-100	11	7	-35
2	14	1	-93	31	13	-58
3	20	2	-92	43	25	-53
4	8	1	-87	48	35	-22
5	12	2	-83	33	20	-39
6	10	2	-80	24	9	-63
7	15	3	-80	43	38	-12
8	8	2	-75	24	20	-17
9	17	4	-74	30	21	-21
10	4	1	-71	14	21	+71
11	11	4	-58	28	12	-51
12	12	5	-58	12	6	-50
13	9	3	-57	35	23	-15
14	15	7	-53	49	38	-22
15	15	8	-47	29	38	+31
16	14	8	-45	40	33	-21
17	16	9	-44	26	23	-12
18	12	7	-42	26	13	-50
19	17	12	-31	28	25	-13
20	11	8	-27	15	17	+13
21	8	6	-25	20	13	-35
22	10	8	-20	19	18	-5
23	9	8	-11	23	23	0
24	25	23	-8	41	52	+27
25	14	13	-7	22	22	0
26	8	9	+13	13	14	+8
27	8	13	+63	23	27	+17

* Change is based on crashes/month due to unequal before and after periods.

The average reduction in target (angle) crashes is 48% or a CMF of 0.52; the average reduction in all crashes is 16% or a CMF of 0.84. Exposure was not accounted for in the analysis. Assuming that traffic volumes either remained constant or increased (as tends to be the case) the CMFs are conservative. However, it cannot be determined if the site selection process introduced any bias into the study results.

Advance Warning Flashers

Gibby et al (1992)

Gibby et al (1992) undertook research into the characteristics of approaches to high-speed, isolated, signalized intersections at 40 locations in California. This analysis included the evaluation of advance warning signs and flashers to these signalized intersections. Ten years of crash data were used, sites were representative of the most and least safe intersections of this type in the California state highway system. The results of the Gibby et al analysis are applicable to locations that are rural, have at least one approach with a posted speed limit of 50 mph or greater, and at least one approach is a state highway.

The advance warning flasher (AWF) was classified as an advance warning sign (AWS) such as a "signal ahead" sign that is supplemented by at least one 300 mm flashing amber beacon. The difference in mean crash rates at the different locations were analysed, and are shown in Table 3.39.

TABLE 3.39: Crash Rates for AWS and AWF in California

Treatment	Number of Approaches	Mean Approach Crash Rate	Standard Deviation
None	14	0.84	0.48
AWS	85	2.83	3.10
AWF	77	1.13	1.14
Both AWS + AWF	14	1.57	1.17

The results of the analysis indicate that the installation of AWSs and AWFs at isolated, high-speed signalized intersections increase crash rates. This conclusion seems to be counterintuitive and is likely untrustworthy because of the following two (main) shortcomings in the study design:

- *The study uses a cross-section rather than a before-after study design to examine the differences in mean crash rates. This type of analysis is not as reliable in controlling for confounding influences between intersections. It is likely that other differences between the study intersections have played some role in shaping the different crash rates.*

- *The allocation of sites to the different treatment groups was likely based on safety performance, and has therefore tainted the analysis. Those sites that have not been treated are likely those with the best safety performance.*

Sayed et al (1999)

Sayed et al (1999) examined the safety of providing advance warning flashers (AWFs) at signalized intersections in British Columbia. A total of 106 intersections were used in the analysis; 25 of which were equipped with AWFs. The treatment consists of a rectangular warning sign that is equipped with two amber beacons mounted on either side, that operate in alternate flashing mode. The signs are illuminated and erected overhead, but positioned over the shoulder. The location of the sign and the onset of flashing operation are in accordance with generally accepted Canadian practice.

The study methodology included the development of SPFs for intersections with and without AWFs to determine the safety impacts of these devices. The use of properly developed SPFs accounts for regression-to-the-mean. The SPFs were developed and used in three different ways to evaluate the safety impacts (i.e., the form of the SPF was constant among all situations but the researchers experimented with model development by varying or holding constant certain parameters between SPFs). The results are shown in Table 3.40.

TABLE 3.40: Safety Impacts of AWFs in British Columbia

Method	CMF		
	Total	Injury+Fatal	Rear-end
1	0.92	0.91	1.03
2	0.88	0.86	0.97
3	0.82	0.86	0.92

All three methods show remarkably similar results. Methods 2 and 3 did not produce statistically significant results at the 95% level of confidence. Upon further investigation, Sayed et al determined that the safety impacts of the AWFs were associated with the minor street volume at signalized locations. The results of this more in-depth investigation are shown in Figure 3.1. The results indicate that when minor street volumes are relatively low, and main street volumes are high, the AWFs actually degrade safety. A minor street volume of about 13,000 vehicles per day is required before the AWFs provide a safety benefit for all major street volumes.

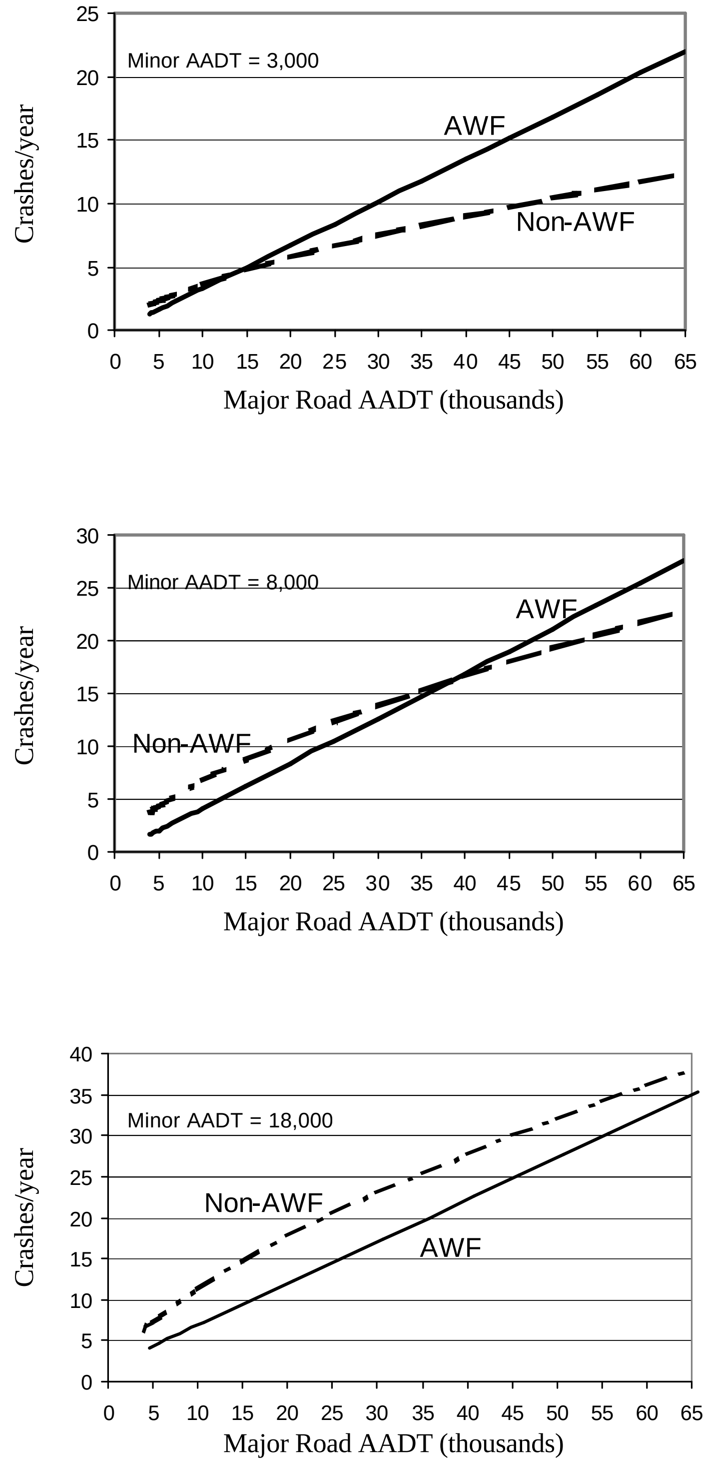


FIGURE 3.1: Advance Warning Flashers

Signal Clearance Timing

Lalani (1991)

The City of San Buenaventura, California as part of a Comprehensive Safety Program improved the clearance timing at three signalized intersections (Lalani, 1991). Sites were selected because they were considered high crash locations. The analysis was a naïve before-after analysis using crash frequency and one-year before and after periods. Details of the treatment are not reported (i.e., what was the original clearance timing? And what was the exact countermeasure?). Nonetheless, the safety impacts of the improved signal timing are shown in Table 3.41.

TABLE 3.41: Safety Impacts of Improved Clearance Timing in California

Location	Crashes		CMF
	Before	After	
A	17	7	0.41
B	7	4	0.57
C	10	6	0.60
Totals	34	17	0.50

Lalani does not account for exposure in the safety analysis but reports that traffic volumes in the city increase at an average rate of 6% per annum.

Signal Coordination

Lalani (1999)

The City of San Buenaventura, California as part of a Comprehensive Safety Program introduced coordination to at least three areas of the city (Lalani, 1991). Sites were selected because they were considered high crash locations. The analysis was a naïve before-after analysis using crash frequency and one-year before and after periods. Details of the treatment are not reported. Nonetheless, the safety impacts of the improved signal coordination are shown in Table 3.42.

TABLE 3.42: Safety Impacts of Signal Coordination in California

Location	Crashes		CMF
	Before	After	
A	179	129	0.72
B	16	12	0.75
C	129	103	0.80
Totals	324	244	0.75

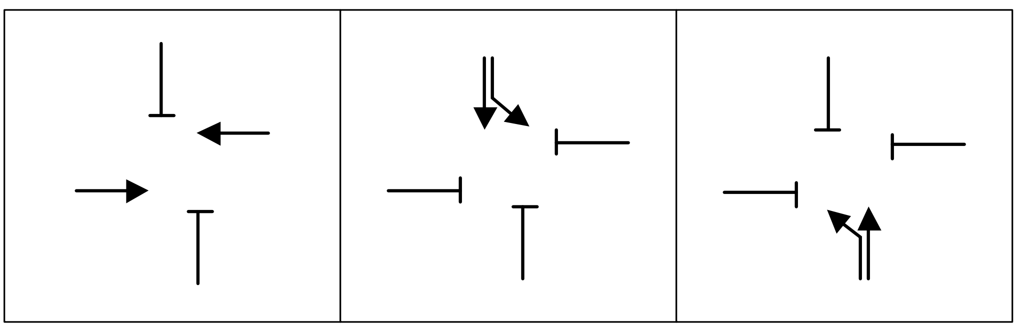
Lalani does not account for exposure in the safety analysis but reports that traffic volumes in the city increase at an average rate of 6% per annum.

Traffic Signal Timing

Greiwe (1986)

The City of Indianapolis, Indiana evaluated the safety impacts of converting two-phase signal operation to a split-phase left-turn sequence as shown in Figure 3.2 (Greiwe, 1986). No indication is given as to the site selection process. The study methodology included a naïve before-after study of crash frequency with two-years of before, and a minimum of one-year of after data. Target crashes were left-turn, right-angle, and rear-end crashes. It is noted by the author that traffic volumes remained nearly the same throughout the study period, so adjustments for exposure were not required.

FIGURE 3.2: Split Phasing for Left-turns



The results of the analysis are shown in Table 3.43.

TABLE 3.43: Safety Impacts of Split Phasing for Left-turns

Site	Before					After				
	LT	RA	RE	Other	All	LT	RA	RE	Other	All
1	4	1	2	3	10	1	1	1	1	4
2	16	3	1	2	22	2	1	2	1	6
3	16	3	11	1	31	5	2	6	3	16
4	4	0	0	2	6	0	0	0	1	1
5	5	1	6	1	13	2	1	5	0	8
6	9	2	2	3	16	2	0	2	1	5
7	4	2	1	0	7	0	2	0	2	4
8	19	0	6	5	30	5	1	6	6	18
Total	77	12	29	17	135	17	8	22	15	62

The CMFs are 0.22, 0.67, 0.76, and 0.46 for left-turn, right-angle, rear-end, and all crashes, respectively.

In the same study, Greiwe (1986) examined the safety impacts of removing protected left-turn phasing that was deemed unwarranted. The Federal Highway Administration MUTCD, and the Guidelines for Signalized Left Turn Treatments were used to assess whether the phasing was warranted.

The results of the analysis are shown in Table 3.44.

TABLE 3.44: Safety Impacts of Removing Unwarranted Protected Left-turn Phasing

Site	Before					After				
	LT	RA	RE	Other	All	LT	RA	RE	Other	All
1	1	1	2	0	4	1	0	1	2	4
2	0	7	4	0	11	3	4	2	2	11
3	0	3	0	3	6	1	2	0	1	4
4	1	2	1	1	5	1	2	1	3	7
5	0	1	2	0	3	0	2	2	3	7
6	3	2	0	0	5	2	0	0	0	2
7	10	1	5	4	20	5	2	3	2	10
8	1	2	2	0	5	1	1	1	0	3
9	2	3	3	2	10	5	4	2	1	12
10	0	0	0	2	2	0	0	0	1	1
11	0	1	0	1	2	1	0	1	0	2
12	2	0	3	0	5	4	1	2	0	7
13	1	0	0	3	4	2	0	2	2	6
14	1	2	3	2	8	11	2	3	2	18
Total	21	23	22	16	82	26	18	17	17	78

The results are not promising. Three sites exhibited no change in total crashes, five sites exhibited a decrease in total crashes, and six sites exhibited an increase in total crashes. Greiwe attempted to explain the somewhat mixed results by correlating the safety impacts with traffic volume, and the presence of a left-turn lane (see Table 3.45). The intersections with volumes of less than 20,000 appear to have no change or a slight safety benefit associated with the removal of the protected left-turn phase, regardless of the presence of the left-turn lane. Intersections with traffic volumes in excess of 20,000 appear to have an adverse safety impact by the removal of the left-turn phase.

TABLE 3.45: Safety Impacts of Left-turn Phasing Correlated with Traffic Volume and the Presence of a Left-turn Lane

Volume	LTL	Crashes		Change
		Before	After	
13520	N	2	2	No change
14375	N	4	4	No change
14589	N	2	1	Decrease
31797	N	8	18	Increase
10691	Y	5	3	Decrease
12216	Y	5	2	Decrease
12270	Y	4	6	Increase
16384	Y	6	4	Decrease
22694	Y	3	7	Increase
23323	Y	5	7	Increase
25697	Y	11	11	No change
30539	Y	20	10	Decrease
36196	Y	10	12	Increase

Hummer et al (1991)

Hummer et al (1991) in developing guidelines for leading and lagging left-turn phasing in Indianapolis examined crash data from 14 intersection approaches with lagging left-turn phasing, and 15 approaches with leading left-turn phasing. Almost all of the study locations were the intersection of a two-way street with a one-way street located in the downtown area. All locations were fixed-time signals. Four years of crash data were used to identify crashes involving a vehicle turning left from the approach of interest (i.e., target crashes). The target crashes were coupled with traffic volumes to determine applicable crash rates for comparison. The results of the comparison are shown in Table 3.46.

TABLE 3.46: Crash Rates for Leading and Lagging Left-turn Phasing

Statistic	Lagging	Leading
No. of approaches	14	15
No. of Left-turn crashes	44	69
Crash rate per 10^6 left-turns	0.8	0.9
Crash rate per MVE	0.06	0.09

The results indicate that lagging left-turn phasing is slightly safer than leading phasing. However, the relatively small sample size is insufficient to draw any definitive conclusions. It should be noted that a more accurate determination of relative safety may have been achieved had the crash rate been calculated using the product of left-turn volume and opposing volume as the measure of exposure. Nonetheless, Hummer et al also examined the distribution of crash severity at each type of left-turn phasing. They found that 35% of the crashes at the leading left-turn approaches were personal injury crashes. In contrast, only 7% of the crashes at the lagging left-turn approaches were personal injury crashes. The difference is significant at a 0.05 level of significance.

Upchurch (1991)

Upchurch (1991) using crash data from several signalized intersection approaches in Arizona, examined the safety performance of five different types of left-turn phasing; permissive, leading exclusive-permissive, lagging exclusive-permissive, leading exclusive only, and lagging exclusive only. Using a cross-sectional study design with left-turn crash rate as the metric, the results shown in Table 3.47 were recorded.

TABLE 3.47: Crash Rates for Left-turn Phasing in Arizona

Left-turn Phasing	Two Opposing Lanes			Three Opposing Lanes		
	No. of Sites	Mean Crash Rate*	CMF	No. of Sites	Mean Crash Rate	CMF
Permissive	162	2.62	---	25	3.83	---
Leading Exclusive-permissive	62	2.71	1.03	52	4.54	1.19
Lagging Exclusive-permissive	44	3.02	1.15	35	2.65	0.69
Leading Exclusive	57	1.02	0.39	80	1.33	0.35
Lagging Exclusive	4	2.09	0.80	2	0.55	0.14

* Left-turn crashes per million left-turning vehicles

The results of the cross-section study indicate that the exclusive phasing, either leading or lagging, demonstrate a safety benefit over permissive-only phasing. The results also seem to indicate that the exclusive-permissive phasing may be detrimental to safety. However, these results are subject to the usual cautions associated with a cross-sectional analysis and may not be reliable. Moreover, the crash rate does not use opposing traffic volume in the measure of exposure.

Upchurch supplements the above analysis with a before-after study from 194 intersection approaches as shown in Table 3.48.

TABLE 3.48: Safety Effects of Changing Left-turn Phasing In Arizona

Treatment	No. Sites	Left-turn Crash Rate		CMF
		Before	After	
Two Opposing Lanes				
P → Lead E/P	17	4.77	3.49	0.73
P → Lag E/P	9	5.44	4.16	0.76
Lead E/P → P	14	2.07	2.66	1.29
Lead E/P → Lag E/P	35	3.10	2.25	0.73
Lead E → Lead E/P	3	0.93	3.11	3.34
Lead E → Lag E/P	6	0.38	1.57	4.13
Lead E → Lag E	10	1.46	1.91	1.31
Three Opposing Lanes				
P → Lead E/P	3	4.64	5.55	1.20
P → Lag E/P	8	8.75	1.37	0.16
P → Lead E	3	18.96	0.36	0.02
Lead E/P → P	3	2.25	5.85	2.60
Lead E/P → Lag E/P	38	4.54	2.74	0.60
Lead E/P → Lead E	2	7.08	0.75	0.11
Lead E → Lead E/P	22	1.40	4.72	3.37
Lead E → Lag E/P	9	2.13	1.03	0.48
Lead E → Lag E	12	0.35	0.35	1.00

The CMFs from the Arizona study are largely consistent with intuition:

- *Changing from a permissive movement to a more restrictive movement (either exclusive-permissive or exclusive) is associated with a safety benefit;*
- *Changing from an exclusive-permissive movement to an exclusive movement is generally beneficial from a safety perspective; and*
- *Changing from a restrictive movement to a more permissive movement is associated with degradation in safety.*

Shebeeb (1995)

Shebeeb (1995) studied the safety of various left-turn phasing using crash rate as the primary variable. The study included 54 intersections from Texas and Louisiana, and was conducted on approaches, as phasing varied among approaches to the same intersection. All of the subject intersections had exclusive left-turn lanes. Three consecutive years of crash data was used in the analysis. The left-turn phasing studied included[3]:

- Permissive Only
- Lead Protected – Permissive
- Lag Protected – Permissive
- Lead Protected Only
- Lag Protected Only

Crash rate was determined through Equation 3.17.

$$\text{Crash rate} = \frac{A * 10^6}{V_{OP}\ V_{LT}} \qquad [3.17]$$

where: A = total number of left-turn crashes
V_{OP} = Volume of opposing vehicles (straight and right turn) (vph)
V_{LT} = Volume of left-turning vehicles (vph)

The results of the analysis are shown in Table 3.49.

TABEL 3.49: Safety Records of Left-turn Phases

Phasing	Number of Approaches	Mean Crash Rate	Standard Deviation
Permissive Only	38	49.2	94.3
Lead Protected – Permissive	40	35.6	70.9
Lag Protected – Permissive	23	61.2	85.9
Lead Protected Only	45	16.7	26.8
Lag Protected Only	13	21.7	30.6

Statistical tests of significance were applied to the crash rates, assuming normal distribution. The following conclusions were reached:

[3] Two additional left-turn phases referred to as *Lead Dallas* and *Lag Dallas* phases were also studied. They are not presented herein.

- There is no significant difference between protected-permissive and permissive only phasing;
- Protected only phasing is safer than protected-permissive phasing;
- There is no significant difference between the crash rates of lead and lag protected only phasing; and
- There is no significant difference between the crash rates of lead and lag protected-permissive phasing.

Despite the erroneous assumption of normal distribution the results of this study clearly support intuition respecting left-turn phasing – protected only is safer than protected-permissive which is safer than permissive only. The results also indicate that leading left-turn phases are safer than lagging left-turn phases.

Using permissive only phasing as the baseline condition, the data from the Shebeeb study produces the CMFs shown in Table 3.50.

TABLE 3.50: CMFs for Left-turn Phasing

Phasing	**CMF**
Lead Protected – Permissive	0.72
Lag Protected – Permissive	1.24
Lead Protected Only	0.34
Lag Protected Only	0.44

Bamfo and Hauer (1997)

Bamfo and Hauer (1997) used a multivariate regression model to investigate the impacts of actuated signal timing on vehicle-vehicle right-angle crashes in Toronto and Hamilton-Wentworth, Ontario. Four years of data from 278 fixed-time and 28 vehicle-actuated traffic signals were analysed. The target crashes represented 28% of the total vehicle crashes at these intersections.

The general conclusion is that 15% more right-angle crashes are expected at intersection approaches with fixed-time control, than those with that are vehicle-actuated The authors note that the difference in crashes is not likely solely due to the different mode of signal operation. Other intersection characteristics such as approach speeds, and the distance to proximate intersections may be contributing factors.

Stamatiadis et al (1997)

Stamatiadis et al (1997) in developing guidelines for left-turn phasing in Kentucky, included an analysis of the safety impacts of left-turn phasing on crashes. A total of 408 approaches at 217 intersections were selected for study. The results are shown in Table 3.51.

TABLE 3.51: Crash Rates for Different Left-turn Phasing in Kentucky

No. Opposing Lanes	Phase Type	No. Approaches	Avg. Peak LT Volume	Avg. Peak Opposing volume	Crash rate*	CMF
1	Protected	23	92	217	0.55	0.25
	Permitted/ Protected	52	144	463	0.82	0.37
	Permitted	77	72	248	2.22	---
2	Protected	102	117	706	0.28	0.15
	Permitted/ Protected	88	153	1077	1.11	0.61
	Permitted	62	64	642	1.81	---
All	Protected	127	119	630	0.23	0.11
	Permitted/ Protected	142	148	850	0.87	0.43
	Permitted	139	69	424	2.02	---

* = crashes per 100,000 cross volume (avg. peak LT volume x Avg. peak opposing volume)

The CMFs from the Kentucky study are applicable to left-turn crashes only.

Tarall and Dixon (1998)

Tarrall and Dixon (1998) in a study designed to measure traffic conflicts created by the use of protected-permitted signal phasing for double-left turn lanes, also performed a before-after analysis of changing to protected-only phasing at an intersection in Atlanta. The results are shown in Table 3.52.

TABLE 3.52: Safety Effects of Protected-only Left-turn Phasing in Atlanta

Signal Phasing	Average Volume (vehicles/hour)		Conflicts	Conflict Rate (/100 vehicles)
	Opposing	Double Left		
Protected-permissive	1068	673	32	1.84
Protected-only	1305	609	8	0.42

The CMF for protected-only phasing is 0.23 (assuming conflicts and crashes are linearly related).

Vogt (1999)

Vogt (1999) in developing crash models for rural intersections examined the safety impacts of protected left-turn phasing for the major road of signalized intersections with four approaches. Forty-nine signalized intersections were included in the analysis. Negative binomial regression analysis was used to model all crashes within 250 feet of the intersection on the main road, and with 100 and 250 feet of the intersection on the side road, in California and Michigan, respectively. Three years of crash data were used in the analysis.

Vogt found that protected left-turn phasing for the major road at a rural, 4-legged, signalized intersection yields a CMF of 0.51.

Bauer and Harwood (2000)

Bauer and Harwood (2000) using three years of crash data from California developed crash prediction models for several types of rural and urban intersections. Model development used statistical sound procedures. With respect to signalized intersections, only urban intersections with four approaches were modelled. It was found that signal timing produced the CMFs shown in Table 3.53.

TABLE 3.53: CMFs for Signal Timing Changes

Treatment	CMF
Pretimed to Semiactuated	0.94
Pretimed to Fully actuated	1.75
Two-phase to multi-phase	0.82

Thomas and Smith (2001)

Thomas and Smith (2001) undertook an examination of the safety impacts of left turn phasing at four intersections in Iowa. The site selection process is not described; the study methodology is a naïve before-after analysis using crash frequency and severity. The crash frequency is comprised of three years of before and three years of after data categorized by severity, and several impact types. The exact change in signal phasing is not described. The results are shown in Table 3.54.

The results indicate that under a 90% degree of confidence, safety benefits can be expected for total crashes (CMF = 0.64). The aetiology suggests that the majority of the safety gains would be a reduction in left-turn crashes. Exposure was not accounted for in the analysis, as this data was not readily available.

TABLE 3.54: CMFs for Adding Left-Turn Phasing (Thomas and Smith)

Crash Type		Mean	# Sites	90% Confidence Intervals	
				Lower	Upper
Severity	Fatal	N/A	0	N/A	N/A
	Major	0.78	3	2.58	
	Minor	0.50	4	1.01	0.00
	Possible	0.63	4	1.35	
	PDO	0.71	4	1.19	0.24
Impact Type	Right-angle	0.70	3	1.24	0.15
	Rear-end	1.00	4	1.80	0.20
	Left-turn	0.49	4	1.05	
	Other	1.60	4	2.75	0.45
Total		0.64	4	0.77	0.52

In the same study, Thomas and Smith investigated the effects of adding turn phasing in conjunction with adding an exclusive left-turn lane(s). The study methodology was similar, a total of seven sites were investigated. The results are as shown in Table 3.55.

TABLE 3.55: CMFs for Adding Left-Turn Phasing and Turn Lanes (Thomas and Smith)

Crash Type		Mean	# Sites	90% Confidence Intervals	
				Lower	Upper
Severity	Fatal	0.00	2	N/A	N/A
	Major	0.15	5	0.34	
	Minor	0.25	6	0.34	0.17
	Possible	0.49	7	0.66	0.32
	PDO	0.43	7	0.58	0.28
Impact Type	Right-angle	0.48	7	0.72	0.23
	Rear-end	0.63	7	0.97	0.28
	Left-turn	0.27	7	0.38	0.16
	Other	0.55	7	0.74	0.37
Total		0.42	7	0.54	0.30

The results indicate that the addition of left-turn phasing in conjunction with exclusive turn lanes yields safety benefits in all categories, except for fatal crashes where no statistically significant findings are available.

Chin and Quddus (2001)

Chin and Quddus (2001) developed crash prediction models for four-legged, signalized intersections using eight years of crash data from 52 intersections in Singapore. The models predict the annual number of crashes on an arterial approach (both directions included). Of the many variables that were found to influence crash occurrence, the type of signal control was included. Adaptive signal control was found to be safer than pretimed signal control, reducing crashes by 13% (CMF of 0.87).

Night-time Flash

Polanis (2002)

Polanis (2002) reported on the removal of red/amber night-time flashing operation from 19 intersections in Winston-Salem, North Carolina. Site selection information is sketchy; the intersections are "not necessarily… high-crash locations", but are described as locations where the crash pattern indicated a safety benefit from night-time flash removal. Target crashes are right-angle crashes that occur during the night-time flashing operation. The results of the study are shown in Table 3.56.

Sixteen of the 19 intersections exhibited a statistically significant reduction in target crashes at the 95% level of confidence. Polanis aggregates the results for all intersections, which yields CMFs of 0.22 for target crashes, and 0.67 for all right-angle crashes.

There is no reason to believe that removal of night-time flashing operation would have any measurable effect on right-angle crashes at other times of the day. Hence, rather than measuring the safety impacts on total right-angle crashes, it is more informative to use the non-target crashes (total right-angle crashes minus target crashes) as a control group. If this is the case the aggregated data produces the statistics shown in Table 3.57.

These data indicate that the target crashes were reduced by 78%. However, the non-target crashes were also reduced, by 19%. Since the night-time flashing operation should have had no effect on the non-target crashes, a 19% reduction in target crashes could reasonably be expected without removal of night-time flash. Taking into account this background crash reduction leads to a CMF for removal of night-time flashing operation of 0.27, or a 73% reduction in right-angle crashes during the flashing operation.

It bears mentioning that the above study did not account for exposure. This is an important consideration because, although traffic volumes are likely increasing and omission from the analysis leads to under estimation of the safety benefits, the growth of traffic during night-time flashing operation is likely significantly less than during the remainder of the day. If this is indeed the case, then not accounting for exposure leads to an over estimation of the safety benefits of night-time flash removal.

TABLE 3.56: Safety Impacts of Night-time Flashing Operation

Site	Before			After			ΔTarget (crash/month)	ΔTotal (crash/month)	M:S	LU	ST
	Months	Target	Total	Months	Target	Total					
1	50	15	79	45	8	29	-41	-59	2:1	D	F
2	33	5	18	48	0	10	-100	-62	2:1	R	S
3	43	8	39	41	5	21	-34	-44	2:1	R	F
4	48	8	26	48	3	26	-62	0	Na	R	F
5	45	12	23	45	2	25	-83	9	2:1	C	A
6	48	12	23	48	1	8	-92	-65	1:1	D	F
7	58	12	31	80	1	28	-94	-34	5:1	R	S
8	46	6	17	43	4	14	-29	-12	2:1	R	S
9	82	9	80	78	1	49	-88	-36	2:1	C	A
10	22	4	10	22	0	4	-100	-60	4:1	D	F
11	48	8	26	48	1	14	-88	-46	1:1	R	A
12	48	7	32	48	2	17	-71	-47	3:1	C	F
13	49	9	35	47	0	23	-100	-32	3:1	C	A
14	46	4	23	46	2	18	-50	-22	1:1	R	A
15	51	11	44	49	1	26	-90	-38	2:1	C	A
16	46	4	13	45	1	16	-74	26	1:1	R	A
17	45	8	25	45	2	32	-75	28	2:1	C	A
18	44	5	11	44	1	12	-80	9	4:1	D	F
19	36	9	57	36	0	41	-100	-28	3:1	D	F

- Targeted Crashes—Targeted Crashes are those crashes expected to be addressed by a particular intervention. In this instance, right-angle crashes that occurred during the hours the signal was in red/yellow flashing operation.
- % targeted and % total—These refer to the percentage change in targeted and total crashes in the before and after periods (measured in crashes/month).
- M:S ratio—This is the ratio of main-street to side-street traffic volumes at each intersection.
- LU—This is the Land Use around the intersection: D = Downtown, R = Residential and C = Commercial.
- ST—This is Signal Type: F = Fixed time, S = Semi-actuated and A = Actuated.

TABLE 3.57: Safety Effects of Night-time Flashing Operation Using a Control Group

	Months	**Target Crashes**	**Non-target Crashes**
Before	888	156	456
		0.18/month	0.51/month
After	906	35	378
		0.04/month	0.42/month

Chapter 4:

Traffic Signs

CHAPTER 4: TRAFFIC SIGNS

Signing General

Lyles et al (1986)

Lyles et al (1986) examined the safety effects of jurisdiction-wide upgrades to traffic control devices in Michigan. Despite the broader title of "traffic control devices", only upgrades to traffic signs were included in the treatment. Jurisdictions where the sign upgrades took place varied in size. The study methodology was a before-after study with a modified control group. Treatment locations were all local streets; the control group consisted of numbered state routes within the same jurisdiction. It is recognized by the researchers that the state routes differ in many respects from local streets. However, it was thought that using a control group from the same jurisdiction of the treatment locations would better control for confounding by weather, traffic volume changes, and other local factors.

The researchers used crash frequency, severity, and the distribution of crash types as the measures of effectiveness. Three years of before, and three years of after crash data were used. The CMF for sign upgrades on the local street system is 1.04 (see Table 4.1).

TABLE 4.1: Safety Impacts of Traffic Sign Upgrades in Michigan

Site	Three-year Crash Count		CMF
	Before	After	
Treatment	3718	3523	0.95
Control	1753	1593	0.91
Adjusted CMF for treated locations			1.04

Lyles et al conclude that there is no evidence to suggest that jurisdiction-wide sign upgrading on local streets has any effect on safety (both crash occurrence, and severity). The researchers acknowledge that a jurisdiction-wide analysis of sign changes which in most cases were minor (ex., minor adjustments to sign placement) is likely inappropriate. The expected safety benefits of sign upgrades are small, and the number of potential confounding factors present and unaccounted for in a jurisdiction-wide analysis overwhelm the results.

Tople (1998)

Tople (1998) in an evaluation of hazard elimination and safety projects included a review of the safety benefits of traffic signing. The documentation available does not indicate what type of signing was implemented. In any event, the evaluation was a naïve before-after study of crash frequency and crash severity. The impact on crash severity was determined through a comparison of equivalent property damage only crashes, using monetary conversations deemed appropriate by the investigation team. Three years of before and three years of after crash data was used in the analysis.

The results of the analysis are presented in Table 4.2.

TABLE 4.2: Safety Impacts of Traffic Signing in South Dakota

Improvement Type	No. of Sites	Crashes			EPDO Crashes*		
		Before	After	CMF	Before	After	CMF
Traffic Signing	6	1403	1330	0.95	48860	41588	0.85

* EPDO crashes were calculated as (1300*F)+(90*I)+(18*N)+(9.5*P)+PDO
where: F = fatal crash
I = incapacitating injury crash
N = non-incapacitating injury crash
P = possible injury crash
PDO = Property damage only crash

The Tople analysis possesses many potentially serious flaws. Most importantly, the sites were selected for treatment as part of a safety program. This means that the crash record was likely abnormally high, and there is a great potential for regression to the mean artefacts. This shortcoming is likely offset somewhat by a failure to account for changes in exposure. Traffic volumes were not controlled for, but typically volumes tend to increase which would lead to a higher "after" count of crashes. In the end, the South Dakota results are based on a limited number of sites and weak analyses.

Transport Research Laboratory (2000)

The TRL (2000) of the United Kingdom has collected information on the safety impacts of signing through the MOLASSES database (see the section on "Signalization" for more information on MOLASSES). Specific information on whether "signing" means to incorporate new signs, improved signing, or some combination on both is not provided. Therefore, the results are generalized and are only adequate to provide cursory guidance on the magnitude of the potential for safety improvement. The results are shown in Table 4.3.

TABLE 4.3: Safety Effects of Signing Modifications in the United Kingdom

Setting	Number of Locations	Number of Crashes		CMF
		Before	After	
Urban	222	1536	1044	0.68
Rural	136	879	521	0.59

Horizontal Curve Signing and Marking

Arnott (1985)

Arnott (1985) undertook a safety evaluation of traffic activated curve speed warning signs in Ontario that were used at freeway interchanges. A total of five sites were evaluated; three had the signs located on the interchange ramp, two had the sign located on the mainline of the freeway. The treatment was essentially a warning sign that is supplemented by a "TOO FAST" tab sign that was illuminated when upstream detectors identified a vehicle exceeding a preset speed threshold. The actual configuration of the warning sign varied somewhat from location to location; the commonality between sites is the actuated "TOO FAST" warning.

The study methodology was a naïve before-after evaluation of crash frequency and distribution. Nine to eleven years of crash data were available for each site, although the after periods at three of the sites were 13 months or less. Crashes that were coded as "speed too fast" under driver action were the target crashes. The results of the analysis are shown in Table 4.4.

TABLE 4.4: Safety Impacts of Traffic Activated Curve Speed Warning Signs

Site	Period	Total Crashes	Target Crashes	Proportion of crashes that are target crashes (%)	Annual Target Crashes	CMF
1	Before	99	77	78	11.5	0.79
	After	50	39	78	9.1	
2	Before	11	5	45	0.9	---
	After	0	0	--	0.0	
3	Before	118	47	40	6.0	0.60
	After	17	4	24	3.6	
4	Before	89	62	70	7.9	0.46
	After	6	4	67	3.6	
5	Before	430	358	83	39.0	0.06
	After	5	2	40	2.4	
All	Before	747	549	74	65.3	0.29
	After	78	49	63	18.7	

Despite the impressive CMF of 0.29 for all sites combined, Arnott expresses a need for caution in trusting the result. Sites 3, 4 and 5 have after periods of less than 13 months, which Arnott believes does not yield stable long-term results. Site 1, which has been in operation for over four years, exhibits a 21% reduction in target crashes. Arnott states that this is a more reasonable long-term reduction.

Apart from Arnott's expressed concerns with the results of his study, there is also a high potential for regression-to-the-mean effects to be overestimating the results. The sites were likely selected because of an elevated incidence of crashes. Furthermore, time trends and exposure have not been controlled for at any of the sites. For example, at Site 1 the 22 non-target crashes (99 total crashes minus the 77 target crashes), while not supposedly impacted by the treatment, decreased by 22% from the before to after periods. This is almost identical to the 21% decrease observed in the target crashes group. One cannot comfortably conclude that the traffic activated curve speed warning signs had any significant impact on safety.

Zador et al (1987)

Zador et el (1987) completed a study to compare driver behaviour at horizontal curves outfitted with chevrons, post-mounted delineators (PMD), and raised pavement markers (RPM). The devices were independently installed at sites that varied systematically in direction, degree of curvature, and steepness of grade. The study used before and after data on lane placement and speed with a comparison group.

There were 51 sites located on two-lane rural roads. The treatments were installed in accordance with the requirements of the FHWA MUTCD. All sites, including the comparison sites, had edge line markings. The treatments being evaluated are as follows:

- RPMs: 4"x4" amber markers installed on both sides of a double yellow directional dividing line and throughout the curve. The RPMs were visible to both directions of travel and were recessed into the pavement. Spacing was typically 80 feet; along sharper curves where the 80 foot spacing did not result in at least three RPMs being visible at all times, the spacing was decreased to 40 feet.

- PMDs: White, round delineators with a 3 inch diameter on metal posts were installed on the outside of the curve. PMDs were visible to drivers from both directions. Spacing was determined such that a motorist could view at least three delineators simultaneously.

- Chevrons: 18" x 24" chevron signs mounted on the outside of the curve and visible to drivers in both directions. Spacing was determined such that a motorist could view at least three chevrons simultaneously.

Zador et al found that all of the treatments affected driver speeds at night. The differences between the treatment type and the roadway alignment were few, and they were fairly constant over time. The overall conclusion was that the improved delineation increased night-time driving speeds as follows:

- PMDs produced a 2.0 to 2.5 ft/s increase
- RPMs produced a 1.0 ft/s increase
- Chevrons produced a 0.5 ft/s decrease at sites in Georgia, and a 3 ft/s increase at sites in New Mexico

In all cases the night-time speeds were noted as being below the daytime speeds.

Lalani (1991)

The City of San Buenaventura, California as part of a Comprehensive Safety Program installed chevron signs at three locations (Lalani, 1991). Sites were selected because they were considered high crash locations. The analysis was a naïve before-after analysis using crash frequency and one-year before and after periods. Details of the treatment are not reported (i.e., is the curve in an urban or rural location? What are the geometrics of the curves?). The results are shown in Table 4.5.

TABLE 4.5: Safety Impacts of Chevron Signs in San Buenaventura, California

Location	Crashes		CMF
	Before	After	
A	8	4	0.50
B	3	0	0.00
C	3	0	0.00
Totals	14	4	0.29

Lalani does not account for exposure in the safety analysis but reports that traffic volumes in the city increase at an average rate of 6% per annum.

Land Transport Safety Authority of New Zealand (1996)

The safety impact of chevron warning signs at horizontal curves was investigated by the Land Transport Safety Authority of New Zealand (1996). It is not specifically stated, although it is implied that the locations selected for treatment were curves that were experiencing a higher than usual number of collisions. A total of 103 sites were included in the analysis, 83 in rural areas (i.e., a speed limit in excess of 70 km/h), and 20 in urban areas (i.e., a speed limit of 70 km/h or less).

Only nine of the sites were treated with chevrons only, the remaining sites had complementary improvements including raised pavement markings, post mounted delineators, new or relocated traffic signs, pavement markings, etc.

The evaluation methodology was a before-after analysis with some control for overall crash trends. The report indicates that an "expected" number of crashes is calculated by adjusting the "before" data with local crash trends. It is not specifically stated how this was done. The average "before" period for collision data was 5.3 years; the average "after" period was 3.1 years. The analysis yielded the CMFs shown in Table 4.6.

TABLE 4.6: Safety Impacts of Chevron Signs in New Zealand

Crash Type	CMF	
	Open Road	**Urban Area**
All	0.52	0.46
Lost Control	0.57	0.38
Head-on	0.24	0.31
Day	0.63	0.55
Night	0.33	0.33
Twilight	0.79	0.73
Fatal	0.30	0.33
Serious	0.26	0.48
Minor	0.77	0.45

All sites combined yielded a CMF of 0.51. Statistical testing establishes 95% confidence intervals of 0.70 to 0.32.

Tribbett et al (2000)

The California Department of Transportation investigated the safety effectiveness of dynamic curve warning sign systems at five locations. The treatment was the erection of a changeable message sign (CMS) that is connected to a radar speed-measuring device, and detection equipment. The CMSs were 10 feet wide by seven feet high and are full matrix light-emitting diodes. The message to be displays varies but is generally related to either the advisory speed of the downstream curve, or the operating speed of the approaching vehicle.

The study methodology is a naïve before-after analysis of crash frequency. Crashes that occurred from the (proposed) CMS location to one-tenth of a mile downstream of the end of the curve were included in the analysis. The evaluation was undertaken about seven months after sign installation, so the "after" data period is very short. To account for seasonal variations in crash data, the available five years of "before" data was culled to include only the crashes that occurred during the same months as was available in the "after" period.

The results of the Tribbett et al study are shown in Table 4.7.

TABLE 4.7: Safety Effects of Dynamic Curve Warning Sign Systems

Crash Type	Period	Site 1	Site 2	Site 3	Site 4	Site 5	All
Casualty crashes	Before	2.0	.4	0	0.4	0.4	3.2
	After	2	0	2	0	0	4
	CMF	1.00	---	---	---	---	1.25
PDO crashes	Before	1.4	1.2	0.8	1.2	1	5.6
	After	1	2	2	0	0	5
	CMF	0.71	1.67	2.50	---	---	0.89
All crashes	Before	3.4	1.6	0.8	1.6	1.4	8.8
	After	3	2	4	0	0	9
	CMF	0.88	1.25	5.00	---	---	1.02

The "after" period in the Tribbett et al study is too short to be representative of the stable long-term crash record that might be expected at these sites post-sign installation. Furthermore, exposure has not been accounted for in the analysis.

Close-following Warning Signs

Helliar-Symon and Ray (1986)

Helliar-Symon and Ray (1986) in a follow-up to an early study examined the safety impacts of active warning signs for following too close. The treatment was a roadside sign that displayed the message "Following Too Closely" or "Too Close Move Apart" when the gap between vehicles was less than 1.6 seconds. The sign was blanked out when not in use; the active message was supplemented with four flashing amber beacons (one at each corner of the rectangular sign) that flashed in an alternating pattern. The message was visible from the detector that measured the gap between vehicles, was illuminated as soon as an inadequate gap was detected, and remained lit for two seconds.

The study methodology was a before-after analysis using a control group. Treatment and control sites were selected on the basis of a high number of close-following crashes, and for their suitability to accept the device installation. The metric used is injury-crash frequency, using two years of before data and four years of after data at three sites. The crash frequency was measured downstream of the sign installation, where the impact is likely to be realized. Either separate sites, or road sections upstream of the sign were used as control sites. The aggregated crash records from all three sites are shown in Table 4.8.

TABLE 4.8: Safety Impacts of Close-following Warning Signs

Site	Crash Frequency (/year)			
	All crashes		Close-following crashes	
	Before	After	Before	After
Treated	36.0	43.3	3.5	12.8
Control	37.5	34.3	7.5	8.3
CMF		1.32		3.31

The authors present the results disaggregated by site and note that the changes in the crash frequencies are not statistically significant. There does not appear to be an accounting for changes in traffic volume that may greatly influence the outcomes.

Restricted Visibility Signing

Kostyniuk and Cleveland (1986)

Kostyniuk and Cleveland (1986) studied the effectiveness of "Limited Sight Distance" (LSD) signing on crash reduction on paved two-lane roads in Michigan. The LSD sign is the standard diamond-shaped, black on yellow warning sign with the legend "Limited Sight Distance". The dimensions of the signs are not provided.

The study design was a before-after analysis using a control group; the treatment and control groups consisted of pairs of sites that were matched on traffic volume, land use, vegetation, road geometry, lane width, and shoulders. Nine matched pairs of sites were included in the analysis with crash records of 3.6 to 5 years and equal length before and after periods. The results of the analysis are shown in Table 4.9.

TABLE 4.9: Safety Impacts of "Limited Sight Distance" Signing

Site	Length of Before & After Periods (years)	No. of Crashes			
		Treatment		Control	
		Before	After	Before	After
1	2.5	8	0	2	1
2	2.0	1	1	0	2
3	2.0	3	4	1	0
4	2.5	4	4	2	2
5	2.5	10	7	13	11
6	1.8	8	16	6	7
7	2.0	3	8	2	1
8	2.0	0	2	0	3
9	2.0	0	0	0	0
Total		37	42	26	27

Although no details are provided in the article, it is mentioned that the sites are matched on traffic volumes. It is assumed that this means exposure has been accounted for, and a comparison of crash frequency is appropriate.

Both treatment and control sites exhibited an increase in the number of crashes, with the increase in the crashes at the treatment site being greater. This is an indication that the LSD warning sign has provided no safety benefits (a resulting CMF of 1.09). The number of crashes used in both the treatment and control groups are statistical small and do not lend themselves to making statistically significant conclusions respecting the safety impacts of the LSD signs. Therefore, it is not appropriate to conclude that the LSD signs have been detrimental to safety.

It is worthy to note that the term “sight distance” is engineering terminology and it’s meaning is not likely understood by many drivers.

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Chapter 5:

Pavement

Markings

CHAPTER 5: PAVEMENT MARKINGS

General

Transport Research Laboratory (2000)

The TRL (2000) of the United Kingdom has collected information on the safety impacts of improved pavement markings through the MOLASSES database (see the section on "Signalization" for more information on MOLASSES). Whether "markings" is comprised of marking previously unmarked roads, revising existing markings, or some combination of both is unclear from the available material. Therefore, the results are generalized and are only adequate to provide cursory guidance on the magnitude of the potential for safety improvement. The "yellow bar markings" are transverse pavement markings. The results are shown in Table 5.1.

TABLE 5.1: Safety Effects of Markings in the United Kingdom

Treatment	Setting	Number of Locations	Number of Crashes		CMF
			Before	After	
Markings	Urban	196	1721	1208	0.70
	Rural	74	599	370	0.62
Yellow Bar Markings	Urban	2	15	4	0.27
	Rural	2	19	12	0.63

The "yellow bar markings" have a particular small sample size that may lead to misleading results.

Migletz and Graham (2002)

A recent report from Migletz and Graham (2002) included information on an examination of the safety impacts of longer lasting more retroreflective pavement marking materials. The Federal Highway Administration study used a before-after methodology at 55 sites that were located on freeways (65%), non-freeways with speed of 72 km/h or more (15%), and non-freeways with speeds of 64 km/h or less (18%). Several types of durable marking materials replaced 48 sites with conventional solvent paint, and seven sites with epoxy markings.

Multi-vehicle crashes at intersections were excluded from the analysis as these types of crashes were not assumed to be related to longitudinal pavement markings. Furthermore, crashes on ice and snow-covered pavements were also excluded from the analysis. The combined before and after database included a total of 10,312 crashes. The results of the study are shown in Table 5.2.

The study methodology included an adjustment for exposure by including site length, duration of the study period, average daily traffic, the proportion of traffic under day and night conditions, and the proportion of traffic under dry and wet conditions. The estimate of the time that the pavement was wet was determined through computer software.

TABLE 5.2: Safety Effects of Durable Pavement Markings

Pavement Condition	CMF	Statistically significant at 5%
Dry	0.89	Yes
Wet	1.15	No
Dry + Wet	0.94	No

The above data is not sufficient to conclude whether durable markings provide any additional safety benefits over traditional marking materials.

Edgelines

Willis et al (1984)

In a before-after study using a control group, Willis et al (1984) studied the effect of edgelines on injury-crash rate and severity on 600 kms of unlit, rural roads in England. The UK method of marking edgelines permits using solid or broken edgelines (the broken lines indicating that road alignment is good, the solid line indicating that a visibility is cause for caution). The study evaluated edgelines at locations with at least some of the alignment was favourable (mix of solid and broken edgelines), and edgelines on roads where the alignment required the use of solid edgelines. The characteristics of the study sites are shown in Table 5.3.

TABLE 5.3: Study Sites for Edgeline Evaluation in England

Characteristic	Broken+solid	Solid	No edgelines
Total length (km)	206	201	203
No. of sections	26	24	25
ADT (in peak month)	4340	4223	5333
Total injury crashes (in 3 years)	381	353	423
Crash rate (10^8/mvk)	27.18	30.33	25.19
Intersection density (/km)	1.12	1.12	1.16

The study sites were also matched on horizontal and vertical curvature.

The edgelines were either applied as hot-paint, or thermoplastic material; all markings were retro-reflective. Three years of before and two years of after crash data were used in the analysis. The results of the analysis are shown in Tables 5.4 and 5.5.

TABLE 5.4: Effects of Edgelines on Crash Severity in England

Crash Severity	Broken+solid		Solid		Control	
	Before	After	Before	After	Before	After
Fatal	6%	5%	4%	4%	7%	4%
Serious	37%	41%	41%	45%	40%	42%
Slight	57%	53%	56%	51%	54%	53%

TABLE 5.5: Effects of Edgelines on Crash Rates in England

Site	Crash Rate (10^8/mvk)		
	Before	After	Change (%)
Broken+solid	26.1	29.2	+11.9
Solid	26.8	26.6	-0.8
Control	24.1	23.5	-2.7

The changes in the crash rates were not found to be statistically significant. However, a cursory review of these results would indicate that, based on the decrease in the crash rate at the control locations, the edgelining appears to have had a negative impact on safety – increasing the crash rate (or at least slowing the decline) at the treatment sites. CMFs for the broken+solid and solid edgelines would be 1.14 and 1.02, respectively.

Cottrell (1987)

Cottrell (1987) evaluated the effect of 200 mm wide edgelines on run-off-the-road (ROR) crashes on three two-lane rural roads (60.7 miles) in Virginia. The study methodology was a before-after study with a comparison group using three years of before and two years of after crash data. The comparison groups were selected to be consistent with the treatment sections with respect to overall roadway geometry, ADT, and crash frequency.

The ROR crashes did not exhibit a statistically significant difference in crash frequency for individual sites, or combined (95% level of confidence). Cottrell also investigated the incidence of ROR crashes that involved driving under the influence, ROR on horizontal curves, and ROR during darkness, and opposite direction crashes – no apparent effect was evident in any of these cases.

Hall (1987)

Hall (1987) investigated the use of eight-inch wide edgelines on rural, non-Interstate highways in New Mexico as a means of reducing run-off-the-road crashes. Sites were selected for treatment because they had a high rate of ROR crashes, as determined by the rate quality control technique. A comparison group was used in the analysis; this group was also considered to have a high rate of ROR crashes, and was chosen to account for regression-to-the-mean effects. A summary of the sites used in the analysis is found in Table 5.6.

TABLE 5.6: Sites used in the Assessment of Wide Edge Lines

Site Type*	Number of Sites	Length of road (miles)	Before Period (months)	After Period (months)
Treatment 1	19	101	41	17
Treatment 2	12	76	52	5
Control	38	353	41 to 52**	5 to 17

* Treatment 1 and Treatment 2 sites are different groups of sites with the same treatment applied.
** The analysis periods vary because some of the control sites were converted to Treatment 2 sites part way through the evaluation.

The ROR crash rates for Treatment Group 1 experienced a 10% decrease while the control group experienced a 16% decrease. The ROR crash rates for Treatment Group 2 experienced a 17% decrease while the control group experienced a 24% decrease. Hall performed some additional analysis on night-time, curve-related, and opposite-direction crashes to attempt to determine if the wide edge lines were more (or less) effective for certain conditions – there were no significant findings.

Hall recommends against the use of wide edgelines on rural roads in New Mexico based on no evidence suggesting a safety benefit. In fact, the results suggest that the wide edge lines actually have a detrimental impact in safety. If regression-to-the-mean is properly accounted for, and the edge lines truly had no effect on safety performance, then one would expect the reduction in the crash rate for the treatment groups to be the same as that experienced by the control group. Both groups experienced a decline in crash rate, likely due to RTTM effects, however, the treatment groups experienced a smaller decline and this suggests that the RTTM was offset by an increase in collision rate.

Lee et al (1997)

A study to examine the relationship between night-time crashes and the retro-reflectivity of longitudinal pavement markings was conducted by Lee et al (1997). The study was undertaken in Michigan and included 46 test sites that constituted 1875 kms of roadway.

The test sites were of varying classification from freeways to city streets, the marking material included water-based paints, thermoplastic, polyester, and tapes. The tests sites were grouped into four geographic areas with characteristics as shown in Table 5.7.

TABLE 5.7: Test Area Characteristics for Examining the Safety Impacts of Pavement Marking Reflectivity

Characteristic		Test Area			
		A	B	C	D
No. Sites		22	8	10	6
Mileage (miles)		223	286	461	195
ADT	Average	13304	14028	6476	26200
	Minimum	3000	4000	1500	9500
	Maximum	30000	44000	19000	45000
Proportion Commercial (%)	Average	3.65	3.83	3.3	0.98
	Minimum	1	1	1	1
	Maximum	12	15	9	4
Speed (mph)	Average	58	59	52	48
	Minimum	35	45	40	40
	Maximum	70	70	55	55
Street Lighting		Mostly unlit	Mostly unlit	Mostly unlit	Mostly lit
Average Annual Snowfall (inches)		50	70	90	40

Retro-reflectivity measurements were recorded every three months for a three year period (providing 11 measurements for each area). Measurements were taken in accordance with the instructions provided by the manufacturer of the retro-reflectometer. Night-time crashes were selected from the crash database if they were assumed to be associated with the visibility of the markings (i.e., crashes had to occur during dawn, dusk, or dark but could not be intersection crashes, wet road condition, alcohol or fatigue-related crashes).

Volume data was not available, so the researchers examined the ratio of night-time to day-time crashes, which were adjusted for duration (i.e., hours of daylight and darkness). The results are shown in Table 5.8.

In all cases the reflectivity of the lines was considered to be equal to or greater than the minimum required by the standards of the day. A linear regression analysis, however, indicated that there is no substantial correlation between longitudinal marking reflectivity and night-time crashes.

TABLE 5.8: Safety Impacts of Pavement Marking Reflectivity

Time Interval	Measure	Area			
		A	B	C	D
1	RR	178	---	---	---
	N/D	0.56	---	---	---
2	RR	215	167	---	---
	N/D	0.56	0.88	---	---
3	RR	217	211	249	254
	N/D	0.96	0.70	0.34	0.48
4	RR	181	196	157	151
	N/D	0.92	0.41	0.37	0.56
5	RR	196	159	151	205
	N/D	0.19	0.70	0.13	0.33
6	RR	226	195	215	208
	N/D	1.43	0.65	0.37	0.58
7	RR	247	226	281	228
	N/D	1.02	0.81	0.67	0.38
8	RR	146	140	122	170
	N/D	0.33	0.63	0.63	0.82
9	RR	202	162	151	228
	N/D	1.21	0.28	0.67	0.84
10	RR	281	184	181	---
	N/D	0.73	0.70	0.22	---
11	RR	280	166	235	---
	N/D	0.43	0.84	0.69	---

Of interest in the Lee et al paper is that the regression equation for all sites combined, although it is poorly correlated, indicates that as reflectivity increases, so does the proportion of night-time crashes. This is contrary to intuition which suggests that more visible markings should assist motorists in control and guidance of their vehicles, and reduce crashes. However, this finding is consistent with more recent thinking that suggests more reflective markings provide the motorist with a better view of the roadway alignment, leading the motorist into thinking they can see better than they in fact can.

Transverse Markings

Retting et al (1997)

Retting et al (1997) conducted a study to determine the impacts of right-and-thru pavement marking arrows on rear-end crashes at four commercial driveways in Virginia.

The arrows were placed several hundred feet upstream of the driveway. Three of the sites were unsignalized, midblock driveways; one site was a signalized driveway. All of the sites were in an urban or suburban setting and located along multilane arterial roads with posted speed limits of 30 to 45 mph. Information on site selection criteria is not provided. The study methodology is a naïve before-after study of right-turn conflict rates. The results are shown in Table 5.9.

TABLE 5.9: Safety Impacts of Pavement Marking Arrows

Site	Speed Limit (mph)	Distance from Conflict area to Arrow	Traffic Control	Conflicts/ 100 potential conflicts		CMF+
				Before	After	
1	45	250, 475*	Unsignalized	4.7	2.4	0.98
2	30	210	Unsignalized	18.6	9.2	0.91
3	30	145	Unsignalized	7.7	6.3	0.99
4	35	300	Signal	4.8	10.4	1.06

* - two arrows were used because of the higher speed limit
+ - assumes that conflicts and crashes are linearly related

The pavement markings at the unsignalized locations have a combined CMF of 0.96 (assuming conflicts and crashes are linearly related).

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Chapter 6: Pedestrian Safety

CHAPTER 6: PEDESTRIAN SAFETY

Marked Crosswalks

Knoblauch and Raymond (2000)

Knoblauch and Raymond (2000) studied the effect of crosswalk markings on approaching vehicle speeds at six locations in Maryland, Virginia, and Arizona. The study sites were recently resurfaced roads at uncontrolled intersections with stop-control on the minor approach. The speed limit at all sites was 56 km/h (35 mph). The “before” condition was the resurfaced and re-marked roadway without the painted crosswalk. The study methodology is a naïve before-after study using mean speed as the metric. Speed measurements were taken during the same time of day in the “before” and “after” periods.

All sites were observed under three pedestrian conditions: no pedestrian present, pedestrian present and looking in the direction of the approaching vehicle, and pedestrian present but looking ahead (i.e., across the crosswalk). In the conditions where the pedestrian was present, they were stopped at the curb and ready to cross. The pedestrian wore neutral, seasonally appropriate clothes. The same pedestrian was always used for both the before and after measures at a site.

The results of the study are presented in Table 6.1.

These results indicate that the crosswalk markings, without the presence of a pedestrian may have caused the greatest reduction in mean speed. The authors attempted to control for many confounding variables by measuring speeds at the same time of the day, using the same “staged” pedestrian in before and after studies, etc. However, without a properly selected control group, other confounders may have crept into the results. The result seems to be counterintuitive and given the shortcomings of the study methodology, it is not likely valid.

TABLE 6.1: Effect of Uncontrolled, Marked Pedestrian Crossings on Speeds in Three States

Site	Description	Ped Condition	Mean Speed (km/h)		Change (km/h)
			Before	After	
1	Dense suburban residential/ shopping area. Four lane road	No Ped	60.6	61.2	0.6
		Ped looking	57.5	65.3	7.8*
		Ped Not looking	59.6	61.5	1.9
2	Dense semi-urban residential area. Two-lane road with pedestrian refuge island	No Ped	55.8	55.5	-0.3
		Ped looking	58.9	56.8	-2.1
		Ped Not looking	56.5	53.7	-2.8
3	Suburban residential/ shopping area. Four lane road	No Ped	72.1	66.4	-5.7*
		Ped looking	68.5	67.0	-1.5
		Ped Not looking	68.6	66.9	-1.7
4	Suburban residential/ commercial area. Six lane arterial	No Ped	75.7	69.5	-6.2*
		Ped looking	73.3	68.4	-4.9*
		Ped Not looking	70.5	67.7	-2.8*
5	Two lane collector street with no sidewalks	No Ped	63.9	58.9	-5.0*
		Ped looking	59.6	58.9	0.7
		Ped Not looking	62.6	55.3	-7.3*
6	Two lane collector street with no sidewalks	No Ped	79.1	59.3	-19.8*
		Ped looking	61.5	59.4	-2.1
		Ped Not looking	66.5	56.7	-9.8*
All	Not applicable	No Ped	67.9	61.8	-6.1*
		Ped looking	63.2	62.6	-0.6
		Ped Not looking	64.1	60.3	-3.8*

* Denotes a statistically significant change at a 95% level of confidence

Pedestrian Refuge Islands and Split PXOs

iTrans (2002)

The safety of pedestrian refuge islands and split pedestrian crossovers (SPXOs) were investigated by the City of Toronto (iTrans, 2002). A brief description of each device is as follows:

- *Pedestrian refuge island are raised islands approximately 1.8 metres wide and 11 metres long located in the middle of roads that are 16 metres wide. Pedestrian warning signs are located upstream on the vehicular approaches, end island markers and keep right signs are posted on both ends of the island, pedestrian entrances are posted with "Wait for Gap" and "Cross Here" signs. Legislation does not provide the pedestrian with the right-of-way.*

- *Pedestrian crossovers are an Ontario-specific traffic control device that consists of a combination of static traffic signs, an internally illuminated overhead "pedestrian crossing" sign, and pedestrian activated flashing amber beacons. It is fully described in the Ontario Manual of Uniform Traffic Control Devices. Essentially a driver approaching an activated PXO must yield the right-of-way to the pedestrian, and may proceed once the pedestrian has cleared the driver's half of the roadway. The islands present at SPXOs are of the same general design as the pedestrian refuge islands. SPXOs are also outfitted with pedestrian warning signs, keep right signs, and end island markers, however, pedestrian signing is "Caution Push Button to Activate Early Warning System".*

The research included 30 pedestrian refuge islands and 20 SPXOs, in a direct comparison of the safety performance of both devices. In addition, a before-after study to determine the effectiveness of pedestrian refuge islands was undertaken. The crash results from the direct comparison between the two devices are shown in Table 6.2.

TABLE 6.2: Safety Performance of Pedestrian Devices in Toronto

Traffic Control	Total Crashes per location (crashes/year)	Crash Severity (%)		
		Fatal	Injury	PDO
Pedestrian Refuge Island	0.7	3	42	55
SPXO	3.6	1	47	52

The crash frequency at the SPXO locations is 5.5 times higher than the crash frequency at the pedestrian refuge islands. Although exposure is not accounted for in this statistic, elsewhere in the report it is noted that the pedestrian volumes at SPXOs average four times the pedestrian volumes at the refuge islands.

There is no statistical difference in the distribution of crash severity at the 95% level of confidence.

The Toronto study continues to examine the types of crashes occurring at each location. The results are as shown in Table 6.3.

TABLE 6.3: Types of Crashes at Pedestrian Refuge Islands and Split Pedestrian Crossovers

Location	Crash Type			
	Vehicle-Vehicle	Vehicle-Pedestrian	Vehicle-Island	Other
Refuge Island	5 (8%)	6 (10%)	47 (80%)	1 (2%)
SPXO	148 (68%)	35 (16%)	28 (13%)	6 (3%)

In this case there is a statistically significant difference in the relative proportions. Refuge islands seem to be associated with more vehicle-island crashes; SPXOs are associated with more vehicle-vehicle crashes.

The effectiveness evaluation of the refuge islands used a naïve before-after analysis of crash frequency. Before and after time periods were both three years. The analysis found that while 22 pedestrian-involved crashes was reduced to 6 in the after period, vehicle-island crashes, which were not possible in the before period, occurred 43 times in the after period. Again exposure was not accounted for in this analysis.

Therefore, while pedestrian safety appears to have increased, overall safety, as determined by crash frequency, has decreased.

It is noted in the Toronto report, that over 50% of the vehicle-island crashes occurred at four of the 27 refuge island locations, and that three of those four locations had “poor lane alignment”, an indication that the decrease in safety might be ameliorated by better island design.

The Toronto study collected data on pedestrian, and vehicular volumes, and crash frequency but did not attempt to integrate the three pieces of information into SPFs for both types of devices. Linear regression was used to attempt to determine the correlation between crashes and vehicular volume, but no strong correlation was found.

Flashing Beacons with Supplementary Signs

Van Houten (unpublished)

Van Houten (unpublished) examined the effects of pedestrian-activated flashing beacons supplemented with traffic signs on vehicle-pedestrian conflicts at two locations in Dartmouth, Nova Scotia. The activated beacons, which have been in solo use, are amber in colour, and suspended over the crosswalk. A sign placed on the pole displayed the message "PRESS BUTTON TO ALERT MOTORISTS." The beacons continued to flash for 35 seconds once activated.

The beacons were supplemented with an internally illuminated pictogram of a pedestrian (pictogram) that was placed between the two flashing beacons, and advance warning signs (advance sign) displaying a pictogram of a pedestrian, and the legend "YIELD WHEN FLASHING". One site was located at an intersection, and the other was a midblock crosswalk linking a major community recreation facility with a convention centre. Both crosswalks traversed a divided six-lane street with a speed limit of 50 km/h. No indication of site selection procedures is provided.

The safety impacts were measured by proportion of pedestrians who activated the beacons, yielding behaviour of drivers, and vehicle-pedestrian conflicts. As we are unaware of any research that definitively links activations and yielding behaviour to crash occurrence, only the conflict data is reported in this report. A vehicle-pedestrian conflict was scored whenever:

- a motorist had to engage in abrupt audible braking, or change lanes abruptly to avoid striking a pedestrian; or
- a pedestrian had to jump or suddenly step back to avoid being struck by a vehicle.

The researchers controlled for exposure by examining 48 pedestrians per day, who crossed when traffic was present. The results of the study are shown in Table 6.4.

TABLE 6.4: Traffic Conflicts at Different Crosswalk Treatments

Traffic Control	Traffic Conflicts per session	
	Location	
	1	2
Beacon Only	1.0	3.0
Beacon+Pictogram	0.91	N/A*
Beacon+Advance Sign	N/A*	0.37
Beacon+Pictogram+Advance Sign	0.25	0.67

*The revisions to the traffic control were introduced sequentially at each location, with the pictogram being introduced first at Site 1, and the advance sign being introduced first at Site 2.

The report indicates that statistical tests were performed on the collected data. However, the report is silent on whether the reduction in conflicts is statistically significant. Assuming that conflicts and crashes are linearly related, the CMF for the pictogram and advance sign combination is approximately 0.22 to 0.25.

"Turning Traffic Must Yield to Pedestrians" Signs

Abdulsattar et al (1996)

The effect of TTMYP signs on vehicle-pedestrian conflicts was studied by Abdulsattar et al (1996) at 12 marked crosswalks at signalized intersections in two Nebraska cities. The treatment was a 61x76 cm rectangular sign with black lettering on a white background. The sign was posted on the far-left side of the intersection for left-turning vehicles, and on the far-right side of the intersection for right-turning vehicles.

The study employed a naïve before-after analysis with the proportion of pedestrian-vehicle conflicts as the primary end point. A conflict was defined as any evasive action taken by the pedestrian to avoid a collision, or a vehicle occupying the crosswalk within 20 feet of a pedestrian already in the crosswalk. Conflicts were further classified according to the position of the pedestrian in the crosswalk. Type A conflicts are when the pedestrian is clear of the receiving lanes for turning vehicular traffic, type B conflicts are when the pedestrian is within the receiving lanes of the turning vehicular traffic.

Six of the crosswalks were evaluated for left-turning conflicts; six of the intersections were evaluated for right-turning conflicts. Site selection was predicated on high volumes of turning vehicles, high pedestrian volumes, and the existence of pedestrian signal heads. All sites were similar with respect to adjacent land use.

One observer was used to improve inter-rater reliability. Data was collected at the same times during weekdays for all sites.

The results for left-turning and right-turning conflicts are shown in Tables 6.5 and 6.6, respectively.

A Chi-square test for categorical differences was applied, and it was found that the reduction in total conflicts at all sites were significant to a level of 95%. Further analysis indicated that the reduction in left-turn conflicts was significantly greater than the reduction in right-turn conflicts. Left-turn conflicts were reduced by 20 to 65%, right turn conflicts were reduced by 15 to 30%.

TABLE 6.5: Left-turn Vehicle-Pedestrian Conflicts

Site	Study	Number of Observations	Proportion of Observations that resulted in Conflicts (%)		
			Type A	Type B	All
A	Before	213	27	26	53
	After	157	9	9	18
B	Before	313	18	38	56
	After	326	22	23	45
C	Before	118	34	37	71
	After	105	33	19	52
D	Before	240	9	51	60
	After	135	14	13	27
E	Before	180	24	31	55
	After	170	19	18	37
F	Before	209	33	15	48
	After	146	14	9	23
TOTAL	Before	1,273	23	33	56
	After	1,039	19	16	35

TABLE 6.6: Right-turn Vehicle-Pedestrian Conflicts

Site	Study	Number of Observations	Proportion of Observations that resulted in Conflicts (%)		
			Type A	Type B	All
A	Before	277	44	26	70
	After	191	35	17	52
B	Before	306	38	20	58
	After	238	32	11	43
C	Before	468	34	23	57
	After	499	27	15	42
D	Before	432	34	10	44
	After	415	29	5	34
E	Before	718	33	15	48
	After	570	21	12	33
F	Before	704	29	14	43
	After	652	29	7	36
TOTAL	Before	2,905	34	17	51
	After	2,565	28	10	38

The researchers correctly identify the following limitations of the above analysis:

- The after study was completed within four weeks of sign installation; longer-term effects of these signs are not known; and

- The sites were geographically focussed and transferability of these results to other jurisdictions is unknown.

School Zone Traffic Control

Schrader (1999)

The City of Springfield, Illinois studied the effectiveness of five school zone traffic control devices by measuring 85th percentile speeds in a before-after analysis with a comparison group (Schrader, 1999). Five treatment sites and one comparison site shared the following characteristics:

- a 20 mph speed limit "on school days when children are present";
- a collector road designation; and
- a high number of pedestrians.

The treatments are as follows:

- Treatment 1: A post-mounted flashing beacon was erected at the entrances to the school zone, and the "on school days when children are present" text on the school zone speed limit sign was replaced with "when flashing".

- Treatment 2: The posts of the school zone speed limit signs were painted lavender, and lavender transverse stripes were marked on the road. The spacing between successive stripes decreased as one moves downstream.

- Treatment 3: Span wire mounted flashing beacons, and school zone speed limit signs were erected at the entrances to the school zone, and the "on school days when children are present" text was replaced with "when flashing".

- Treatment 4: The school zone speed limit signs were replaced with internally illuminated, fiber optic signs with the legend "School Speed Limit 20". The signs were illuminated during school hours.

- Treatment 5: The entrance to the school zone was supplemented with a pavement marking reading "20" in 2.44 metre high lettering.

Speed studies were conducted one month and six months after installation, and the results are shown in Table 6.7. only Treatment 4, the fibre optic sign resulted in a statistically significant speed reduction.

TABLE 6.7: Impacts of Different School Zone Traffic Control Devices on Speed

Site	85th Percentile Speed (mph)		
	Before	After	
		One month	Six months
Control	28.4	29.7	29.7
Treatment 1	27.3	26.8	26.9
Treatment 2	27.4	26.0	27.4
Treatment 3	25.6	26.7	25.3
Treatment 4	33.1	29.8	30.3
Treatment 5	32.7	31.9	N/A

General

University of Washington (2002)

A synopsis of articles on child pedestrian injury interventions using environmental techniques has been assembled by the University of Washington (2002). The synopsis includes mainly reviews of traffic calming devices that are intended to increase pedestrian safety, but also includes some operational changes such as traffic signing. The articles reviewed used crashes, conflicts, and injuries as the outcomes (measures of effectiveness). However, some surrogates that are not definitively correlated with crashes are also included. The main finding is that area-wide traffic calming appears to reduce injury crashes with CMFs of 0.75, and 0.90 for local and main streets, respectively. The reader is referred to http://depts.washington.edu/hiprc/childinjury/topic/pedestrians/environment.html for further information.

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Chapter 7: Bicycle Safety

Chapter 7: Bicycle Safety

CHAPTER 7: BICYCLE SAFETY

Wheeler (1992)

Wheeler (1992) examined the effectiveness of advanced stop lines for cyclists at three signalized intersections in the United Kingdom. An advance stop line provides cyclists with a four to five metre storage area downstream of the stop line for vehicles, thereby permitting cyclists to advance through the intersection before motor vehicles, reducing the risk of conflict. The layout of a typical advance stop line is shown in Figure 7.1.

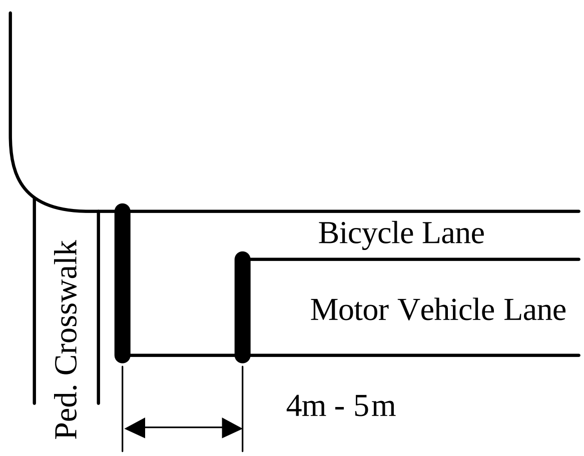

FIGURE 7.1: Typical Layout of an Advance Stop Line for Cyclists

The pavement markings, as shown in Figure A, were supplemented with advance warning signs, and an auxiliary traffic signal head placed at the “setback” stop line for motor vehicle traffic. The auxiliary signal head was comprised of a typical three-section signal head supplemented with a fourth section that displayed a green bicycle symbol while the approach rested in “red”. The idea was to indicate to cyclists that they may proceed to the downstream stop line (where the primary and secondary heads were visible and displaying a red indication). Site selection was not based on an unusually high incidence of cyclist crashes.

A naïve before-after analysis of crash frequency was employed to assess the safety impacts of the advance stop lines. The results are shown in Table 7.1.

TABLE 7.1: Safety Impacts of Advance Stop Lines in the United Kingdom

Measure	Period	Site		
		1	2	3
Duration (years)	Before	5.6	5.6	6.8
	After	6.4	3.4	2.2
Total Crashes	Before	9	16	23
	After	5	1	4
Total Cyclist Crashes	Before	4	4	2
	After	4	0	1
Total Crashes on treated approach(s)	Before	0	12	15
	After	3	0	2
Total Cyclist crashes on treated approach(s)	Before	0	4	0
	After	3	0	1

The overall number of crashes is too low to draw any statistically significant conclusions. Furthermore, any effect that may, or may not, have been caused by the advance stop line has been confounded by additional treatments applied to the subject intersections. It is noted by the author that at one intersection the signal phasing was modified five months prior to the installation of the advance stop line, at another the advance stop lines were installed in conjunction with intersection signalization, and at the final site a turn prohibition was implemented shortly before being treated with the advance stop line.

Chapter 8: Legislation and Enforcement

CHAPTER 8: LEGISLATION AND ENFORCEMENT

SPEED LIMITS

Ullman and Dudek (1987)

Ullman and Dudek (1987) examined the effects of reduced speed limits in rapidly developing urban fringe areas at six locations in Texas. The sites were two and four-lane undivided highways where the 55 mph speed limit was lowered to 45 mph, despite 85th percentile speeds 50 mph and greater. Attempts were made to maintain enforcement levels, and no public advertising was undertaken to minimize confounding by these factors. The study methodology was a naïve before-after study with crash rate as the measure of effectiveness. Crash data used one year before and one year after periods. The results are shown in Table 8.1.

TABLE 8.1: Safety Effects of Speed Limits

Site	Crash rate*					
	All Crashes			Fatal+Injury Crashes		
	Before	After	CMF	Before	After	CMF
1	4.08	2.57	0.63+	1.53	1.47	0.96
2	1.11	1.08	0.97	0.26	0.58	2.23
3	2.02	1.22	0.60+	0.83	0.46	0.55
4	7.32	9.14	1.25	2.98	2.98	1.00
5	7.10	7.03	0.99	3.15	2.79	0.89
6	2.41	3.04	1.26	0.92	1.66	1.80+
Average	4.01	4.01	1.00	1.61	1.66	1.03

*Crash rate = crashes per million-vehicle-miles.
+Significant difference at 95% Confidence level (assuming Poisson distribution)
A lowering of the speed limit has no apparent effect on crash rates in urban fringe areas.

City of Winnipeg (1991)

The City of Winnipeg (1991) studied the effects of speed limit changes on crash rates for two urban streets as follows[4].

- Kenaston Boulevard, a four-lane divided arterial road had the speed limit reduced from 56 km/h to 50 km/h in 1978. Crash rates from 1971 to 1989, inclusive were reported for the subject section of Kenaston Boulevard and for all “regional roads” in Winnipeg. The results are shown in Table 8.2.

[4] A speed limit change on a third urban street was included in the same report, however, there is insufficient “after” data included in the report to substantiate any conclusions on the safety impacts.

- Taylor Avenue between Lindsay and Wilton Streets had the speed limit reduced from 60 km/h to 50 km/h in 1982. Taylor Avenue is a four-lane divided arterial road between Waverly and Wilton Streets, and a two-lane road between Lindsay and Waverly Streets. Crash rates from 1971 to 1989, inclusive were reported for the subject section of Taylor Avenue and for all "regional roads" in Winnipeg. The results are shown in Table 8.2.

TABLE 8.2: Safety Impacts of Speed Limits in Winnipeg, Manitoba

Street	Average Crash Rate		CMF
	Before	After	
Kenaston	6.9	4.6	0.66
All	10.11	7.1	0.70
Taylor	6.7	4.2	0.63
All	9.3	6.8	0.73

Taking into account the general trend of a crash rate reduction on all streets, it would appear that the speed limit reduction is not as pronounced as the results suggest. A CMF of 0.95 to 0.86 is more likely. The before and after periods ranged from seven to twelve years.

Merriam (1993)

Merriam (1993) studied the impacts of speed limit reductions on speeds, speed dispersion, and safety on rural, arterial roads in the Region of Hamilton-Wentworth. Three categories of speed limit reductions were examined. The safety impacts were determined by the change in the collision rate using three years of before and three years of after data in a simple before-after analysis

Sites were selected for treatment for a variety of reasons. None of the "after" speed limits were considered warranted based on the 85th percentile criterion. The results are shown in Table 8.3.

TABLE 8.3: Safety Impacts of Speed Limits in Hamilton, Ontario

Speed Limit Change	Number of Sections	Crash Rate		Average Speed (km/h)		CMF
		Before	After	Before	After	
80 to 60	13	1.47	1.33	76.7	70.9	0.90
80 to 70	12	1.73	1.09	61.2	75.0	0.63
60 to 50	5	2.05	3.07	54.7	64.8	1.50

Crash severity was not investigated.

There is a great potential for the results of this research to mislead the practitioner. First and foremost, the study methodology is a naïve before-after study and therefore does not account for confounders or regression to the mean effects (although the site selection was not undertaken on the basis of a high incidence of collisions, therefore regression to the mean effects are likely not as troublesome).

A speed limit reduction is indicated on the roadway by a change in speed limit signing. While this is certainly a change to the "road" part of the road-driver-vehicle system, it is unlikely that the revised signing by itself has any impact on safety. For the speed limit reduction to have any effect on safety, the signs must evoke a change in driver behaviour. The impetus for the modified behaviour is of no real concern in this discussion, but it suffices to say that the threat of being a larger fine may be the causal chain. At any rate, for the new speed limit to have an impact on safety, the new speed limit must also have an impact on driver behaviour. The conventional wisdom in this respect is that the average speed of the traffic stream is not affected, but the variation in speeds among the travel stream is affected. As the speed limit is moved closer to the 85th percentile speed, the speed variance decreases, and as the speed limit is moved away from the 85th percentile speed, the speed variance increases.

Belanger (1994)

Belanger (1994) studied the safety of unsignalized intersections with four approaches including an examination of the impacts of the main road speed limit on intersection safety. Belanger examined the safety of 149 intersections in eastern Quebec with average annual daily traffic volumes ranging from 388 to 15,942. Crashes that occurred within 30 metres of the intersection, or were recorded as intersection-related were included in the analysis. Regression-to-the-mean effects were accounted for through the application of a multivariate Empirical Bayes technique.

Belanger developed simple SPFs for unsignalized, cross intersections for main road speed limits of 50 km/h and 90 km/h. The SPF is shown in Equation 8.1, and the SPF parameters are as shown in Table 8.4.

$$N = a\ AADT_1^{\,b}\ AADT_2^{\,c} \qquad [8.1]$$

where: N = Crashes/year
AADT = Average annual daily traffic
a, b, c = model parameters (see Table 8.4)

TABLE 8.4: Safety Impacts of Speed Limits at Intersections

Speed Limit (km/h)	a	b	c	k
50	0.003906	0.34	0.49	3.10
90	0.001230	0.41	0.59	5.10

Using the above models it is possible to determine the CMF for intersection crashes that results from a posted speed limit reduction from 90 km/h to 50 km/h on the main road. The results are plotted in Figure 8.1.

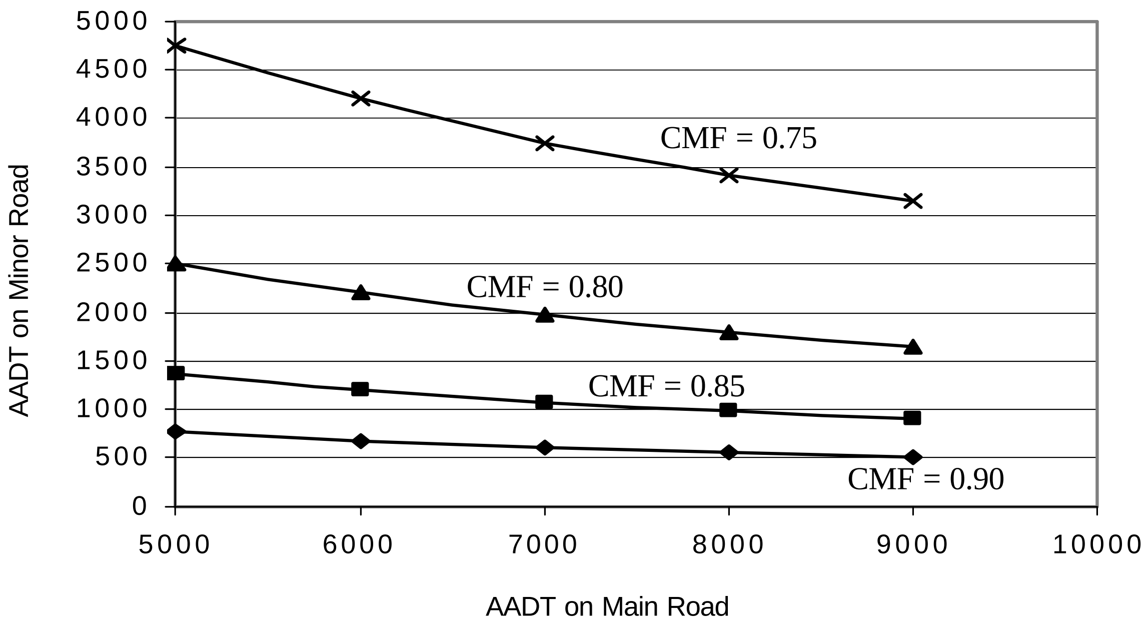

FIGURE 8.1: Crash Modification Factors for Intersection Crashes Resulting from Speed Limit Changes

The plot indicates that the safety benefits of a reduction in the posted speed limit increase with increasing traffic volume on either the main street, or the side minor street. The following caution should be exercised in applying these CMFs:

- As traffic volumes increase determining the need for a change in intersection control should be a primary consideration; and
- The Belanger study does not provide any information on the link between the posted speed limit and the physical conditions of the site. It has long been purported that a change in the posted speed limit has no measurable effect on actual travel speeds. The tenuous link between speed limit and behavioural changes suggests that there may not be a plausible mechanism associated with this assumed cause-effect. Caution should be exercised in applying these CMFs.

Hadi et al (1995)

Hadi et al (1995) in developing crash prediction models for nine types of roadways in Florida, included the speed limit as a potential independent variable. Speed limit was included because of the availability of the data, and the expectation that it would correlate with crash occurrence. Four years of crash data were used in the analysis. Crash prediction models were developed using negative-binomial regression. The equations were disaggregated by crash severity, the resulting CMFs for changes in the speed limit are shown in Table 8.5.

TABLE 8.5: CMFs for Changes in the Speed Limit in Florida

Setting	Class	No. Lanes	Median	CMF for a 1 mph increase in speed		
				Total	Injury	Fatal
Rural	Highway	2	U	0.97	0.98	1.00
		4	D	1.00	0.98	1.00
	Freeway	4 and 6	D	1.00	1.00	1.00
Urban	Highway	2	U	0.97	0.98	1.00
		4	U	0.95	0.96	1.00
		4	D	1.00	1.00	1.00
		6	D	1.00	0.97	1.00
	Freeway	4	D	0.97	0.98	1.00
		6	D	0.97	0.97	1.00

The regression equations that produce these CMFs demonstrate that an increase in the posted speed limit is associated with either no change or a decrease in the number of crashes.

Liu and Popoff (1997)

Liu and Popoff (1997) examined the relationship between travel speed and casualty crash occurrence on Saskatchewan highways. Property damage only collisions were excluded from the analysis, because of changes in the reporting threshold over time. The authors produced the following equations relating casualties and casualty rates to speed measurements.

$$C = -17126.1 + 190.71\, V_{avg} \qquad R^2 = 0.81 \qquad [8.2]$$
$$C = -12473.8 + 133.88\, V_{85} \qquad R^2 = 0.85 \qquad [8.3]$$

where: C = number of casualties
V_{avg} = average travel speed (km/h)
V_{85} = 85th percentile speed (km/h)

$$CR = 0.0298\, V_{avg} + 0.0405\, (V_{85} - V_{15}) - 3.366 \qquad R^2 = 0.94 \qquad [8.4]$$

where: CR = Casualty rate (casualties/million-vehicle-kilometre)
V_{avg} = average travel speed (km/h)
V_{85} = 85th percentile speed (km/h)
V_{15} = 15th percentile speed (km/h)

The results of the study are consistent with the conventional wisdom respecting travel speed and safety. However, by excluding the property damage only crashes, it cannot be determined if a change in speed is correlated with a change in collision occurrence. The finding that an increase in average speed, or 85th percentile speed increases the number of casualties does not necessarily mean that these variables also have a direct relationship with all crashes. It is well known that increased speed increases crash severity. Without information on the PDO crashes, one may hypothesize that the overall crash frequency is unchanged, but the proportion of casualty crashes increases, and the proportion of PDO crashes decreases.

Vogt and Bared (1999)

Vogt and Bared (1999) also during a study of the safety of intersections on two-lane roads examined the influence that the posted speed limit has on crash occurrence. The researchers developed safety performance models assuming a negative-binomial distribution, and using data from 389 three-legged intersections in Minnesota. It was found that the speed limit on the main road influenced crash occurrence, with a corresponding CMF of:

$$CMF = 0.983^{\Delta V} \qquad [8.5]$$

where: ΔV = change in speed (km/h)

In other words a 10 km/h drop in posted speed on the main road yields a CMF of 0.84. The CMF is independent of other variables. Interestingly, posted speed on the main road was not a statistically significant factor in crashes at four-legged intersections.

Brown and Tarko (1999)

In a study on access control and safety in Indiana, Brown and Tarko (1999) included in their analysis the effects of speed limit on crash occurrence. The thrust of the study was to develop crash prediction models for urban, multilane, arterial roads using negative binomial regression models for PDO, injury, and fatal crashes. Five years of crash data from 155 homogeneous road segments were used in model development. The output from the model indicates that the speed limit had no predictive value in determining crash frequency. In other words, it does not appear as if the speed limit had any impact on crash frequency.

Haselton (2001)

In a recent study by Haselton (2001), the impacts of speed limits on highways in California were evaluated. Three groups of roads were studied, as shown in Table 8.6.

TABLE 8.6: Safety Effects of Speed Limits in California

Speed Limit Change	No. of Sites	Years of Crash Data	No. of Crashes	Total Length (miles)	Road types
65 mph to 70 mph	27	10	270	1315	Rural freeways
55 mph to 65 mph	112	10	1120	1674	Urban and rural freeways
Retain 55 mph	19	9	171	100	Urban freeways

The crash data used in the Haselton study included crashes that occurred on interchange ramps, but not the crashes that occurred at ramp-arterial intersections. The metrics included total crash rate, wet crash rate, dark crash rate, and fatal+injury crash rate. Several study methodologies were employed, the most reliable one is the before-after analysis with a comparison group. The generalized results are reported as follows:

- statistically significant increases in all and fatal crashes coincided with the increase in speed limits from 65 to 70 mph, and 55 to 65 mph;
- the increased speed limit had no effect on crashes fatal+injury crashes and crashes during wet road conditions; and
- the speed limit increase from 55 to 65 mph produced a statistically significant increase in dark crashes, while the speed limit from 65 to 70 mph did not.

Wilson et al (2001)

Wilson et al (2001) in a study respecting safety and speed limits on New Brunswick roadways produced the results shown in Table 8.7 and 8.8. The study used a cross-sectional analysis of 4,459 crashes from 1997 to 1999, inclusive. The results indicate that a lowering of the speed limit from 100 km/h to 90 km/h would be associated with CMFs of 0.71, 0.75, and 0.44 for property damage only, injury, and fatal crashes, respectively. The distribution of crash severities has no appreciable change.

TABLE 8.7: Speed Limits and Crash Rates in New Brunswick

Speed Limit (km/h)	Crash Rate (per 100 million vehicle-kilometres		
	PDO	Injury	Fatal
90	18.33	14.67	0.67
100	25.67	19.67	1.50
CMF	0.71	0.75	0.44

TABLE 8.8: Speed Limits and Crash Severity in New Brunswick

Speed limit (km/h)	Crash Severity (%)		
	PDO	Injury	Fatal
90	68%	30%	2%
100	67%	30%	3%

City of Edmonton (2002)

The City of Edmonton (2002) in the late 1980's piloted a test to raise the speed limit from 50 km/h to 60 km/h on two streets. No information is available to determine how these sites were selected for the pilot study. Nonetheless, the effects of the raised speed limit on speed characteristics are as shown in Table 8.9. The results support the general philosophy that speed limits have very little effect on average and 85th percentile speeds.

TABLE 8.9: The Effects of Raising Speed Limits from 50 km/h to 60 km/h in Edmonton

Site	Direction	Average Speed (km/h)	85th Percentile Speed (km/h)	Proportion Exceeding the Speed Limit (%)
1	SBND	-0.9	-3.1	-65.3
	NBND	0.0	0.4	-60.6
2	EBND	1.9	0.7	-46.7
	WBND	1.0	0.3	-63.3

The City of Edmonton also investigated the use of 30 km/h school zone speed limits on two collector streets with daily traffic volumes ranging from 1,300 to 9,000 [Cebryk and Boston, 1996]. Speed limits on both streets prior to implementation of the school zone speed limit was 50 km/h. The treatment consisted of a 30 km/h speed limit that was effective during the following times:

- 08:00 to 09:30 hrs;
- 11:30 to 13:30 hrs; and
- 15:00 to 16:30 hrs.

In addition, signs were posted at the beginning of the zones. The study methodology was a naïve before-after study using average speed. The treatment was evaluated during the presence and absence of police enforcement. The results are shown in Table 8.10.

TABLE 8.10: Effects of 30 km/h School Zone Speed Limit on Average Speed in Edmonton

Site	Average Speed (km/h)		
	Before	After (No police)	After (Police)
A	44.5	42.7	N/A
B	49.7	47.2	44.9

There is a marginal reduction in speed at both locations, the police presence increase the effect. It is not reported whether this change is statistically significant. The researchers also evaluated the effects of the treatment on speed limit compliance. While voluntary compliance is an important policy consideration, there is no credible evidence to suggest that compliance is correlated with crash occurrence or severity. Hence, this information is not repeated here.

Research on the safety impacts of speed limits is fractured. The less than ideal conditions in which speed-safety studies must take place are difficult if not impossible to overcome, and this has lead to conflicting results. Nonetheless, it seems that there are three monographs that present comprehensive reviews and are likely to provide the practitioner with the most complete picture respecting speed limits and safety. They are:

- *IBI Group (1997) "Safety, Speed & Speed Management: A Canadian Review", Final Report, Transport Canada, Ottawa, Ontario.*

- *Parker MR (1997) "Effects of Raising and Lowering Speed Limits on Selected Roadway Sections", FHWA-RD-92-084, United States Department of Transportation, Federal Highway Administration, Washington, DC.*

- *Managing Speed: Review of Current Practice for Setting and Enforcing Speed Limits. Transportation Research Board, Special Report 254. Washington, DC, 1998.*

Speed Display Boards

Bloch (1998)

Bloch (1998) studied the effectiveness of speed display boards with and without enforcement in Riverside, California. Speed display boards are devices that measure the speed of approaching vehicles, and display the speed to the motorist (sometimes) along side the posted speed limit. The study was conducted on 11 to 12 metre wide collector roads with 40 km/h posted speed limits, in residential areas. Before speed data was collected in the two weeks preceding treatment, after speed data was collected in the two weeks post-treatment.

The treatments are as follows:

- Display Board Only: Speed display board operating from 07:00 hrs to 18:00 hrs on two days; and

- Display Board + Enforcement: Speed display board operating from 07:00 hrs to 18:00 hrs on two days, for two hours of each day, while the board is in operation a police officer on a motorcycle was stationed across the street from the board or immediately downstream of the board.

Speeds were measured at the speed display board, and 320 metres downstream. The carryover effects of enforcement were also measured by recording speeds from 45 minutes after the end of enforcement for two hours. The results are shown in Table 8.11.

TABLE 8.11: Mean Speed Effects of Speed Display Boards

Treatment	Reduction in Mean Speed (km/h)	
	Along side	Downstream
Board Only	9.3	4.7
Board + Enforcement	9.8	9.5

Bloch also examined the effects on those traveling 16 km/h or more over the 40 km/h posted speed limit and found the results in Table 8.12.

TABLE 8.12: Effect of Speed Display Boards on Speeders

Treatment	Proportion 16 km/h or more over posted limit (%)			
	Alongside		Downstream	
	Before	After	Before	After
Board Only	52.5	17.6	62.4	34.4
Board+Enforcement	42.9	11.1	52.5	8.4

The residual or carryover effects of these treatments were assessed one week after removal; the results are in Table 8.13.

TABLE 8.13: Mean Speed Effects of Speed Display Boards after Removal

Treatment	Reduction in Mean Speed (km/h)	
	Along side	Downstream
Board Only	2.7	1.0
Board + Enforcement	0.3	1.0

Curbside Parking

Main (1984)

Main (1984) undertook a before-after analysis of collision rates on a sampling of arterial streets in the City of Hamilton, Ontario, to assess the safety impacts of curbside parking prohibitions. The parking prohibitions were full-time and generally accompanied by a stopping prohibition during the weekday morning and afternoon peak travel periods. Site selection was intended to be representative of the entire system, with all sites in areas that were fully developed.

The results of the safety analysis are shown in Table 8.14. The total number of collisions do not include collisions at signalized intersections. All six of the road sections under analysis displayed a reduction in the collision rate, with an average reduction of 37% or a CMF of 0.63.

TABLE 8.14: Safety Effects of Prohibiting Curbside Parking

Section	Length	Year Implemented	Before			After			CMF
			#	AWDT	Rate	#	AWDT	Rate	
1	1.03	1969	19	3660	4.8	9	3860	2.2	0.46
2	0.75	1969	38	4410	11.0	19	4610	5.2	0.47
3	1.71	1968/9	76	7850	5.4	58	9600	3.4	0.63
4	0.43	1975	40	20560	4.3	30	21600	3.1	0.72
5	0.84	1973	76	16180	5.3	62	18910	3.7	0.70
6	1.90	1970	477	20180	11.9	463	24370	9.5	0.80
Average									0.63

McCoy et al (1990)

McCoy et al (1990) using an extensive dataset from the Nebraska highway system investigated the relative safety of different types of curbside parking (i.e., parallel, low-angle and high-angle parking). The study employed a cross-sectional study design with non-intersection and parking crash rates, and the proportion of parking crashes as the metrics. Parking crashes were defined as crashes that involved a parked vehicle or a vehicle that was either parking or unparking. Crashes that were caused by a vehicle slowing or stopped to park, but that did not involve the parking vehicle were not included in parking crashes because of limitations on data.

The crash rates for the different types of parking are shown in Table 8.15.

TABLE 8.15: Crash Rates for Curbside Parking in Nebraska

Type of Parking	Major Street		Two-way, two-lane streets	
	Non-intersection Crash Rate	Parking Crash Rate	Non-intersection Crash Rate	Parking Crash Rate
	Crashes per MVM			
Painted Parking				
Parallel	1.65	0.55	1.83	0.85
Low-angle	---	---	3.38	2.60+
High-angle	1.20	0.53	3.59	2.91+
Unpainted Parking				
Parallel	1.32	0.28	0.67	0.26
Angle	1.57	0.52	1.67	1.11+
	Crashes per BVMHPS*			
Painted Parking				
Parallel	6.50	2.17	6.58	3.05
Low-angle	---	---	9.59	7.38+
High-angle	7.19	3.19	12.90+	10.50+
Unpainted Parking				
Parallel	7.67	1.65	5.44	2.13
Angle	13.19+	4.40+	12.10+	8.04+

*BVMHPS = Billion vehicle-miles-hours per stall
+ Significantly different from equivalent parallel parking at 5% level of significance

The crash rates calculated using exposure that is based on traffic volume and parking activity (i.e., crashes per BVMHPS) is likely to provide the most valid measure of the relative safety of the different types of curbside parking. In all cases, the streets that permit parallel parking have lower crash rates than the streets with angle parking. Moreover, the low-angle parking yields typically lower crash rates than the high-angle parking streets.

Recognizing the limitations of a cross-sectional study in determining the relative safety of a particular treatment, McCoy et al attempted to control for several confounders by re-examining a subset of the data using "similar block faces". Similar block faces were a subset of two-way, two-lane streets that were matched on traffic, roadway, and land use characteristics. The results of this analysis are shown in Table 8.16, and they support the early findings.

TABLE 8.16: Crash Rates for Curbside Parking in Nebraska

Type of Parking	Two-way, two-lane streets	
	Non-intersection Crash Rate	**Parking Crash Rate**
	Crashes per MVM	
Painted Parking		
Parallel	1.41	1.41
Low-angle	3.88+	2.91
High-angle	4.48+	3.77+
Unpainted Parking		
Parallel	0.91	0.25
Angle	1.74	1.45+
	Crashes per BVMHPS	
Painted Parking		
Parallel	5.00	5.00
Low-angle	8.96+	8.44
High-angle	14.40+	12.10+
Unpainted Parking		
Parallel	2.81	0.77
Angle	5.39	4.49+

*BVMHPS = Billion vehicle-miles-hours per stall

+ Significantly different from equivalent parallel parking at 5% level of significance

Using parallel parking as the base condition, and crash rates calculated using exposure that is based on traffic volume and parking activity (i.e., crashes per BVMHPS), the CMFs for conversion to low-angle, and high-angle parking with painted stalls are 1.69 and 2.42, respectively.

One-way Streets

Hocherman et al (1990)

Hocherman et al (1990) undertook a cross-sectional study of streets in Jerusalem, Israel using three years of injury crash data. Streets were classified according to function (arterial, collector, local) and setting (central business district, and other). One-way arterial streets in the CBD were excluded from the analysis because of insufficient numbers. Collector and local streets in the CBD were grouped for analysis, for similar reasons. Intersections crashes were examined separately from midblock crashes. Intersections were defined as one-way, if at least one of the approaches was one-way; the class of the intersection was determined by the highest class/function of the approaches. Volume data were not available for intersections.

The results of the analysis are shown in Tables 8.17 to 8.19. These results indicate the one-way streets are likely providing a safety benefit in the CBD but are actually detrimental to safety in non-CBD areas.

TABLE 8.17: Midblock Crashes for Non-arterial One-way and Two-way Streets in the CBD

Treatment	Crashes per MVK		
	Pedestrian	**Vehicle**	**All**
One-way	0.49	0.18	0.68
Two-way	0.62	0.15	0.77
CMF	0.79	1.20	0.88

TABLE 8.18: Midblock Crashes for Non-arterial One-way and Two-way Streets Outside the CBD

Treatment	Crashes per MVK			
	Arterial	**Collector**	**Local**	**All**
Pedestrian				
One-way	---	0.61	0.73	0.73
Two-way	0.14	0.39	0.49	0.37
CMF	---	1.57	1.49	1.90
Vehicle				
One-way	---	0.38	0.47	0.45
Two-way	0.12	0.23	0.41	0.28
CMF	---	1.68	1.14	1.63

TABLE 8.19: Intersection Crashes for Intersections Outside the CBD

Treatment	Crashes per Intersection			
	Arterial	Collector	Local	All
Pedestrian				
One-way	0.53	0.35	0.06	0.20
Two-way	0.35	0.17	0.02	0.04
CMF	1.53	2.04	3.71	4.65
Vehicle				
One-way	1.20	0.50	0.14	0.36
Two-way	0.58	0.38	0.04	0.09
CMF	2.06	1.32	3.53	3.96

ENFORCEMENT

Ontario Provincial Police (1998)

The Ontario Provincial Police (1998) from the Kawartha Detachment developed a programme intended to address aggressive driving issues on Highway 7, titled “Safe on Seven”. The OPP chaired a committee (it is unclear who was represented on the committee) that formulated and implemented a plan to educate the public on the principles of safe driving. Although not specifically identified as a high crash location, the 26 kilometre section of Highway 7 that was targeted, experienced more than 700 collisions in five years.

The OPP analysed the crash data to determine the primary causes of crashes, the locations, and other conditions that seem to correlate with crash occurrence. The treatment was as follows:

- *educate the public on the principles of safe driving;*
- *use a system of public complaints respecting aggressive driving, and*
- *send a formal warning to the registered owner of the vehicle that was being aggressively driven.*

An eight-month naïve before-after study indicated that the number of crashes had been reduced to a five year low, the number of injured motor vehicle occupants was reduced by one third, and the number of “local” drivers that were at-fault in crashes was reduced by 10%.

The short “after” period, the significant potential for regression-to-the-mean effects, and the naïve before-after study methodology all draw into question to validity of the results.

Royal Canadian Mounted Police (1999)

Five hundred kilometres of Highway 43 in British Columbia is a two-lane highway that was subject to an enforcement, education, and awareness campaign led by the Royal Canadian Mounted Police (1999). Through a series of consultations and committee meetings the RCMP developed a long-term plan, the first year to focus on enforcement of traffic laws in the corridor. In the first year after implementation of the plan, crash modification factors for fatal, injury, and property damage only crashes were 0.10, 0.87, and 0.82, respectively. The study methodology is not specifically stated, but is assumed that a naïve before-after study was employed. This RCMP study suffers from the same serious shortcomings as the 1998 OPP study previously mentioned.

Beenstock et al (1999)

Beenstock et al (1999) investigated the impacts of police enforcement on crash frequency on non-urban roads in Israel. The study uses multivariate regression analysis, and accounts for seasonal variations, time effects, and the characteristics of different road sections. The researchers examined both the "halo" (spatial) effects of policing, and the time effects. The hypothesis is that crash frequency and the level of police enforcement are inversely related.

The level of police enforcement was measured indirectly through the number of offence notices issued. Three years of enforcement data was obtained for 135 road sections. Over 470,000 offence notices were issued during this time, a third of which were for exceeding the speed limit. During this same time 10,500 crashes occurred on the study sections; 6.2% of the crashes were fatal. The enforcement and crash data were aggregated as monthly totals and resulted in 4,185 observations.

It is recognized that society will likely always require police to be deployed for traffic law enforcement. Therefore, the question of enforcement versus "no enforcement" is not a central issue. Rather the focus of the efforts was on establishing the dose-response relationship between police enforcement and crash occurrence.

This study finds that the effect of policing on crash frequency is non-linear. On *average* a one percent increase in policing (offence notices issued) results in a 0.00358% decrease in crash occurrence. However, on road sections with higher concentrations of police presence there is a 0.51% decrease in crash occurrence. The general findings of the Beenstock et al research are:

- *Small-scale enforcement has no apparent effect on crashes;*
- *Large-scale enforcement has a measurable effect on crashes;*
- *Enforcement has no effect on fatal crashes;*
- *The effects of enforcement dissipate rapidly after cessation; and*
- *The "halo" effect is weak.*

Eger (2002)

Eger (2002) studied the effects of enforcement on injury crashes in Kentucky. Using a negative binomial regression model, and county level crashes from across Kentucky, Eger estimates the impacts of the number of police officers, and the number of sheriff law enforcement officers on injury crashes. The models accounted for many confounding variables such as the mileage of two-lane roads, alcohol availability, young male population, etc. The results indicate that crash modification factors for the number of police officers, and the number of sheriff law enforcement officers, are as follows:

- $CMF = 0.98^N$ where N= Number of police officers available [8.6]
- $CMF = 0.9973^N$ where N=Number of county sheriff officers available [8.7]

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Chapter 9:

Turn Lanes

CHAPTER 9: TURN LANES

Left-turn Lanes

Main (1984)

Main (1984) examined the impacts of adding exclusive left-turn lanes and raised medians at signalized intersections on arterial streets in the City of Hamilton, Ontario. The typical installation consists of a 3.5 metre wide left-turn lane, and 1.5 metre wide raised median that is 30 metres long. The approach taper is 20:1 and pavement markings consist of three left-turn arrows.

Eight locations were selected for analysis. These sites are considered by the author to be representative of the population. Three years of before, and three years of after crash data are used in the analysis. The study is a naïve before-after analysis using crash rates. The results are shown in Table 9.1.

TABLE 9.1: Safety Impacts of Left-turn lanes at Signalized Intersections

Section	Before			After			CMF
	#	AWDT	Rate	#	AWDT	Rate	
1	80	29,870	2.55	47	35,130	1.28	0.50
2	71	28,960	2.34	12*	28,960	0.59	0.25
3	66	25,820	2.44	19	29,020	0.62	0.25
4	42	23,160	1.73	31	27,960	1.06	0.61
5	54	25,160	2.05	42	28,880	1.39	0.68
6	56	26,440	2.02	20	28,500	0.67	0.33
7	51	30,670	1.59	45	40,000	1.07	0.67
8	47	23,270	1.93	16*	25,840	0.59	0.31
Average							0.45

* - Only two years of data are used.

Greiwe (1986)

The City of Indianapolis, Indiana evaluated the safety impacts of re-striping intersection approaches with four lanes (two in each direction), to include opposing left-turn lanes (Greiwe, 1986). No indication is given as to the site selection process. The study methodology included a naïve before-after study of crash frequency with two-years of before, and a minimum of one-year of after data. Target crashes were left-turn, right-angle, and rear-end crashes. It is noted by the author that traffic volumes remained nearly the same throughout the study period, so adjustments for exposure were not required.

The results of the analysis are shown in Table 9.2.

TABLE 9.2: Safety Impacts of Re-striping to Provide Left-turn Lanes

Site	Before					After				
	LT	RA	RE	Other	All	LT	RA	RE	Other	All
1	9	3	6	6	24	5	1	3	3	12
2	2	3	0	1	6	0	0	0	0	0
3	1	12	4	0	17	0	4	0	0	4
4	4	0	3	5	12	0	1	3	2	6
5	0	1	5	3	9	0	3	2	2	7
6	3	2	4	3	12	0	0	4	1	5
7	4	3	1	4	12	3	1	0	0	4
8	4	0	3	3	10	3	1	2	0	6
Total	27	24	25	25	102	11	11	14	8	44

The CMFs are 0.41, 0.46, 0.56, and 0.43 for left-turn, right-angle, rear-end, and all crashes, respectively.

In this same study Greiwe examined the safety impacts of modernizing existing traffic signals by making improvements to signal head visibility and adding opposing left-turn lanes. Five locations were studied, the site selection process is not provided, and the details concerning the exact treatment are unknown. Nonetheless, the results of the analysis are shown in Table 9.3.

TABLE 9.3: Safety Impacts of Providing Left-turn Lanes and Signal Modernization

Site	Before					After				
	LT	RA	RE	Other	All	LT	RA	RE	Other	All
1	7	6	3	2	18	1	0	3	2	6
2	1	9	1	1	12	0	1	0	1	2
3	10	10	2	2	24	2	1	1	1	5
4	12	7	3	5	27	1	2	3	2	8
5	2	2	4	2	10	3	1	3	2	9
Total	32	34	13	12	91	7	5	10	8	30

The CMFs are 0.22, 0.15, 0.77, and 0.33 for left-turn, right-angle, rear-end, and all crashes, respectively.

Tople (1998)

Tople (1998) in an evaluation of hazard elimination and safety projects included a review of the safety benefits of left turn lanes that were introduced through pavement markings, and through reconstruction. The evaluation was a naïve before-after study of crash frequency and crash severity. The impact on crash severity was determined through a comparison of equivalent property damage only crashes, using monetary conversations deemed appropriate by the investigation team. Three years of before and three years of after crash data was used in the analysis.

The results of the analysis are presented in Table 9.4.

TABLE 9.4: Safety Impacts of Left Turn Lanes in South Dakota

Improvement Type	No. of Sites	AADT Range	Crashes			EPDO Crashes*		
			Before	After	CMF	Before	After	CMF
LTL through re-painted	2	17807 – 36545	26	17	0.65	873.5	236.5	0.27
LTL through reconstruction	3	4115 - 10614	13	9	0.69	423	356.5	0.84
Combined	5		39	26	0.67	1296.5	593	0.46

* EPDO crashes were calculated as (1300*F)+(90*I)+(18*N)+(9.5*P)+PDO
where: F = fatal crash
I = incapacitating injury crash
N = non-incapacitating injury crash
P = possible injury crash
PDO = Property damage only crash

The Tople analysis possesses many potentially serious flaws. Most importantly, the sites were selected for treatment as part of a safety program. This means that the crash record was likely abnormally high, and there is a great potential for regression to the mean artefacts. This shortcoming is likely offset somewhat by a failure to account for changes in exposure. Traffic volumes were not controlled for, but typically volumes tend to increase which would lead to a higher "after" count of crashes. In the end, the South Dakota results are based on a limited number of sites and weak analyses.

Vogt (1999)

Vogt (1999) in developing crash models for rural intersections examined the safety impacts of left-turn lanes for the major and minor roads at stop-controlled and signalized intersections with three and four approaches. Eighty-four stop-controlled intersections with three approaches, 72 stop-controlled intersections with four approaches, and 49 signalized intersections with four approaches were included in the analysis. Generalized linear regression using a negative binomial distribution was used to model all crashes within 250 feet of the intersection on the main road, and with 100 and 250 feet of the intersection on the side road, in California and Michigan, respectively.

Vogt determined that a left-turn lane on the major road at a stop-controlled, rural intersection with four approaches is associated with a CMF of 0.62. For the other two types of intersections it was found that left-turn lanes on the major road are either not significantly correlated with crash occurrence, or were positively correlated with some other variable under consideration, and did not exhibit an apparent safety effect. Left-turn lanes on the minor roads at all three types of intersections had no safety impact of were cross-correlated with another variable.

Harwood et al (2000)

Harwood et al (2000) using an expert panel of road safety professionals reviewed the available evidence on the safety impacts of adding turn lanes to the main street approaches at the intersections of rural, two-lane highways. The panel opined that CMFs for left-turn lanes are as shown in Table 9.5.

TABLE 9.5: Crash Modification Factors for Left-turn Lanes

Intersection Type	Traffic Control	Number of main street approaches on which left-turn lanes are added	
		One Approach	Both Approaches
Three-leg	Stop	0.78	---
	Signal	0.85	---
Four-leg	Stop	0.76	0.58
	Signal	0.82	0.67

Rimiller et al (2001)

The safety benefits of left-turn lanes were studied at 13 intersections in Connecticut by Rimiller et al (2001). Using an Empirical Bayes model to account for regression-to-the-mean effects, Rimiller examined the crash data from 1989 to 1998, inclusive at 13 intersections that were categorized by population density, intersection control, number of approaches, and number of lanes. The results are shown in Table 9.6.

TABLE 9.6: Safety Impacts of Left-turn Lanes (Rimiller, 2001)

Control	Legs	Lanes	No. Sites	AADT	CMF
Signal	4	2	1	7,500	0.41
Signal	4	4	6	21-43k	0.65
Signal	3	4	2	24-25k	0.61
Unsignal.	3	2	1	13600	0.51
Signal	3	4 (Div)	1	25000	0.98
Signal	3	2	1	16400	0.47

Intersections that were selected for analysis were intersections that had already been reconstructed with left-turn lanes. No information is provided on the site selection criteria, but it is assumed that these sites "warranted" the turn lanes. Again, this method of site selection limits the applicability of the results to those intersections that warrant left-turn lanes.

Thomas and Smith (2001)

Thomas and Smith (2001) undertook an examination of the safety impacts of providing exclusive turning lanes at 8 intersections in Iowa. The site selection process is not described; the study methodology is a naïve before-after analysis using crash frequency and severity. The crash frequency is comprised of three years of before and three years of after data categorized by severity, and several impact types. The results are shown in Table 9.7, some outliers have been removed from the dataset.

TABLE 9.7: CMFs for Adding Exclusive Turn Lanes (Thomas and Smith)

Crash Type		Mean	# Sites	90% Confidence Intervals	
				Lower	Upper
Severity	Fatal	0.00	1	N/A	N/A
	Major	1.40	5	3.86	
	Minor	1.96	7	3.28	0.65
	Possible	1.05	7	1.28	0.82
	PDO	0.77	6	0.91	0.62
Impact Type	Right-angle	1.40	7	2.19	0.61
	Rear-end	0.78	7	1.06	0.49
	Left-turn	2.27	7	3.61	0.92
	Other	0.69	7	0.87	0.60
Total		0.88	7	1.12	0.64

The results indicate that under a 90% degree of confidence, safety benefits can be expected for total crashes. The aetiology supports the apparent increase in crash severity, as the exclusive turn lanes might be expected to increase the overall speed travelled through the intersection. It is surprising, and perhaps counterintuitive, to see that left-turn crashes actually increase (CMF=2.27) by the addition of left-turn lanes.

Region of Waterloo (2001)

As part of an ongoing program the Region of Waterloo, Ontario routinely assesses the street system for locations with an elevated risk for motor vehicle crashes, and implements appropriate countermeasures. The Region of Waterloo (2001) reports that in

1998 one of their high crash locations was modified to include a left and right turn lane on one of the intersection approaches. Crashes were reduced from nine to four, for a CMF of 0.44.

While impressive, the Waterloo analysis is a naïve before-after study of crash frequency using one-year of before and one year of after data. Furthermore, the countermeasure was implemented, at least in part, because this location had an aberrant crash record. The results are very unreliable due to a failure to account regression-to-the-mean, the limited sample size, and the failure to account for exposure.

Harwood et al (2002)

In a comprehensive analysis of the safety impacts of left- and right-turn lanes, Harwood et al (2002) conducted before-after studies for intersection improvements in eight states. A total of 280 sites were treated and 300 sites were used as comparison or reference sites. The sites were in urban and rural settings and were either two-way stop controlled, or signalized (i.e., all-way stop controlled sites were excluded from the analysis). All sites has either three or four approaches; all approaches were public streets (i.e., no private driveways were included in the sites).

The Harwood et al study employed three different evaluation methods: the matched-pair approach, the before-after evaluation with a comparison group, and the before-after study using Empirical Bayes methods. Crashes that were included in the analysis are those that occurred within 250 feet of the intersection, and were coded as intersection-related. Before and after time periods varied between one and 10 years, with averages of 6.7 years and 3.9 years, for the before and after periods, respectively.

The results of the analysis and the recommendations of the researchers are shown in Table 9.8.

TABLE 9.8: Safety Impacts of Left-turn Lanes on Major Road Approaches

Intersection Type	Setting	Traffic Control	No. of Approaches on which Left-turn Lanes are added	
			One	Two
Three-leg	Rural	STOP control	0.56	--
		Signal	0.85	--
	Urban	STOP control	0.67	--
		Signal	0.93	--
Four-leg	Rural	STOP control	0.72	0.52
		Signal	0.82	0.67
	Urban	STOP control	0.73	0.53
		Signal	0.90	0.81

2WLTL

Hoffman (1974)

Hoffman (1974) examined the safety effects of constructing 2WLTLs on four four-lane arterial streets in Michigan. All of the streets had strip commercial development. The 2WLTL were provided with signs and pavement markings as prescribed by the standards of the day. Traffic volumes on the four streets ranged from 15,000 to 30,000 ADT. A total of 6.58 miles of 2WLTL were evaluated. The study methodology was a naïve before-after study using crash frequency as the measure of effectiveness. The results are shown in Table 9.9. Only one year of before and one year of after crash data were used in the analysis.

TABLE 9.9: The Safety Impacts of 2WLTL in Michigan

Crash Type	No. Crashes		CMF
	Before	After	
Head-on	94	52	0.55
Rear-end	238	90	0.38
Right-angle	92	105	1.14
Sideswipe	42	39	0.93
Other	66	70	1.06
Total	532	356	0.67

The analysis does not account for exposure. Hoffman does mitigate this weakness by stating that the traffic volumes in the after period were about seven percent higher than the before period, suggesting that the safety benefit may be even greater. However, it is not clear why these sections of road were selected for treatment, and there may be a significant regression-to-the-mean bias. Furthermore, without a control group it is difficult to apportion to crash reduction to the 2WLTL installation.

Main (1984)

Main (1984) examined the effects of 2WLTLs on arterial streets in the City of Hamilton, Ontario. The 2WLTLs were five metres wide, marked and signed in accordance with the then current version of the Transportation Association of Canada's Manual of Uniform Traffic Control Devices. All 2WLTLs were constructed on four lane roads, resulting in a five-lane cross-section. At the time only four sections of 2WLTL were available for analysis. The study methodology was a naïve before-after study using crash rate as the metric. Three years of before data were compared to one and two year after periods. The results are shown in Table 9.10.

TABLE 9.10: Safety Impacts of 2WLTL

Section	Length (km)	Before			After			CMF
		#	AWDT	Rate	#	AWDT	Rate	
1	0.5	49	22,390	4.17	12+	23,050	2.98	0.71
2	0.5	63	22,560	5.32	11+	22,030	2.86	0.54
3	0.4	114	24,190	11.23	32*	22,470	5.09	0.45
4	0.6	100	19,900	7.98	23*	19,400	2.83	0.35
Avg.								0.51

*Two years of data
+one year of data

Yagar and Van Aerde (1984)

Yagar and Van Aerde (1984) studied the safety impacts of 2WLTLs on jurisdictions across Canada and the United States. Data was collected from road authorities via a survey, hence the configuration of the 2WLTLs and the settings in which they were implemented are expected to vary greatly within the dataset. Nonetheless, crash data was supplied for 30 road sections that had been provided with a 2WLTL.

The study methodology was a naïve before-after study using crash frequency. The results are shown in Table 9.11.

TABLE 9.11: Safety Impacts of 2WLTL in North America

Crash Type	No. of Sites	No. of Crashes		CMF
		Before	After	
Left-turn from 2WLTL	14	130	83	0.64
Left-turn into 2WLTL	13	59	46	0.78
Left-turn from 2WLTL with left-turn into 2WLTL	18	174	112	0.64
No left-turn involved	21	817	513	0.62
All crashes	30	2479	1788	0.72

The analysis presented in the Yagar and Van Aerde study fails to account for exposure. However, elsewhere in the paper the authors examine the pre and post traffic volumes on the 2WLTL equipped streets, and the general trend appears to be an increase in traffic. This would mean the results of this study are conservative. The researchers did not gather any information on site selection processes, so the potential and magnitude of any regression-to-the-mean effects are unknown.

Lalani (1991)

The City of San Buenaventura, California as part of a Comprehensive Safety Program introduced 2WLTLs to five areas of the city (Lalani, 1991). Sites were selected because they were considered high crash locations. The analysis was a naïve before-after analysis using crash frequency and one-year before and after periods. Details of the treatment are not reported (i.e., what was the original cross-section? And was reconstruction required?). The results of the analysis are shown in Table 9.12.

TABLE 9.12: Safety Impacts of 2WLTLs in San Buenaventura, California

Location	Crashes		CMF
	Before	After	
A	18	9	0.50
B	9	5	0.56
C	8	5	0.63
D	3	1	0.33
E	25	11	0.31
Totals	73	31	0.42

Lalani does not account for exposure in the safety analysis but reports that traffic volumes in the city increase at an average rate of 6% per annum. Furthermore, there is no information provided on access density.

Bonneson and McCoy (1997)

Bonneson and McCoy (1997) developed crash prediction models to assess the safety impacts of median treatment on urban and suburban arterial roads in Omaha, Nebraska and Phoenix, Arizona. Roads included in the dataset shared the following characteristics:

- Annual traffic volume in excess of 7000;
- Speed limit between 30 and 50 mph;
- 350 foot or more signalized intersection spacing;
- Direct access from abutting properties; and
- No more than 3 lanes in each direction

Three years of crash data were used in the analysis. Crashes excluded from the database included signalized intersection-related crashes, and crashes resulting from extraordinary circumstances (i.e., driving under the influence, and crashes on snow covered streets). Overall 126.5 kilometres of street and 6,391 crashes were included in the analysis.

The equations were developed using generalized linear regression assuming a negative binomial distribution. They are as follows:

$$A_T = ADT^{0.91}\ L^{0.852}\ e^{(-14.15 + 0.018Ib - 0.093Ir + 0.0077(DD+SD)Ib + 0.0255PDO)} \quad [9.1]$$
$$A_U = ADT^{(0.91+1.021Ir)}\ L^{0.852}\ e^{(-14.15 - 10.504Ir + 0.57Ip + 0.0077(DD+SD)Ib + 0.0255PDO)} \quad [9.2]$$

where: A_T = Annual number of accidents for streets with 2WLTLs
A_U = Annual number of accidents for undivided streets
ADT = Annual daily traffic
L = Length of street (metres)
DD = driveway density (/km)
SD = unsignalized intersection density (/km)
PDO = proportion of property damage only crashes (%)
Ib = Business land use (=1 if business or office use, =0 otherwise)
Ir = Residential land use (=1 if residential or industrial, =0 otherwise)
Ip = Parking (=1 if parallel, curbside parking permitted, =0 otherwise)

Therefore, to determine the safety impacts of a 2WLTL installation, the analyst should estimate the annual crash frequency produced by each equation, and compare the results.

Tople (1998)

Tople (1998) in an evaluation of hazard elimination and safety projects included a review of the safety benefits of 2WLTLs that were introduced through pavement markings, and through reconstruction. The documentation available does not indicate if the 2WLTLs created through pavement markings were a result of a four-lane cross-section being converted to three lanes, or if a wide two-lane street was restriped as a three lane street. In any event, the evaluation was a naïve before-after study of crash frequency and crash severity. The impact on crash severity was determined through a comparison of equivalent property damage only crashes, using monetary conversations deemed appropriate by the investigation team. Three years of before and three years of after crash data was used in the analysis.

The results of the analysis are presented in Table 9.13.

The Tople analysis possesses many potentially serious flaws. Most importantly, the sites were selected for treatment as part of a safety program. This means that the crash record was likely abnormally high, and there is a great potential for regression to the mean artefacts. This shortcoming is likely offset somewhat by a failure to account for changes in exposure. Traffic volumes were not controlled for, but typically volumes tend to increase which would lead to a higher “after” count of crashes. In the end, the South Dakota results are based on a limited number of sites and weak analyses.

TABLE 9.13: Safety Impacts of 2WLTL in South Dakota

Improvement Type	No. of Sites	AADT Range	Crashes			EPDO Crashes*		
			Before	After	CMF	Before	After	CMF
2WLTL through re-painted	3	1500 – 24300	176	160	0.91	4444.5	3436	0.77
2WLTL through reconstruction	5	15000 – 22775	295	270	0.92	5876.5	8243.5	1.40
Combined	8		471	430	0.91	10321	11679.5	1.13

* EPDO crashes were calculated as (1300*F)+(90*I)+(18*N)+(9.5*P)+PDO
where: F = fatal crash
I = incapacitating injury crash
N = non-incapacitating injury crash
P = possible injury crash
PDO = Property damage only crash

Brown and Tarko (1999)

In a study on access control and safety in Indiana, Brown and Tarko (1999) included an examination of the effects of 2WLTLs on crash occurrence. The thrust of the study was to develop crash prediction models for urban, multilane, arterial roads using negative binomial regression models for PDO, injury, and fatal crashes. Five years of crash data from 155 homogeneous road segments were used in model development. The output from the model indicates that the presence of a 2WLTL reduces crash frequency as shown in Table 9.14.

TABLE 9.14: Safety Benefits of 2WLTLs

Crash Type	CMF
PDO	0.50
Injury + Fatal	0.42
Total	0.47

Harwood et al (2000)

Harwood et al (2000) using an expert panel to synthesize the available research determined that the CMF for 2WLTLs when applied to two-lane rural highways is:

$$CMF = 1 - 0.7\, P_{lt/d}\, \frac{0.0047D + 0.0024D^2}{1.199 + 0.0047D + 0.0024D^2} \qquad [9.3]$$

where: $P_{lt/d}$ = proportion of driveway-related crashes that are left-turn crashes susceptible to relief by a 2WLTL expressed as a decimal
D = Driveway density (driveways/mile)

In the absence of information on the proportion of left-turn crashes that are susceptible to relief by a 2WLTL, it may be assumed that it is 50% of the total driveway-related crashes.

Right-turn lanes

Vogt and Bared (1999)

Vogt and Bared (1999) found that the presence of a right turn lane on the main road of a stop-controlled "T" intersections in rural Minnesota was associated with a CMF of 0.79. This finding was statistically significant, although the same result was not found for rural, stop-controlled cross intersections.

Bauer and Harwood (2000)

In a comprehensive crash modelling exercise conducted by Bauer and Harwood (2000) a right turn lane on the main road at 4-leg, rural, stop-controlled intersections was associated with a CMF of 0.85. CMFs for other intersection and lane configurations are shown in Table 9.15.

TABLE 9.15: CMFs for Turn Lanes at Different Intersection Types

Intersection	Lane type	CMF
Rural, 4-leg, stop controlled	Right turn on main road	0.85
Rural, 3-leg, stop controlled	Painted left turn on main road	0.81
	Curbed left turn on main road	0.91
Urban, 4-leg, signal-controlled	Right-turn Channelization on major road	1.12
Urban, 4-leg, stop-controlled	Right-turn Channelization on cross road	1.47
Urban, 3-leg, stop-controlled	Right-turn Channelization on cross road	1.75
	Painted left turn on main road	0.98
	Curbed left turn on main road	1.21

It is curious to note that the right-turn channelization in urban areas is associated with an increase in crashes.

Harwood et al (2000)

Harwood et al (2000) using an expert panel of road safety professionals reviewed the available evidence on the safety impacts of adding right turn lanes to the main street approaches at the intersections of rural, two-lane highways. The panel opined that CMFs for right-turn lanes are as shown in Table 9.16.

TABLE 9.16: CMFs for Right Turn Lanes

Traffic Control	Number of main street approaches on which right-turn lanes are added	
	One Approach	Both Approaches
Stop	0.95	0.90
Signal	0.975	0.95

Chin and Quddus (2001)

Chin and Quddus (2001) in analysing four-legged signalized intersections in Singapore, Japan found that right turn lanes on both approaches increased crash frequency by 38% (CMF=1.38), and left-turn lanes on both approaches decreases crash frequency by 53% (CMF=0.47)[5].

Harwood et al (2002)

In a comprehensive analysis of the safety impacts of left- and right-turn lanes, Harwood et al (2002) conducted before-after studies for intersection improvements in eight states. A total of 280 sites were treated and 300 sites were used as comparison or reference sites. The sites were in urban and rural settings and were either two-way stop controlled, or signalized (i.e., all-way stop controlled sites were excluded from the analysis). All sites had either three or four approaches; all approaches were public streets (i.e., no private driveways were included in the sites).

The Harwood et al study employed three different evaluation methods: the matched-pair approach, the before-after evaluation with a comparison group, and the before-after study using Empirical Bayes methods. Crashes that were included in the analysis are those that

[5] The rules of the road in Singapore requiring driving to the left of the centreline. Therefore, in the Chin and Quddus article, right-turns are equivalent to left-turns in Canada and have been adjusted accordingly.

occurred within 250 feet of the intersection, and were coded as intersection-related. Before and after time periods varied between one and 10 years, with averages of 6.7 years and 3.9 years, for the before and after periods, respectively.

The results of the analysis and the recommendations of the researchers are shown in Table 9.17.

TABLE 9.17: Safety Impacts of Right-turn Lanes on Major Road Approaches

Traffic Control	No. of Approaches on which Left-turn Lanes are added	
	One	Two
STOP control	0.86	0.74
Signal	0.96	0.92

Chapter 10:

Traffic Claming

CHAPTER 10: TRAFFIC CALMING

Lynam et al (1988) and Mackie et al (1990)

Lynam et al (1988) and Mackie et al (1990) reported on the safety impacts of area-wide traffic calming in five cities in England. The characteristics of the sites and the area-wide traffic calming are provided in Table 10.1. The study methodology was a before-after study with a control group. The control groups were matched to the treatment sites on the basis of land use, road network, and crash record. The treatment sites were noted as being of average crash risk, so regression to the mean artefacts are not expected.

Five years of before and two years of after data were used in the analysis, the results of which are abbreviated in Table 10.2. The analysis identified and controlled for time-series and seasonal variations.

TABLE 10.1: Site Descriptions for Area Wide Traffic Calming Analysis in the UK

<table>
<tr><th colspan="2">SITE</th><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td colspan="2">Hierarchy changes</td><td>Restrict through traffic</td><td>Concentrate N/S traffic</td><td>Restrict local roads used to bypass centre</td><td>Reduce number of distributors</td><td>Close one through route, discourage others</td></tr>
<tr><td colspan="2">Identified safety issues</td><td>Two-wheeler crashes on main roads</td><td>Congestion on main roads</td><td>Child pedestrian crashes</td><td>Crashes on collectors</td><td>Crashes at intersections on arterials</td></tr>
<tr><td colspan="2">Prime safety objectives</td><td>Improve main road intersections; restrict side road access</td><td>Improve intersection control; restrict side road access</td><td>Reduce through traffic on locals; improve intersections</td><td>Improve selected collectors; restrict others</td><td>Modify traffic routing; reduce speeds on collectors</td></tr>
<tr><td rowspan="4">Main measures</td><td>Main Road</td><td colspan="3">Mini roundabouts</td><td>No general treatment</td><td>Intersection redesign</td></tr>
<tr><td>Side road intersections</td><td colspan="2">Turning bans</td><td colspan="2">Footway crossovers</td><td></td></tr>
<tr><td rowspan="2">Collector roads</td><td rowspan="2">Central islands; improve ped. crossings</td><td colspan="3">Sheltered parking/pavement extensions</td><td rowspan="2">Central refuges</td></tr>
<tr><td>Peak bus lane</td><td colspan="2">Bus gate</td></tr>
</table>

TABLE 10.2: Safety Impacts of Area Wide Traffic Calming in the UK

Site	CMF Local Control Site	CMF Large Control Area
1	0.90 to 0.75	0.88
2	0.96 to 0.85	0.91
3	0.93	0.91
4	0.81 to 0.68	0.82
5	0.86	0.86
All	0.87	0.88

The 13% crash reduction is unlikely to be due to chance. Further analysis of the crash data provided the following conclusions:

- Crash saving are experienced by all road users with somewhat greater benefits to cyclists and motorcyclists;
- Crashes were reduced on arterial streets and in residential areas; and
- Slight-injury crashes were reduced proportionally more than fatal and serious-injury crashes.

Engel and Thomsen (1992)

Engel and Thomsen (1992) examined the safety effects of speed reducing measures in residential areas in Denmark. The treatments were an advisory speed limit of 15 km/h or 30 km/h, coupled with a variety of physical speed reducing measures. The study methodology is a before-after analysis with a control group using crash rate and speeds as the metrics. Three years of before, and three years of after data were used. In the analysis of crashes a total of five 15 km/h streets and thirty-nine 30 km/h were calmed, and 52 streets were used for control. In the analysis of speeds a total of 41 calmed streets and 13 control streets were used.

The CMF for traffic calming is an impressive 0.27. The reduction in crashes was significant at the 95% level of confidence.

The mean speeds on the control streets did not change from before to after periods. The changes in speed on the treated streets are shown in Table 10.3. Devices were spaced approximately 100 metres apart, therefore measurements 50 metres from the device would likely capture the maximum speeds if motorists increased speed between devices.

The results indicate that the devices that provide a vertical deflection are more effective at reducing speed than the lateral deflections. The authors developed the following regression model that can be used to predict the speed change:

$$\begin{aligned} SC = {} & 29.058-0.6451V_b+0.00376D_{pc}+0.0005352D_{pc}^{2}-148.32\,ln[1+1/D] \\ & -81.50\,ln[1/D_a]-10.001H-2.017X_1-4.724X_2-4.680X_3 \end{aligned} \qquad [10.1]$$

where: SC= Speed change (km/h)
V_b = mean speed before treatment (km/h)
D_{pc} = distance from vehicle to device (metres)
D = spacing from device to previous device (metres)
D_a = spacing from device to next downstream device (metres)
H = height of device (centimetres)
X_1 = 1 if single lateral dislocation; 0 otherwise
X_2 = 1 if double lateral dislocation; 0 otherwise
X_3 = 1 if road narrowing; 0 otherwise

TABLE 10.3: Speed Changes Resulting from Traffic-calming in Denmark

Device	Change in speed (km/h)		
	50m upstream	**At device**	**50m downstream**
Hump, circle segment	-13.7	-7.9	-3.6
Hump, elevated junction	-13.7	-14.7	-8.3
Hump, plateau and circle segment	-6.5	-21.2	-23.6
Hump, plateau	-26.8	-8.1	-16.8
Lateral dislocation, single	-12.1	4.1	3.2
Lateral dislocation, double	-3.7	0.8	-9.5
Road narrowing	-1.7	-3.4	-14.1

Hamilton Associates (1996)

Hamilton Associates (1996) estimated the safety effects of traffic calming by analysing four traffic calming projects in the Greater Vancouver area. Site selection was based on the availability of data. A description of each traffic calming project is provided in Table 10.4. Crashes on internal neighbourhood streets were included in the analysis.

TABLE 10.4: Traffic Calming Site Descriptions

Project	Setting	Concern	No. of Devices	Types of Devices
1	Downtown with medium to high density residential land use	Non-local traffic, safety, liveability	22	Forced turn islands, street closures, diagonal diverters, traffic circles, bulb-outs, one-way street
2	Low density residential	Non-local traffic, safety	17 (not including stop signs)	Stop signs, forced turn islands, bulb-outs, traffic circles, one-way streets
3	Medium density, single family, residential	Non-local traffic		Stop signs at every second street
4	Single family residential	Non-local traffic	17 speed humps	Speed humps, parking and turn restrictions

Crash data typically was comprised of three years of before, and three years of after data. Some shorter analysis periods were used. The results of the analysis are shown in Table 10.5 and 10.6

TABLE 10.5: The Impacts of Traffic Calming in Greater Vancouver on Crash Frequency

Project	Crashes/year		CMF
	Before	After	
1	75.7	62.0	0.82
2	19.3	5.3	0.54*
3	15.0	6.0	0.40
4	41.5	27.5	0.66
Average	30.3	20.2	0.61

*A change in reporting threshold between before and after periods. The CMF has been adjusted to account for the change.

TABLE 10.6: The Impacts of Traffic Calming in Greater Vancouver on Crash Severity

Project	% Injury+Fatal		Change
	Before	After	
1	15	17	+2
2	21	56	+35
3	37	27	-10
4	54	19	-35
Average			-2

The shift to more severe crashes may be the result of an increase in pedestrian and cyclist volumes. The speed humps that were used on Project 4 would have reduced operating speeds, which would have in turn reduced the proportion of casualty crashes.

Berger and Linauer (1998)

Berger and Linauer (1998) examined the effects of gateway treatments on five two-lane roads at the transition from rural to urban areas in Austria. The treatments were raised islands placed in between the two travel lanes that were supplemented with appropriate signs and markings. Four different island designs were used, each intending to provide some degree of road narrowing (by dividing the two lanes), and deflection. The island shapes are shown in Figure 10.1.

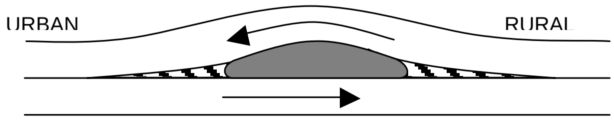
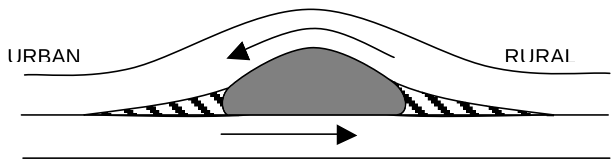
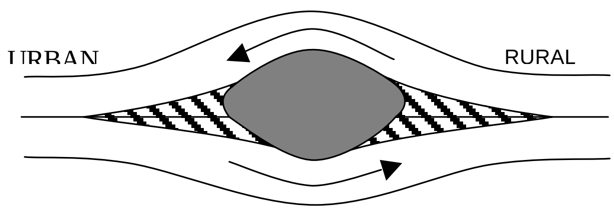
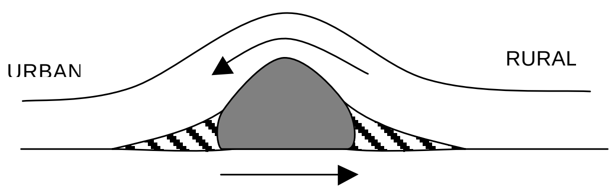
<table>
<tr><th>Sites Where Applied</th><th>Island Geometry</th><th>Description</th></tr>
<tr><td>1</td><td>
</td><td>Lane approaching urban area has minor deflection, straight lane departing urban area</td></tr>
<tr><td>2 and 4</td><td>
</td><td>Lane approaching urban area has a medium deflection, straight lane departing urban area</td></tr>
<tr><td>3</td><td>
</td><td>Approaching and departing lanes have a medium deflection</td></tr>
<tr><td>5</td><td>
</td><td>Lane approaching urban area has a large deflection, straight lane departing urban area</td></tr>
</table>

FIGURE 10.1: Traffic Calming Islands in Austria

The study methodology was a naïve before-after analysis of mean, 85th percentile, and maximum observed speeds recorded near the town sign (which presumably is proximate to the island). The results are shown in Table 10.7.

TABLE 10.7: Speed Effects of Gateway Treatments in Austria

Speed	Period	Site

		1	2	3	4	5
Mean	Before	54.0	58.0	60.0	65.0	65.0
	After	54.1	48.4	44.1	47.2	40.1
	Change	0	-17	-27	-27	-38
85^{th} Percentile	Before	62.0	67.0	70.0	76.0	77.0
	After	61.0	54.5	50.5	55.2	44.6
	Change	-2	-19	-28	-27	-42
Maximum	Before	70.0	88.0	86.0	95.0	97.0
	After	76.2	59.3	56.1	65.8	46.9
	Change	+9	-33	-35	-31	-52

The authors note that Island No. 3 has the additional advantages of:

- requiring motorists to reduce speed just prior to leaving the urban area (which may promote a uniform, lower speed throughout the urban area), and
- preventing motorists entering the urban area from using the opposing lane (anecdotal observations).

Not surprisingly as the deflection increases, so does the speed reduction. Using these results, the following regression models relating the island shape and expected speed were developed:

$$V_{85} = 14.797Ln(L/2d)+19.779 \quad R^2 = 0.9098 \qquad [10.2]$$
$$V_m = 12.907Ln(L/2d)+17.753 \quad R^2 = 0.9693 \qquad [10.3]$$

Where: V_{85} = 85^{th} Percentile speed (km/h)
V_m = Mean speed (km/h)
L = length of island + length of both tapers
d = lateral deflection of lane

Ewing (1999)

Ewing (1999) undertook a comprehensive review of the impacts associated with traffic calming, including the impacts on safety. Using an amalgam of naïve before-after studies that measured crash frequency and included an adjustment for changes in traffic volumes, he identified 55 sites that had been traffic calmed using primarily traffic circles and speed humps. The overall CMF for these calmed sites was 0.96. This report was intended to capture the state of the practice in traffic calming, and not to provide a rigourous overview of the various safety analyses that Ewing collected. Hence, there is no substantial information on site selection procedures, and other information that is required to critically appraise the safety data.

Forbes and Gill (2000)

Forbes and Gill (2000) undertook a review of arterial traffic calming on a road in Ancaster, Ontario. The road was a two-lane arterial with mainly residential development and a speed limit of 50 km/h. The treatment was to construct a series of raised islands between the two lanes of travel, created a narrowing of the pavement, and chicane effect along the road. The islands were supplemented with trees to further enhance the narrowing effect.

The study methodology included an examination of mean speed, and the proportion of traffic exceeding the speed limit using a before-after analysis with a comparison group. The mean speed dropped 9% in the traffic-calmed section (from 54.0 to 49.3 km/h), while the mean speed in the control section dropped only 3%. The effects of the islands on speeding are presented in Table 10.8.

TABLE 10.8: Speed Effects of Traffic Calming in Ancaster, Ontario

Section	Proportion Exceeding the Speed Limit (%)	
	Before	After
Traffic-calmed	67	47
Control	88	85

Statistical testing indicated that both the change in mean speed, and the proportion of vehicles exceeding the speed limit are significant at a 99% level of confidence. It might be questioned, however, if the control section was a suitable comparator since the "before" proportion exceeding the speed limit is considerably different from the test section.

Transport Research Laboratory (2000)

The TRL (2000) of the United Kingdom has collected information on the safety impacts of various traffic calming measures on local streets through the MOLASSES database (see the section on "Signalization" for more information on MOLASSES). The results are generalized and are only adequate to provide cursory guidance on the magnitude of the potential for safety improvement. The results are shown in Table 10.9.

The results from the United Kingdom indicate that traffic-calming measures are effective safety devices. However, caution should be exercised in using the associated CMFs, since:

- *There is no information on why the sites were selected for treatment;*
- *In some instances the sample size is very small;*
- *The study methodology is a naïve before-after study of crash frequency and does not account for regression-to-the-mean; and*
- *The study methodology does not account for exposure or other possible confounding factors.*

TABLE 10.9: Safety Effects of Traffic Calming in the United Kingdom

Treatment	Setting	Number of Sites	Number of Crashes		CMF
			Before	After	
Chicanes/narrowings	Urban	18	189	86	0.46
Gateways	Urban	3	48	15	0.31
Guard-rail and pedestrian barriers	Urban	13	161	86	0.53
Pedestrian crossings	Urban	70	808	490	0.61
New roundabouts and mini-roundabouts	Urban	56	526	288	0.55
Splitter islands	Urban	5	38	27	0.71
Mass Action Schemes	Urban	40	644	472	0.73
Route Action Schemes	Urban	40	535	320	0.60
Area-wide schemes	Urban	9	153	81	0.53
Cycle schemes	Combined	5	150	52	0.35
Anti-skid surfaces	Rural	4	40	15	0.38
Guard-rail and pedestrian barriers	Rural	3	49	26	0.53
Pedestrian crossings	Rural	2	18	3	0.17
New roundabouts and mini-roundabouts	Rural	15	216	45	0.21
Speed tables	Rural	1	1	1	1.00
Mass Action Schemes	Rural	11	151	50	0.33
Route Action Schemes	Rural	15	128	64	0.50
Area-wide schemes	Rural	1	23	3	0.13

Huang et al (2001)

Huang et al (2001) evaluated the safety impacts of lane reduction measures on 12 streets in Washington and California. The treatment was the conversion of undivided roads with four, 3.35 metre lanes to a 3.66 metre wide 2WLTL, two 3.35 metre wide through lanes (one in each direction), and two 1.5 metre bicycle lanes. Sites were selected based on availability of data, and input from local transportation officials. The study methodology was a before-after study with a comparison group that was matched on functional classification, development, speed limit, intersection spacing, and access control. A total of 25 comparison sites were used.

Ideally, three years of before and after data were used in the analysis, a minimum of one year of before and after data was required. Crashes that occurred at the intersections on either end of the three-lane section were excluded from the analysis. Huang et al used the metrics of crash frequency, crash rate, crash severity, and crash type. The conclusions are as follows:

- Crash frequency on the three-lane streets were found to be 6% lower than the crash frequency on the comparison (four-lane) streets;
- Crash rates between the before and after periods did not change significantly at either the treatment or comparison sites;
- The treatment did not impact crash severity; and
- The treatment did not impact the distribution of crash types.

Corkle et al (2001)

Corkle et al (2001) investigated the safety impacts of traffic calming at seven locations in Minnesota. The study methodology was a naïve before-after study of mean speed, 85th percentile speed, and reduction in proportion of vehicles traveling faster than the "before" 85th percentile speed. The sites were selected because they were about to undergo reconstruction to install the traffic calming. The results of the analysis are shown in Table10.10.

Speed Humps

City of Vancouver (1999)

The City of Vancouver (1999) undertook a pilot study of speed humps on 10 local, urban streets that were selected by considering operating speed, traffic volumes, and proximity to pedestrian generators. Two hump designs were tested; one for streets with 30 km/h speed limits, and one for streets with 50 km/h speed limits. The evaluation methodology was a naïve before-after study of 85th percentile speeds. The results indicate that 85th percentile speeds were reduced by an average of 11 km/h.

Transport Research Laboratory (2000)

The MOLASSES database from the United Kingdom (Transport Research Laboratory, 2000) contains information on the safety effects of road humps in urban, local road projects. The database includes 10 different projects where road humps were implemented. Crashes were reduced from 107 in the three-year "before" period to 12 in the three year "after" period for a CMF of 0.11. The study methodology is a naïve before-after study and the exceptional good results likely overestimate the true safety effects.

TABLE 10.10: Impact of Traffic Calming on Speeds in Minnesota

Site	Description	AADT	Device	Direction	Mean Before	Mean After	85th Before	85th After	Exceeding the "before" 85th
1	Local residential street posted at 30 mph	950 – 1050	Landscaped Choker (34 ft to 22 ft)	East	34	30	36	32	1
				West	34	31	39	35	2
2	Local residential street posted at 30 mph	950 – 1050	"SLOW" Pavement Markings in both directions	East	29	30*	33	35	25
				West	28	28*	32	31	12
3	Local residential street posted at 30 mph	950 – 1050	Landscaped Choker (34 ft to 22 ft)	East	33	30	36	33	7
				West	32	31*	37	35	8
4	Collector street with residential development	4000	Converging chevron pattern and "30 MPH" pavement markings	East	37	31	42	35	0
				West	35	31	39	34	2
5	Minor arterial posted at 35 mph	5400 - 9100	Convert from 4 to 3 lanes	East+	45	44	51	50	N/A
				West+	45	43	51	49	N/A
6	Arterial posted at 30 mph in a commercial area	14500	Convert from 4 to 3 lanes, chokers, and landscaping	East+	30	26	35	30	0 to 5
				West+	28	26	32	30	0 to 13
7	Residential street posted at 30 mph	1600	Raised pedestrian crossing and edge striping	North	34	22	38	26	0
				South	33	23	37	28	0

* Not statistically significant at the 95% level of confidence
+ Average of speeds taken at four different locations along the traffic calmed section

Transverse Rumble Strips

Kermit and Hein (1962)

Kermit and Hein (1962) looked into the safety effects of transverse rumble strips at four locations in Contra Costa, California. All of the test locations were rural approaches to intersections; two "T" intersections, one "Y" intersection, and one cross intersection. The rumble strip patterns varied slightly between applications, but were substantially similar. The study used a naïve before-after methodology with crash rate as the metric. The results are shown in Table 10.11.

TABLE 10.11: Safety Effects of Transverse Rumble Strips in California

Site	Crash Rate		CMF	Proportion Fatal + Injury (%)		Change
	Before	After		Before	After	
1	2.5	0.4	0.16	67	0	-67
2	4.9	2.0	0.41	86	0	-86
3	4.2	1.0	0.24	75	100	+25
4	1.7	0.0	0.00	50	--	--
Average	3.32	0.85	0.26	69	33	-42

Before crash rates were based on 20 to 32 months of data; after crash rates used 10 to 31 months of data. The study did not use a control group to account for confounding factors. However, the author notes that crashes on County roads have been increasing.

Owens (1967)

An evaluation of the safety impacts of transverse rumble strips at six rural, stop-controlled intersections in Minnesota was carried out by Owens (1967). The study used a naïve before-after methodology with crash frequency as the metric. The sites were high-speed intersections, with at least 1000 feet of unobstructed visibility on all approaches. Traffic control devices conformed to the then current MUTCD, in addition to all approaches having "STOP AHEAD" pavement markings.

The treatment consists of rumble strips placed in the following pattern:

- Four strips 25 ft long spaced 100 ft apart;
- Six strips 25 ft long spaced 50 ft apart; and
- One 50 ft long strip at the intersection.

Only two of the intersections had been installed such that at least one year of "after" data was available (the remainder of the intersections were studied for approach speed, stop sign compliance, etc.). The results of the crash analysis is shown in Table 10.12.

TABLE 10.12: Safety Impacts of Transverse Rumble Strips in Minnesota

Approach			Crashes/year	CMF

	Approach ADT	Intersection Type	Before	After	
A1	640	Cross	2.0	1.0	0.50
A2	960	Cross			
C	715	T	6.0	0.0	0.00
Total			8.0	1.0	0.13

It was noted by the author that the approach ADT did not change between the before and after period.

Carstens (1983)

Carstens (1983) reviewed the impacts of transverse rumble strips on rural, stop-controlled intersections in Iowa, and found practically no effect on safety. The methodology was a before-after study with a control group using crash rates. There was 111 locations in each of the treatment and control groups. The results are in Table 10.13.

TABLE 10.13: Safety Impacts of Transverse Rumble Strips in Iowa

Site	Crash Rate		CMF
	Before	After	
Treatment	1.000	0.352	0.35
Control	0.793	0.304	0.38

Helliar-Symons (1981)

The safety impacts of transverse pavement markings (visual rumble strips) at the approaches to roundabouts in Scotland and England were investigated by Helliar-Symons (1981). The treatment was a series of transverse yellow lines painted across the approach lanes to 48 roundabouts, and 2 severe horizontal curves. There are 30 individual markings that start approximately 220 metres upstream of the hazard, and end about 35 metres upstream of the hazard. The spacing between the lines decreases as one approaches the hazard. Sites were selected and allocated to the control and treatment groups randomly, so regression to the mean effects should not confound the results.

The study was a before-after analysis of crash occurrence and severity using a control group and a classical experimental design (i.e., random selection and allocation) to control for regression-to-the-mean. Target crashes were "speed-related" crashes on the approach to the "hazard". All other crashes were termed "non-relevant". Ninety-two percent of the target crashes was either single vehicle, out-of-control crashes at the roundabout, a vehicle failing to stop and colliding with a vehicle on the roundabout, or a two-vehicle crash where the first vehicle yields upon entry to the roundabout and is struck by the second.

The effects of the transverse pavement markings on crash frequency is as shown in Table 10.14. Two years of before and two years of after data were used in the analysis. Property damage only crashes are not included in the analysis because of different reporting thresholds across jurisdictions. Using the non-target crashes as a control, the CMF for speed-related crashes produced by the transverse pavement markings is 0.43. This result is statistically significant to a 99% level of confidence.

TABLE 10.14: Safety Effects of Transverse Rumble Strips on Approaches to Roundabouts in the United Kingdom

Crash Type	**Before**	**After**
Target (speed-related)	96	47
Non-target	265	303

Similar results were found when examining target crashes at the identified test and control sites (CMF=0.41 with a 99% level of significance). The CMF for all crashes on the treated approach was likewise found to be 0.48.

In examining crash severity, Helliar-Symons found that fatal and serious-injury (target) crashes were significantly reduced (CMF=0.26). Slight injury crashes that were speed-related were also reduced CMF=0.48. Finally, the author examined crash occurrence within a one-kilometre square area of the roundabout to check for crash migration; a 12% overall reduction in crashes was observed. This reduction was not statistically significant, nonetheless it is a strong indication that crash migration did not occur.

Harwood (1993)

Harwood (1993) in undertaking an analysis of transverse rumble strip usage concluded the following…

> *The author of this report indicates that most of the before and after studies are small, not statistically significant, poorly designed and difficult to quantify. Given the limitations of the information available, the author of the report was able to draw only limited conclusions. He indicated that:*
>
> - *Despite the lack of rigor in their accident evaluation designs, the study results in the literature generally indicate that rumble strip installation in the travel lane can be effective in reducing accidents. However, the study results are not reliable enough to quantify the expected accident reduction effectiveness.*

- *Rumble strip installation in the travel lane should be considered at locations where rear-end accidents and ran-STOP-sign accidents involving an apparent lack of driver attention are prevalent.*
- *Care should be taken not to overuse rumble strips by placing them in too many locations in the travel lane.*
- *Normally, placement of the rumble strips in the travel lane should be considered only where a documented accident problem exists and only after more conventional treatments, such as signing, have been tried and been found to be ineffective (Harwood pp. 11-12).*

Chapter 11:

Other Safety Issues

CHAPTER 11: OTHER SAFETY ISSUES

In research associated with the safety effect of conversion to all-direction stop control, Persaud (1986) used the dataset to investigate other traffic operational safety issues such as variability of CMFs, safety migration, novelty effect, and proliferation of devices on crash occurrence. The data set was from Philadelphia and included 893 intersections in a one-way street grid, many of which were converted to all-direction stop control (see Table 11.1).

TABLE 11.1: Intersection Control in the Study Area

Intersection Control	No. of Intersections	
	Before	After
One-way stop	419	191
All-way stop	99	321
Traffic Signal	375	381
Total	893	893

Variability of CMFs

The issue investigated in this portion of the analysis is whether CMFs for a particular treatment is relatively constant across locations. The hypothesis is that those locations that experience higher numbers of crashes derive greater benefits from treatment than similar locations with lower crash frequencies. This belief is propagated in most "accident warrants" which specify a minimum crash frequency before installation is warranted.

The Philadelphia dataset clearly demonstrates a relationship between CMF and the expected annual number of crashes; as the expected number of crashes increases the CMF also increases. The relationship between expected crashes and CMF is exponential with the rate of increase declining as the expected number of crashes increases.

Safety Migration

Persaud (1986) used the Philadelphia dataset to examine the expected number of crashes at 61 intersections that were converted to all-direction stop, and 277 intersections that were unchanged in a one-year before and one-year after period. All of the intersections were located in the same area, such that if safety migration occurred it was reasonable to expect that the crashes would migrate from the treated to the unchanged intersections. The results of the analysis are shown in Table 11.2 and seem to show some support for the existence of safety migration. Persaud is quick to point out that 0.3 migrated collisions per intersection is a relatively small number and to determine the cause (i.e., changed travel patterns, driver unfamiliarity, etc.) would require further analysis.

TABLE 11.2: Crash Migration

	Treated	Untreated
No. of Intersections	61	277
Crashes recorded before	219	445
Crashes expected after	168	493
Crashes Recorded after	72	575
Change	96	-82

Novelty Effect

A change in traffic control is often associated with a period of driver uncertainty and adjustment. This is particularly valid on commuter routes where frequent motorists are not as attentive to traffic control, as the controls are known, or thought to be known from past driving experience. New controls are potential violations on a priori driver expectancies and may lead to erratic manoeuvres and crashes.

To examine the decline in safety that is thought to accommodate a change in traffic control, Persaud (1986) examined the effectiveness of all-direction stop control using "after" data collected immediately following conversion, compared to "after" data collected no sooner than six months after conversion. By omitting the first six months of "after" data in the second dataset, the novelty effect may be determined. The results of the analysis are shown in Table 11.3, and indicate that there is little or no novelty effect, or that the overall gain in safety more than compensates for the short-term confusion caused by the introduction of a new device.

TABLE 11.3: Novelty Effect Associated with Conversion to All-Way Stop Control

Crash Type	CMF		Difference
	0 months	6 months	
Right-angle	0.21	0.24	0.03
Rear-end	0.83	0.86	0.03
Fixed Object	1.31	1.40	0.09
Pedestrian	0.61	0.54	0.07
Injury	0.27	0.35	0.08
Total	0.55	0.57	0.02

Proliferation

Again using the Philadelphia dataset for conversion to all-way stop control, Persaud (1986) examined the safety effectiveness of all-way stops by year of installation. As 222 intersections were converted over four years, there was a good faith belief that the all-way stops that were installed at the end of the conversion phase would be less effective because of the proliferation of all-way stops in the area. The data do not seem to support this hypothesis (see Table 11.4).

TABLE 11.4: Safety Effects of Sign Proliferation

Crash Type	CMF			
	Year 1 (74 sites)	Year 2 (67 sites)	Year 3 (38 sites)	Year 4 (43 sites)
Right-angle	0.24	0.18	0.18	0.20
Rear-end	0.77	0.70	1.23	0.79
Fixed Object	1.27	1.43	1.15	1.33
Pedestrian	0.70	0.55	0.50	0.65
Injury	0.26	0.33	0.21	0.27
Total	0.55	0.57	0.50	0.50

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Chapter 12: Summary

CHAPTER 12: SUMMARY

Those studies that have been identified through the literature search, and have yielded at least some basic CMFs or SPFs for use by the practitioner, are summarized in Tables 12.1 to 12..8. The reader is cautioned against use of the recorded results without referring to either the critical review provided in the body of this Synthesis, or the original report/article.

TABLE 12.1: CMFs for Intersection Control Changes

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Agent (1987)	Stop → Stop+Beacon Stop → Signal	Cross-sectional of crash frequency	64 73	1.09 1.09	Rural, high-speed locations
	Addition of Beacon to Stop sign	Naïve before-after of crash frequency	11	0.91	
	Stop → Signal		16	1.38	
	Stop+Beacon → Signal		20	0.79	
Lalani (1991)	Stop → Signal	Naïve before-after of crash frequency	4	0.18	One year before and after periods, sites selected for "high" crash frequency
Poch and Mannering (1996)	Uncontrolled → Stop control	Regression modelling	63	2.12	Applies to an intersection approach
	Uncontrolled → Signal (two-phase operation)			2.01	
	Uncontrolled → Signal (eight-phase operation)			1.77	
Tople (1998)	Stop → Signal	Naïve before-after of crash frequency	9	0.74	Sites selected as part of safety improvement program
TRL (2000)	Urban signalization Rural signalization	Naïve before-after of crash frequency	26 8	0.45 0.22	
Thomas and Smith (2001)	Signalization	Naïve before-after of crash frequency and severity	0	N/A	Fatal
			7	1.29 to 0.00*	Major Injury
			16	1.42 to 0.43	Minor Injury
			13	2.00 to 0.88	Possible Injury
			14	0.71 to 0.48	PDO

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
			15	0.34 to 0.16	Right-angle
			12	1.25 to 0.68	Rear-end
			12	1.82 to 0.71	Left-turn
			15	0.92 to 0.48	Other
			15	0.93 to 0.53	All Crashes
Thomas and Smith (2001)	Signalization + Left-turn lanes	Naïve before-after of crash frequency and severity	3	N/A	Fatal
			9	N/A	Major Injury
			8	0.45 to 0.23	Minor injury
			11	1.13 to 0.34	Possible injury
			11	1.32 to 0.57	PDO
			11	0.52 to 0.22	Right-angle
			11	2.02 to 0.86	Rear-end
			11	0.00 to 0.30	Left-turn
			11	1.16 to 0.50	All Crashes
Region of Waterloo (2001)	Signalization	Naïve before-after of crash frequency	3	0.50	One year before and after periods of sites selected for high crash risk
Lovell and Hauer (1986)	Two-way → All-way stop control	Before-after study using likelihood functions	360	0.28	Right-angle crashes
				0.87	Rear-end crashes
				0.80	Left-turn crashes
				0.61	Pedestrian crashes
				0.29	Injury crashes
				0.53	All crashes
Laplante and Kropidlowski (1992)	Two-way → All-way stop control (warranted)	Naïve before-after of crash frequency	16	0.29	All
				0.21	Angle crashes
			14	0.65	All

<table>
<tr><th>Researcher</th><th>Treatment</th><th>Study Methodology</th><th>No. Sites</th><th>CMF</th><th>Comments</th></tr>
<tr><td rowspan="5"></td><td>Two-way → All-way stop control (unwarranted)</td><td>Naïve before-after of crash frequency</td><td></td><td>0.57</td><td>Angle</td></tr>
<tr><td rowspan="2">Two-way → All-way stop control (unwarranted) with ADT < 12,000</td><td rowspan="2">Naïve before-after of crash frequency</td><td rowspan="2">3</td><td>0.25</td><td>All</td></tr>
<tr><td>0.30</td><td>Angle</td></tr>
<tr><td rowspan="2">Two-way → All-way stop control (unwarranted) with ADT > 12,000</td><td rowspan="2">Naïve before-after of crash frequency</td><td rowspan="2">3</td><td>1.71</td><td>All</td></tr>
<tr><td>1.29</td><td>Angle</td></tr>
<tr><td>Main (1984)</td><td>Regular pattern of stop signs in a residential area (by changing the stop direction at some intersections)</td><td>Naïve before-after of crash frequency</td><td>9</td><td>0.76</td><td>All crashes</td></tr>
<tr><td>Laplante and Kropidlowski (1992)</td><td>Regular pattern of stop signs in a residential area (converting uncontrolled to stop controlled intersections)</td><td>Naïve before-after of crash frequency</td><td>9</td><td>0.15</td><td>All crashes</td></tr>
<tr><td rowspan="2">Pant et al (1999)</td><td>Stop → Stop + Beacon (adequate sight distance)</td><td>Cross-sectional study of crash rate</td><td>7</td><td>0.95
1.12</td><td>Angle crashes
All crashes</td></tr>
<tr><td>Stop → Stop + Beacon (inadequate sight distance)</td><td>Cross-sectional study of crash rate</td><td>6</td><td>1.60
1.01</td><td>Angle crashes
All crashes</td></tr>
</table>

* CMFs are the bounds of the 90% confidence interval

Safety Performance Functions for Intersections

Ministry of Transportation for Ontario (1998)
Ontario
Signalized intersections

Four approaches: $N = 0.0002283\ AADT^{0.54866}$ for fatal crashes
$N = 0.0103469\ AADT^{0.54866}$ for injury crashes
$N = 0.0169214\ AADT^{0.54866}$ for property damage only crashes

Three approaches: $N = 0.0000853\ AADT^{0.54925}$ for fatal crashes
$N = 0.0038654\ AADT^{0.54925}$ for injury crashes
$N = 0.0063216\ AADT^{0.54925}$ for property damage only crashes

where: N = annual number of crashes
AADT = Average annual daily traffic of the main road

Sayed and Rodriguez (1999)
British Columbia
Unsignalized intersections

Four approaches: $N = 1.5406\ (AADT_{major}/1000)^{0.4489}\ (AADT_{minor}/1000)^{0.675}$
Three approaches: $N = 0.9333\ (AADT_{major}/1000)^{0.4531}\ (AADT_{minor}/1000)^{0.5806}$

where: N = Crashes per 3 years
$AADT_{major}$ = Average annual daily traffic of the major road
$AADT_{minor}$ = Average annual daily traffic of the minor road

Vogt (1999)
Michigan and California

For four lane main roads, with stop-controlled two-lane minor roads and three approaches:

$$N = 0.000000192\ ADT_m^{1.433}\ ADT_s^{0.269}\ \exp(-0.0612M + 0.0560D)$$

where:
N = Number of crashes per year
ADT_m = Average two-way major road traffic per day
ADT_s = Average two-way side street traffic per day
M = Median width on the major road (metres)
D = Number of driveways on the major road within 76 metres of the intersection centre

For four lane main roads, with stop-controlled two-lane minor roads and four approaches:

$$N = 0.0000777\ ADT_m^{0.850}\ ADT_s^{0.329}\ \exp(0.110PL - 0.484L)$$

where:
N = Number of crashes per year
ADT_m = Average two-way major road traffic per day
ADT_s = Average two-way side street traffic per day
PL = Proportion of peak hour traffic approaching on the major road that is turning left (%)
L = 0 if major road has no left-turn lane; 1 if at least one left-turn lane.

For the signalized intersection of two-lane roads with four approaches:

$$N = 0.000955\ ADT_m^{0.620}\ ADT_s^{0.395}\ \exp(-0.0142PL_s + 0.0315T) * \exp(-0.675L_T + 0.130V)$$

where: N = Number of crashes per year
ADT_m = Average two-way major road traffic per day
ADT_s = Average two-way side street traffic per day
PL_s = Proportion of peak hour traffic approaching on the side street that is turning left (%)
T = Proportion of peak hour traffic approaching the intersection that consists of trucks (%)
L_T = 0 if the major road does not have a protected left turn; 1 if the major road has at least one protected turn phase
$V = 0.5 * (V_m + V_s)$
V_m = the sum of the absolute percent grade change per 100 feet for each vertical curve along the major road, any portion of which is within 800 feet of the intersection centre, divided by the number of such curves
V_s = the sum of the absolute percent grade change per 100 feet for each vertical curve along the side street, any portion of which is within 800 feet of the intersection centre, divided by the number of such curves

Bauer and Harwood (2000)
California
Urban, four-leg intersections

Stop-controlled: $N = 0.009429\ ADT_{main}^{0.620}\ ADT_{side}^{0.281}\ e^{-0.941X1}\ e^{-0.097X2}\ e^{0.401X3}\ e^{0.120X4} e^{-0.437X5}\ e^{-0.384X6}\ e^{-0.160X7}\ e^{-0.153X8}\ e^{-0.229X9}$

where: X1 = 0 if main road left-turns are permitted; 1 otherwise
X2 = Average lane width on main road (metres)
X3 = 1 if the number of lanes on main road is 3 or less; 0 otherwise
X4 = 1 if the number of lanes on main road is 4 or 5; 0 otherwise
X5 = 1 if no access control on main road; 0 otherwise
X6 = 1 if right-turn is NOT free flow from main road; 0 otherwise
X7 = 1 if no illumination; 0 otherwise
X8 = 1 if the main road is a minor arterial; 0 otherwise
X9 = 1 if the main road is a major collector; 0 otherwise

Signal-controlled: $N = 0.032452\ ADT_{main}^{0.503}\ ADT_{side}^{0.224}\ e^{0.063X1} e^{0.622X2} e^{-0.200X3} e^{-0.310X4} e^{-0.130X5} e^{-0.053X6}\ e^{-0.115X7}\ e^{-0.225X8}\ e^{-0.130X9}$

where:
X1 = 1 if pre-timed signal; 0 otherwise
X2 = 1 if fully-actuated signal; 0 otherwise
X3 = 0 if two-phase signal; 1 otherwise
X4 = 1 if no access control on main road; 0 otherwise
X5 = 1 if 3 or less lanes on the side road; 0 otherwise
X6 = Average lane width on main road (metres)
X7 = 0 if no free flow right turn from main road; 1 otherwise
X8 = 1 if 3 or less lanes on the main road; 0 otherwise
X9 = 1 if 4 or 5 lanes on the main road; 0 otherwise

Harwood et al (2000)
Minnesota, California, and Michigan
Rural intersections with four, two-lane approaches

Stop-control: $N = \exp(-9.34 + 0.60 \mathit{ln}\ ADT_{main} + 0.61 \mathit{ln}\ ADT_{side} + 0.13ND - 0.0054SKEW)$

where:
N = annual number of crashes
ADT_{main} = Average daily traffic on the main road
ADT_{side} = Average daily traffic on the minor road
ND = number of driveways on the major road legs within 76 metres of the intersection
SKEW = intersection angle (degrees) expressed as one-half of the angle to the right minus one-half of the angle to the left for the angles between the major road leg in the direction of increasing stations and the right and left legs, respectively.[6]

[6] In most instances SKEW is computed as the absolute value of the angle of intersection minus 90 degrees.

Signal Control: $N = \exp(-5.46 + 0.60 \mathit{ln}\ ADT_{main} + 0.20 \mathit{ln}\ ADT_{side} - 0.40PL - 0.018LT + 0.11V + 0.026T + 0.041ND)$

where:

N = annual number of crashes
ADT_{main} = Average daily traffic on the main road
ADT_{side} = Average daily traffic on the minor road
PL = 0 if no protected left-turn phasing on major road; 1 otherwise
LT = Proportion of minor road traffic turning left during AM and PM peak combined (%)
V = grade rate for all vertical curves within 76 metres of the intersection along the main and minor roads
T = Proportion of trucks entering in the AM and PM peak hours combined (%)
ND = number of driveways on the major road legs within 76 metres of the intersection

Region of Durham (2001)
Region of Durham, Ontario

Signalized Intersections

Three Approaches:

$N = 0.0771\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ Casualty crashes in CBD
$N = 0.1440\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ PDO crashes in CBD
$N = 0.0822\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ Casualty crashes in Suburban
$N = 0.1390\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ PDO crashes in Suburban
$N = 0.0347\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ Casualty crashes in Rural
$N = 0.0662\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ PDO crashes in Rural
$N = 0.0740\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ Casualty crashes in Semiurban
$N = 0.1810\ (AADT_{major} + AADT_{minor})^{0.304}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.157}$ PDO crashes in Semiurban

Four Approaches:

$N = 0.00000144\ AADT_{major}^{1.111}\ AADT_{minor}^{0.373}$	Casualty crashes in CBD
$N = 0.00000324\ AADT_{major}^{1.111}\ AADT_{minor}^{0.373}$	PDO crashes in CBD
$N = 0.00000126\ AADT_{major}^{1.111}\ AADT_{minor}^{0.373}$	Casualty crashes in Semiurban
$N = 0.00000313\ AADT_{major}^{1.111}\ AADT_{minor}^{0.373}$	PDO crashes in Semiurban
$N = 0.0000711\ (AADT_{major} + AADT_{minor})^{b0.997}$	Casualty crashes in Suburban
$N = 0.0001570\ (AADT_{major} + AADT_{minor})^{0.997}$	PDO crashes in Suburban
$N = 0.0001040\ (AADT_{major} + AADT_{minor})^{0.997}$	Casualty crashes in Rural
$N = 0.0001620\ (AADT_{major} + AADT_{minor})^{0.997}$	PDO crashes in Rural

Unsignalized Intersections

Three Approaches:

$N = 0.000003420\ AADT_{major}^{1.021}\ AADT_{minor}^{0.219}$	Casualty crashes in CBD
$N = 0.000007890\ AADT_{major}^{1.021}\ AADT_{minor}^{0.219}$	PDO crashes in CBD
$N = 0.000000638\ AADT_{major}^{1.152}\ AADT_{minor}^{0.292}$	Casualty crashes in Suburban
$N = 0.000001560\ AADT_{major}^{1.152}\ AADT_{minor}^{0.292}$	PDO crashes in Suburban
$N = 0.000041800\ AADT_{major}^{0.598}\ AADT_{minor}^{0.484}$	Casualty crashes in Rural
$N = 0.000090300\ AADT_{major}^{0.598}\ AADT_{minor}^{0.484}$	PDO crashes in Rural
$N = 0.000002310\ AADT_{major}^{1.021}\ AADT_{minor}^{0.219}$	Casualty crashes in Semiurban
$N = 0.000005390\ AADT_{major}^{1.021}\ AADT_{minor}^{0.219}$	PDO crashes in Semiurban

Four Approaches:

$N = 0.00317\ (AADT_{major} + AADT_{minor})^{0.676}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$	Casualty crashes in CBD
$N = 0.01200\ (AADT_{major} + AADT_{minor})^{0.676}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$	PDO crashes in CBD
$N = 0.00239\ (AADT_{major} + AADT_{minor})^{0.676}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$	Casualty crashes in Suburban
$N = 0.00496\ (AADT_{major} + AADT_{minor})^{0.676}\ (AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$	PDO crashes in Suburban

$N = 0.00325\,(AADT_{major} + AADT_{minor})^{0.676}\,(AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$ Casualty crashes in Rural
$N = 0.00516\,(AADT_{major} + AADT_{minor})^{0.676}\,(AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$ PDO crashes in Rural
$N = 0.00293\,(AADT_{major} + AADT_{minor})^{0.676}\,(AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$ Casualty crashes in Semiurban
$N = 0.00603\,(AADT_{major} + AADT_{minor})^{0.676}\,(AADT_{minor}/(AADT_{minor} + AADT_{major}))^{0.450}$ PDO crashes in Semiurban

where: $AADT_{major}$ = Total entering AADT on major road
$AADT_{minor}$ = Total entering AADT on minor road

Region of Halton (2001)
Signalized Intersections
Region of Halton, Ontario

Three Approaches:

$N = 0.000070\ TOTAL^{0.934}\ RATIO^{0.165}$ Casualty crashes
$N = 0.000250\ TOTAL^{0.934}\ RATIO^{0.165}$ PDO crashes

Four Approaches:

$N = 0.00810\ TOTAL^{0.591}\ RATIO^{0.688}$ Casualty crashes in Urban/suburban
$N = 0.02320\ TOTAL^{0.591}\ RATIO^{0.688}$ PDO crashes in Urban/suburban
$N = 0.00104\ TOTAL^{0.581}\ RATIO^{-0.940}$ Casualty crashes in Rural
$N = 0.00316\ TOTAL^{0.581}\ RATIO^{-0.940}$ PDO crashes in Rural

Unsignalized Intersections

Three Approaches:

$N = 0.00250\ TOTAL^{0.614}\ RATIO^{0.5253}$ Casualty crashes
$N = 0.00732\ TOTAL^{0.614}\ RATIO^{0.5253}$ PDO crashes

Four Approaches:

$N = 0.00072\ TOTAL^{0.838}\ RATIO^{0.591}$ Casualty crashes
$N = 0.00164\ TOTAL^{0.838}\ RATIO^{0.591}$ PDO crashes

where: $TOTAL = AADT_{main} + AADT_{minor}$
$RATIO = AADT_{minor} / TOTAL$
$AADT_{main}$ = Average daily traffic entering from the main road
$AADT_{minor}$ =Average daily traffic entering from the minor road

Pernia et al (2002)
Florida
Unsignalized Intersections

$$N = \exp(0.6827 + 0.2777ADT + 0.1193LU + 0.1705L - 0.1695SL + 0.2752M - 0.1679S)$$

where: N = Annual number of crashes
ADT = 0 if ADT < 15,000; 2 if ADT > 30,000; 1 otherwise
LU = 1 if urban; 0 otherwise
L = 1 if > 4 lanes on major road; 0 otherwise
SL = 1 if posted speed limit on major road > 45 mph; 0 otherwise
M = 1 if median on the major road; 0 otherwise
S = 1 if paved shoulders; 0 otherwise

Signalized Intersections

$$N = \exp(0.5718 + 0.4868ADT + 0.0949LU + 0.1728L + 0.1845M - 0.1102S)$$

where:
N = Annual number of crashes
ADT = 0 if ADT < 15,000; 2 if ADT > 30,000; 1 otherwise
LU = 1 if urban; 0 otherwise
L = 1 if > 4 lanes on major road; 0 otherwise
M = 1 if median on the major road; 0 otherwise
S = 1 if paved shoulders; 0 otherwise

TABLE 12.2: CMFs for Traffic Signal Timing and Design Changes

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Tople (1998)	Signal upgrading	Naïve before-after study of crash frequency	6	0.66 0.56	All crashes EPDO Crashes
TRL (2000)	Signal modification	Naïve before-after study of crash frequency	80 (urban) 10 (rural)	0.62 0.49	
Cottrell (1995)	Addition of white strobe light to supplement red indication	Naïve before-after study of crash frequency	6	1.42 0.85 1.27	Rear-end crashes Angle crashes All crashes
Sayed et al (1998)	Increase size of amber and red lenses and add reflective tape to the backboard	Before-after with control group using EB methods	10	0.91 0.79	Casualty crashes All crashes
Region of Waterloo (2001)	New signal heads and revised timing	Naïve before-after study of crash frequency	2	0.43	
Bhesania (1991)	Replace post-mounted signal heads with mast arm mounted heads, and add a one second all-red interval	Naïve before-after study of crash frequency	5	0.37 0.81 1.35 0.73 0.75	Right-angle Rear-end Left-turn Other All crashes
Thomas and Smith (2001)	Replace pedestal-mounted heads with mast arm-mounted heads	Naïve before-after study of crash frequency	31	0.35 to 0.21*	Right angle
			32	1.51 to 0.90	Rear-end
			24	1.23 to 0.81	Left-turn
			31	0.82 to 0.64	Other
			31	0.72 to 0.57	All crashes

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Hamilton Associates (1998)	Additional primary signal head	Cross-sectional study of crash rates	63	0.91 0.72 0.78	Casualty crashes PDO All Crashes
		Before-after using EB methods	8	0.83 to 0.79 0.69 to 0.64 0.78 to 0.72	Casualty crashes PDO All Crashes
Polanis (1998)	Replace eight inch lenses with 12 inch lenses	Naïve before-after of crash frequency	38	0.52 0.84	Angle crashes All crashes
Sayed et al (1999)	Addition of advance warning signs	Before-after using EB methods	25	1.03 to 0.92 0.91 to 0.86 0.92 to 0.82	Rear-end crashes Casualty crashes All crashes
Gibby et al (1992)	Addition of advance warning signs	Cross-section study of crash rates	85	3.37	
	Addition of advance flashers		77	1.35	
	Addition of advance warning sign and flasher		14	1.87	
Lalani (1991)	Improved clearance timing	Naïve before-after study of crash frequency	3	0.50	No details of exact treatment, small sample size
	Improved signal coordination		3	0.75	No details of exact treatment, small sample size
Greiwe (1986)	Convert two-phase operation to split (three) phase operation	Naïve before-after study of crash frequency	8	0.22 0.67 0.76 0.46	Left-turn crashes Angle crashes Rear-end crashes All crashes

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
	Remove unwarranted protected left-turn phase		14	1.24 0.78 0.77 0.95	Left-turn crashes Angle crashes Rear-end crashes All crashes
Hummer et al (1991)	Convert leading to lagging left-turn phase	Cross-section study of crash rates	29	0.67	Left-turn crashes
Upchurch (1991)	Permissive → leading protected/permissive	Naïve before-after of left-turn crash rates	224	1.19	Left-turn crashes
	Permissive → lagging protected/permissive		206	0.69	
	Permissive → leading protected		219	0.35	
	Permissive → lagging protected		166	0.14	
	Permissive → leading protected/permissive		17	0.73	Two opposing lanes
	Permissive → lagging protected/permissive		9	0.76	
	Leading protected/ permissive → Permissive		14	1.29	
	Lead protected/ permissive → Lagging protected/ permissive		35	0.73	
	Leading protected → Leading protected/ permissive		3	3.34	

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
	Leading protected → Lagging protected/permissive		6	4.13	
	Leading protected → Lagging protected		10	1.31	
	Permissive → Leading protected/permissive		3	1.20	Three opposing lanes
	Permissive → Lagging protected/permissive		8	0.16	
	Permissive → Leading protected		3	0.02	
	Leading protected/permissive → permissive		3	2.60	
	Leading protected/ permissive → Lagging protected/ permissive		38	0.60	
	Leading protected/ permissive → Leading protected		2	0.11	
	Leading protected → Leading protected/permissive		22	3.37	
	Leading protected → Lagging protected/permissive		9	0.48	
	Leading protected → Lagging protected		12	1.00	

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Bamfo and Hauer (1997)	Fixed time to vehicle actuated operation	Cross-section using EB methods	306	0.87	Right-angle crashes
Shebeeb (1995)	Permissive → leading protected/ permissive	Cross-section using left-turn crash rate	78	0.72	Left-turn crashes
	Permissive → Lagging protected/permissive		61	1.24	
	Permissive → Leading protected		83	0.34	
	Permissive → Lagging protected		51	0.44	
Stamatiadis et al (1997)	Permissive → protected/ permissive	Cross-section using left-turn crash rate	100 164	0.25 0.15	One opposing lane Two opposing lanes
	Permissive → Protected	Cross-section using left-turn crash rate	129 150	0.37 0.61	One opposing lane Two opposing lanes
Tarall and Dixon (1998)	Protected/permissive → Protected	Naïve before-after using left-turn conflicts	?	0.23	At double left-turn lanes
Vogt (1999)	Protect left-turn phase on the main road	Cross sectional study using EB methods	49	0.51	Intersection of rural, two-lane roads
Thomas and Smith (2001)	Adding left-turn phasing	Naïve before-after study using crash frequency	4	0.77 to 0.52*	Details of phasing change not provided
	Adding a left-turn lane and left-turn phasing	Naïve before-after study using crash frequency	7	0.54 to 0.30*	Details of phasing change not provided

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Chin and Quddus (2001)	Pretimed → Adaptive signal timing	Cross-section study using EB methods	52	0.87	
Polanis (2002)	Removal of red/amber night-time flashing operation	Before-after study with control group	19	0.27	Applicable to night-time, right-angle crashes

* 90% CONFIDENCE LIMITS

TABLE 12.3: CMFs for Traffic Signs

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Lyles et al (1986)	Area-wide traffic sign upgrades	Before-after study with control group using crash frequency	--	1.04	
Tople (1998)	Improved traffic signing	Naïve before-after study using crash frequency	6	0.95 0.85	All crashes EPDO crashes
TRL (2000)	Traffic signing	Naïve before-after study using crash frequency	222 (Urban) 136 (Rural)	0.68 0.59	No details the signing upgrades
Arnott (1985)	Activated Curve Speed Warning Signs	Naïve before-after study using crash frequency	5	0.29	Short after periods and no accounting for confounders
Tribbett et al (2000)	Dynamic Curve Warning Sign	Naïve before-after study of crash frequency	5	1.25 0.89 1.02	Casualty crashes PDO crashes All crashes
Lalani (1991)	New chevron signs	Naïve before-after study using crash frequency	3	0.29	
Land Transport Safety Authority of New Zealand (1996)	Chevron signs	Before-after study with some adjustments for crash trends	103	0.52 0.46	Rural areas Urban areas
Helliar-Symon and Ray (1986)	Active warning signs for following too close	Before-after study with a control group	3	3.31 1.32	Close following crashes All crashes
Kostyniuk and Cleveland (1986)	"Limited Sight Distance" sign	Before-after study with a control group	9 paired sites	1.09	

TABLE 12.4: CMFs for Pavement Markings

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
TRL (2000)	Improved markings	Naïve before-after study of crash frequency	196 74	0.70 0.62	Urban Rural
	Yellow bar markings	Naïve before-after study of crash frequency	2 2	0.27 0.63	Urban Rural
Migletz and Graham (2002)	Replacing conventional with longer lasting pavement markings	Naïve before-after study of crash rates	55	0.89 1.15 0.94	Dry pavement Wet pavement All crashes
Willis et al (1984)	Edgelines	Before-after study using a control group	75	1.02 1.14	Visibility is cause for caution Visibility is mixed
Cottrell (1987)	200 mm wide edgelines	Before-after study with a control group	3	No significant effect	Applicable to run-off-the-road crashes
Hall (1987)	200 mm wide edgelines	Before-after study with a control group	69	1.4 to 1.6	Applicable to run-off-the-road crashes
Lee et al (1997)	Reflectivity of longitudinal pavement markings	Before-after study	46	No significant effect	Applicable to night-time crashes
Retting et al (1997)	Placement of right-through arrow pavement markings in right lane near driveways	Naïve before-after study of conflict frequency	3 1	0.96 1.06	Unsignalized driveways Signalized driveways

TABLE 12.5: CMFs for Pedestrian Safety Measures

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
iTrans (2002)	Pedestrian refuge island → Split Pedestrian Crossover	Cross-sectional study using crash frequency	50	5.14	Did not account for pedestrians volumes
	Pedestrian refuge island installation	Naïve before-after study using crash frequency	30	0.27 2.23	Pedestrian crashes All crashes
Van Houten (unpublished)	Pedestrian activated flashing beacons with supplementary signs	Naïve before-after study using conflicts	2	0.22 to 0.25	Pedestrian-vehicle conflicts

TABLE 12.6: CMFs for Legislation and Enforcement

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Ullman and Dudek (1987)	Speed limit reduction in urban fringe areas	Naïve before-after study of crash rates	6	1.03 1.00	Casualty crashes All crashes
Merriam (1993)	Speed limit reductions	Naïve before-after study of crash rates	13 12 5	0.93 0.63 1.50	80 → 60 km/h 80 → 70 km/h 60 → 50 km/h
City of Winnipeg (1991)	Speed limit reduction in urban area	Before-after study with a control group	2	0.86 to 0.95	
Vogt and Bared (1999)	10 km/h speed limit reduction	Cross-section study using EB methods	389	0.85	Applies to crashes at rural intersections with three approaches
Hadi et al (1995)	1 mph speed limit reduction	Cross-section study using EB methods	---	0.97 to 1.00 0.96 to 1.00	Rural Urban
Main (1984)	Prohibition of curbside parking	Naïve before-after study of crash rate	6	0.63	
McCoy et al (1990)	Converting parallel to angle parking	Cross-sectional study of crash rates	---	1.69 2.42	Low angle parking High angle parking
Hocherman et al (1990)	Converting to one-way operation	Cross-sectional study using mid-block crash rates	---	0.88 1.63	CBD area Non-CBD
Ontario Provincial Police (1998)	Stepped-up police enforcement	Naïve before-after study using crash frequency	1	0.90	Casualty crashes
Royal Canadian Mounted Police (1999)	Stepped-up police enforcement	Naïve before-after study using crash frequency	1	0.10 0.87 0.82	Fatal crashes Injury crashes PDO crashes

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Eger (2002)	Police enforcement	Cross-sectional study using EB methods	---	0.98^{N} 0.9973^{Ns}	N is the number of police officers available; Ns is the number of county sheriff officers available

TABLE 12.7: CMFs for Turn Lanes

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Main (1984)	Adding exclusive left-turn lanes with raised medians at signalized intersections	Naïve before-after study of crash rates	8	0.45	
Greiwe (1986)	Restriping four lane approaches to include a left-turn lane	Naïve before-after study of crash frequency	8	0.41 0.46 0.56 0.43	Left-turn crashes Angle crashes Rear-end crashes All crashes
	Provision of opposing left-turn lanes and signal modernization	Naïve before-after study of crash frequency	5	0.22 0.15 0.77 0.33	Left-turn crashes Angle crashes Rear-end crashes All crashes
Tople (1998)	Restriping to include left-turn lanes	Naïve before-after study of crash frequency	2	0.65 0.27	All crashes EPDO crashes
	Reconstruction to include left-turn lanes	Naïve before-after study of crash frequency	3	0.69 0.84	All crashes EPDO crashes
Vogt (1999)	Provision of left-turn lane on main street	Regression analysis using negative binomial model	72	0.62	Applies to rural, stop controlled intersections with four approaches
Rimiller et al (2001)	Provision of left-turn lane	Cross-sectional study using EB methods	13	0.98 to 0.41	

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Thomas and Smith (2001)	Provision of left-turn lanes	Naïve before-after study of crash frequency	8	2.19 to 0.61* 1.06 to 0.49 3.61 to 0.92 0.87 to 0.60 1.12 to 0.64	Angle crashes Rear-end crashes Left-turn crashes Other crashes All crashes
Region of Waterloo (2001)	Provision of left-turn and right-turn lanes on one approach	Naïve before-after study of crash frequency	1	0.44	Very small sample
Harwood et al (2002)	Provision of one left-turn lane on major road approach	Before-after study using EB methods	280 treated 300 control	0.56 (rural) 0.67 (urban)	Three approaches, stop controlled
				0.85 (rural) 0.93 (urban)	Three approaches, signalized
				0.72 (rural) 0.73 (urban)	Four approaches, stop controlled
				0.82 (rural) 0.90 (urban)	Four approaches, signalized
	Provision of left-turn lanes on both major street approaches			0.52 (rural) 0.53 (urban)	Stop controlled
				0.67 (rural) 0.81 (urban)	Signalized
Hoffman (1974)	Provision of a 2WLTL on a four lane arterial	Naïve before-after study of crash frequency	---	0.55 0.38 1.14 0.93 1.06 0.67	Head-on crashes Rear-end crashes Angle crashes Sideswipe crashes Other crashes All crashes
Main (1984)	Provision of a 2WLTL on a four lane arterial	Naïve before-after study of crash rate	4	0.51	

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Yagar and Van Aerde (1984)	Provision of a 2WLTL	Naïve before-after study of crash frequency	14	0.64	Left-turn crashes from 2WLTL
			13	0.78	Left-turn crashes to 2WLTL
			18	0.64	Left-turn crashes to and from 2WLTL
			21	0.72	All crashes
Lalani (1991)	Provision of a 2WLTL	Naïve before-after study of crash frequency	5	0.42	
Tople (1998)	Restriping to provide a 2WLTL	Naïve before-after study of crash frequency	3	0.91 0.77	All crashes EPDO crashes
	Reconstructing to provide a 2WLTL	Naïve before-after study of crash frequency	5	0.92 1.40	All crashes EPDO crashes
Brown and Tarko (1999)	Provision of a 2WLTL	Cross-sectional study using EB methods	155	0.50 0.42 0.47	Casualty crashes PDO crashes All crashes
Bauer and Harwood (2000)	Provision of a right-turn lane	Cross-sectional study using EB methods	---	0.85	Rural, 4-leg, stop controlled
			---	1.12	Urban, 4-leg, signalized, channelized on main road

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
			---	1.75 (rural) 1.47 (urban)	4-leg, stop controlled, channelized on cross road
	Provision of a left-turn lane on the main road		---	0.81 (rural) 0.98 (urban)	3-leg, stop controlled, painted
			---	0.91 (rural) 1.21 (urban)	Urban, 3-leg, stop controlled, curbed
Harwood et al (2000)	Provision of right-turn lane on rural two-lane roads	Expert opinion from a review of several studies	---	0.95 (stop) 0.975 (signal)	Right-turn lane on one main street approach
				0.90 (stop) 0.95 (signal)	Right-turn lanes on both main street approaches
Chin and Quddus (2001)	Provision of turn lanes on approaches to signalized intersections	Cross-sectional study using EB methods	---	1.38 0.47	Left-turn lanes Right-turn lanes
Harwood et al (2002)	Provision of right-turn lane on major street	Before-after study using EB methods	280 treated 300 control	0.86 0.96	Stop controlled Signalized
	Provision of right-turn lanes on both major street approaches			0.74 0.92	Stop controlled Signalized

* 90% Confidence intervals

Safety Performance Functions

Bonneson and McCoy (1997)
Nebraska and Arizona
Urban and Suburban Arterial Roads

Undivided roads: $N = ADT^{(0.91+1.021Ir)}\ L^{0.852}\ e^{(-14.15 - 10.504Ir + 0.57Ip + 0.0077(DD+SD)Ib + 0.0255PDO)}$

2WLTL roads: $N = ADT^{0.91}\ L^{0.852}\ e^{(-14.15 + 0.018Ib - 0.093Ir + 0.0077(DD+SD)Ib + 0.0255PDO)}$

where:
N = Annual number of crashes
ADT = Annual daily traffic
L = Length of street (metres)
DD = driveway density (/km)
SD = unsignalized intersection density (/km)
PDO = proportion of property damage only crashes (%)
Ib = Business land use (=1 if business or office use, =0 otherwise)
Ir = Residential land use (=1 if residential or industrial, =0 otherwise)
Ip = Parking (=1 if parallel, curbside parking permitted, =0 otherwise)

Harwood et al (2000)
Two-lane rural highways

$$CMF = 1 - 0.7\ P_{lt/d}\ \frac{0.0047D + 0.0024D^2}{1.199 + 0.0047D + 0.0024D^2}$$

where:
$P_{lt/d}$ = proportion of driveway-related crashes that are left-turn crashes susceptible to relief by a 2WLTL expressed as a decimal
D = Driveway density (driveways/mile)

TABLE 12.8: CMFs for Traffic Calming

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
Lynam et al (1988) and Mackie et al (1990)	Area-wide traffic calming	Before-after study with control group	5	0.87 to 0.88	
Engel and Thomsen (1992)	Traffic calming	Before-after study with control group	45 treated 52 control	0.27	A variety of treatments were used with either a 15 km/h or 30 km/h posted speed limit
Hamilton Associates (1996)	Traffic calming	Naïve before-after study of crash frequency	4	0.61	
Ewing (1999)	Traffic calming (primarily traffic circles and speed humps)	Naïve before-after study of crash frequency	55	0.96	Amalgam of studies
TRL (2000)	Chicanes/narrowings	Naïve before-after study of crash frequency	18	0.46	Urban areas
	Gateways		3	0.31	Urban areas
	Guardrails and pedestrian barriers		13 (urban) 3 (rural)	0.53 0.53	
	Pedestrian crossings		70 (urban) 2 (rural)	0.61 0.17	
	Roundabouts/ mini roundabouts		56 (urban) 15 (rural)	0.55 0.21	
	Splitter islands		5	0.71	Urban areas
	Mass action schemes		40 (urban) 11 (rural)	0.73 0.33	

Researcher	Treatment	Study Methodology	No. Sites	CMF	Comments
	Area-wide traffic calming		9 (urban) 1 (rural)	0.53 0.13	
	Cycle schemes		5	0.35	Rural areas
	Anti-skid surfaces		4	0.38	Rural areas
	Speed tables		1	1.00	Rural areas
	Route action scheme		15	0.50	Rural areas
Huang et al (2001)	Conversion of undivided four lane road, to a two-lane road with a 2WLTL and bicycle lanes	Before-after study with control group using crash rates	12 treated 25 control	1.00	
TRL (2000)	Speed humps in urban areas on local roads	Naïve before-after study of crash frequency	10	0.11	
Kermit and Hein (1962)	Transverse rumble strips on the approach to a rural intersection	Naïve before-after study using crash rates	4	0.26	
Owens (1967)	Transverse rumble strips on the approach to a rural intersection	Naïve before-after study of crash frequency	3	0.13	
Carstens (1983)	Transverse rumble strips on the approach to a rural intersection	Before-after study with a control group using crash rates	111 treated 111 control	0.92	
Helliar-Symons (1981)	Transverse pavement markings (visual rumble strips) on approach to roundabout	Before-after study with control group using crash frequency	50 treated	0.43 to 0.41	Apply to speed-related crashes

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References

REFERENCES

Abbess C, Jarrett D, and Wright CC (1981) "Accidents at Blackspots: Estimating the Effectiveness of Remedial Treatment, With Special Reference to the `Regression-to-mean' Effect", Traffic Engineering & Control, 22(10).

Abdulsattar HN, Tarawneh MS, McCoy PT, and Kachman SD (1996) "Effect on Vehicle-Pedestrian Conflicts of 'Turning Traffic Must Yield to Pedestrian' Signs", Transportation Research Record 1553, pp. 38-45.

Agent KR (1988) "Traffic Control and Accidents at Rural High-Speed Intersections", Transportation Research Record 1160, Transportation Research Board, National Academies of Science, Washington DC, pp. 14-21.

Arnott D (1985) "An Analysis of Traffic Activated Curve Speed Warning Signs in Ontario", Masters of Engineering Thesis, McMaster University, Hamilton, Ontario, 93pp.

Austrian Road Safety Board (1999) "Investigations on Influences Upon Driver Behaviour - Safety Approaches in Comparison and Combination", Final Report, Guarding Automobile Drivers through Guidance Education and Technology (GADGET), Vienna, Austria.

Bamfo JK and Hauer E (1997) "Which is Safer in Terms of Right-angle Vehicle Accidents? Fixed-time or Vehicle-actuated Signal Control" Proceedings of the Canadian Multidisciplinary Road Safety Conference, Canadian Association of Road Safety Professionals, pg 352-360.

Bauer KM, and Harwood DW (2000) "Statistical Models of At-Grade Intersection Accidents – Addendum", FHWA-RD-99-094, Federal Highway Administration, Office of Safety and Traffic Operations Research and Development, Washington, DC.

Beenstock M, Gafni D and Goldin E (1999) "The Effect of Traffic Policing on Road Safety in Israel", Hebrew University of Jerusalum, Israel, http://atar.mscc.huji.ac.il/~economics/facultye/beenstock/manta.pdf, accessed on January 29, 2003.

Belanger C (1994) "Estimation of Safety of Four-legged Unsignalized Intersections" Transportation Research Record 1467, Transportation Research Board, National Academy Press, Washington DC, pp. 23-29.

Berger WJ and Linauer M (1998) "Raised Traffic Islands at City Limits – Their Effect on Speed", Proceedings of 1998 Meeting of the International Cooperation on Theories and Concepts in Traffic Safety, Budapest.

Bhesania RP (1991) "Impact of Mast-Mounted Signal Heads on Accident Reduction", ITE Journal, Institute of Transportation Engineers, Washington, DC, pp. 25-29.

Bloch SA (1998) “Comparative Study of Speed Reduction Effects of Photo-radar and Speed Display Boards”, Transportation Research Record 1640, Transportation Research Board, National Academies of Science, pp. 27-36.

Bonneson JA and McCoy PT (1997) “Effect of Median Treatment on Urban Arterial Safety: An Accident Prediction Model”, Paper No. 970101, Preprint for the 76th Annual Meeting of the Transportation Research Board, Washington, DC.

Brown HC and Tarko AP (1999) “Effects of Access Control on Safety on Urban Arterial Streets”, Transportation Research Record 1665, Transportation Research Board, National Academies of Science, Washington, DC, pp. 68-74.

Canadian Council of Motor Transport Administrators (2000) “Road Safety Vision 2001 - 2000 Update”, Ministry of Public Works and Government Services, Ottawa, Ontario, ISBN 0-662-29902-7.

Carstens RL (1983) “Safety Effects of Rumble Strips on Secondary Roads”, Transportation Research Record 926, Transportation Research Board, National Academies of Science, Washington DC, pp. 12-15.

Cebryk G and Boston G (1996) “School Zone Pilot Project and Evaluation”, Report to the City of Edmonton Utilities & Public Works Committee of City Council, August 15, 1996, Transportation Department, File: 96TR3462.

Chin HC and Quddus MA (2001) “A Crash Prediction Model for Signalized Intersections”, Proceedings of the Canadian Multidisciplinary Road Safety Conference XII, Canadian Association of Road Safety Professionals, London, Ontario, pp. 1-8.

Corkle J, Giese JL, and Marti MM (2001) “Investigating the Effectiveness of traffic Calming Strategies on Driver Behavior, Traffic Flow, and Speed”, Final Report No. MN/RC-2002-02, Minnesota Local Road Research Board, Minnesota Department of Transportation, St. Paul, Minnesota.

Cottrell BH (1987) “Evaluation of Wide Edgelines on Two-lane Rural Roads”, Transportation Research Record 1160, Transportation Research Board, National Academies of Science, Washington DC, pp. 35-42.

Cottrell BH (1995) “Evaluation of Strobe Lights in Red Lens of Traffic Signals”, Transportation Research Record 1495, Transportation Research Board, National Academy of Science, Washington, DC., pp. 36-40.

Durham, Region of (2001) “Safety Improvement Program and Software”, Final Report, Regional Municipality of Durham, Public Works Department.

Edmonton, City of (2002) “178 Street Speed Limit (S. Mandel)”, Report to the Transportation and Public Works Committee, Agenda Item No.: D.2.a.

Eger RJ (2002) “Injury Crashes: The Role of Law Enforcement”, a paper submitted for presentation at the 81st Annual Meeting of the Transportation Research Board, Washington, DC.

Engel U and Thomsen LK (1992) “Safety Effects of Speed Reducing Measures in Danish Residential Areas”, Accident Analysis and Prevention 24(1), Permagon Press, UK, pp. 17-28.

Ewing (1999) “Traffic Calming: State of the Practice”, Informational Report IR-098, Institute of Transportation Engineers, Washington, DC.

Forbes GJ (2002) “Community Safety Zones – State-of-the-Practice”, Intus Road Safety Engineering Inc., prepared for the City of Hamilton, unpublished.

Forbes G and Gill T (2000) “Arterial Speed Calming – Mohawk Road Case Study”, Urban Street Symposium Conference Proceedings, Transportation Research E-Circular E-C019, Transportation Research Board, National Academies of Science, Washington, DC, pp. I-2/1 – I/2-7.

Gibby AR, Washington SP, and Ferrara TC (1992) “Evaluation of High-speed Isolated Signalized Intersections in California”, Transportation Research Record 1376, Transportation Research Board, National Academies of Science, Washington, DC.

Greiwe RR (1986) “Intersection Management Techniques for the Left-Turning Vehicle: The Indianapolis Experience”, ITE Journal, Institute of Transportation Engineers, Washing, DC, pp. 23-28.

Hadi MA, Aruldhas J, Chow LF, and Wattleworth JA (1995) “Estimating Safety Effects of Cross-Section Design for Various Highway Types Using Negative Binomial Regression”, Transportation Research Record 1500, Transportation Research Board, National Academy of Science, Washington, DC., pp. 169-177.

Hall JW (1987) “Evaluation of Wide Edgelines” Transportation Research Record 1114, Transportation Research Board, National Academy of Science, Washington, DC., pp. 21-30.

Halton, Region of (2001) “Safety Improvement Program and Software”, Regional Municipality of Halton, Planning and Public Works Department, April 2001.

Hamilton Associates (1989) “Traffic Conflict Procedure Manual”, Second Edition, Insurance Corporation of British Columbia.

Hamilton Associates (1996) “The Safety Benefits of Traffic Calming”, Insurance Corporation of British Columbia, February 1996.

Hamilton Associates (1997) “Update to the Economic Evaluation Methodologies for Road Improvement Programs”, Insurance Corporation of British Columbia, April 1997.

Hamilton Associates (1998) “The Safety Benefits of Additional Primary Signal Heads”, Insurance Corporation of British Columbia, January 1998.

Harwood D (1993) “Synthesis of Highway Practice 191 – Use of Rumble Strips to Enhance Safety”, Transportation Research Board, National Academies of Science, Washington, DC.

Harwood DW, Council FM, Hauer E, Hughes WE, and Vogt A (2000) “Prediction of the Expected Safety Performance of Rural Two-lane Highways”, Technical Report FHWA-RD-99-207, Federal Highway Administration, United States Department of Transportation, Washington DC, 197 pages.

Harwood DW, Bauer KM, Potts IB, Torbic DJ, Richard KR, Kohlman-Rabbani ER, Hauer E, and Elefteriadou L (2002) “Safety Effectiveness of Intersection Left- and Right-turn Lanes”, Federal Highway Administration Report, United States Department of Transportation, Final Report No. FHWA-RD-02-089.

Haselton CB, Gibby AR, Ferrara TC, and Khorashadi AK (2001) “Methodologies Used to Analyze Collision Experience Associated with Speed Limit Changes on Selected California Highways”, paper to be presented at the 81st Annual Meeting of the Transportation Research Board, Washington, DC.

Hauer E. (1997) “Observational Before-After Studies in Road Safety: Estimating the Effect of Highway and Traffic Engineering Measures on Road Safety”, Elsevier Science Inc., Tarrytown, NY, ISBN 0-08-043 053 8.

Helliar-Symons RD (1981) “Yellow Bar Experimental Carriageway Markings – Accident Study”, TRRL Laboratory Report 1010, Transport and Road Research Laboratory, Crowthorne, Berkshire, UK, 17pp.

Helliar-Symons RD and Ray SD (1986) “Automatic Close-following Warning Sign – Further Trials”, Research Report 63, Transport and Road Research Laboratory, Crowthorne, Berkshire, UK, 12pp.

Hemsing S and Forbes G (2000) “Ottawa-Carleton Traffic Calming Evaluation Study”, Final Report, Synectics Transportation Consultants Inc., prepared for the Region of Ottawa-Carleton and the City of Ottawa.

Hill AB (1965) “The environment and disease: association or causation?” Proceedings of the Royal Society of Medicine 58; 295-300.

Hocherman I, Hakkert AS, and Bar-Ziv J (1990) “Safety of One-way Urban Streets”, Transportation Research Record 1270, Transportation Research Board, National Academies of Science, Washington, DC.

Hoffman MR (1974) “Two-way Left-turn Lanes Work!”, Traffic Engineering, Institute of Transportation Engineers, Washington, DC, pp. 24-27.

Huang HF, Stewart JR, and Zeeger C (2001) “Evaluation of Lane Reduction ‘Road Diet’ Measures on Crashes and Injuries”, paper to be presented at the 81st Annual Meeting of the Transportation Research Board, Washington, DC.

Hummer JE, Montgomery RE, and Sinha KC (1991) “Guidelines for Use of Leading and Lagging Left-turn Signal Phasing”, Transportation Research Record 1324, Transportation Research Board, National Academies of Science, Washington, DC, pp. 11-20.

IBI Group (1997) “Safety, Speed & Speed Management: A Canadian Review”, Final Report, produced for Transport Canada.

iTrans Consulting Inc. (2002) “Review of Refuge Islands and Split Pedestrian Crossovers – Phase 2”, Final Report, prepared for the City of Toronto.

Kermit ML and Hein TC (1962) “Effect of Rumble Strips on Traffic Control and Driver Behaviour”, Highway Research Board Proceedings, Highway Research Board, National Academies of Science, pp. 469-482.

Knoblauch RL and Raymond PD (2000) “The Effect of Crosswalk Markings on Vehicle Speeds in Maryland, Virginia, and Arizona”, Research Report FHWA-RD-00-101, Federal Highway Administration, McLean, Virginia, 9 pp.

Kostyniuk LP and Cleveland DE (1986) “Sight Distance, Signing, and Safety on Vertical Curves”, ITE Journal, Institute of Transportation Engineers, Washington, DC, pp 25-28.

Lalani N (1991) “Comprehensive Safety Program Produces Dramatic Results”, ITE Journal, Institute of Transportation Engineers, Washington, DC, pp. 31-34.

Land Transport Safety Authority (1996) “Treatment at Bends Using Chevrons”, http://www.ltsa.govt.nz/roads/crash_reduction_programme.html, accessed on November 30, 2002.

Laplante JN and Kropidlowski CR (1992) “Stop Sign Warrants: Time For Change”, ITE Journal, Institute of Transportation Engineers, Washington, DC, pp 25-29.

Lee JT, Maleck TL, and Taylor WC (1977) "Analysis of the Correlation Between Pavement Marking Visibility and Night-time Accidents", Paper presented to the 77th Annual Meeting of the Transportation Research Board, National Academies of Science, Washington DC.

Liu GX and Popoff A (1997) "Provincial-wide Travel Speed and Traffic Safety Study in Saskatchewan", Transportation Research Record 1595, Transportation Research Board, National Academies of Science, Washington, DC, pp. 8-13.

Lovell J and Hauer E (1986) "The Safety Effect of Conversion to All-way Stop Control", Transportation Research Record 1068, Transportation Research Board, National Academies of Science, Washington, DC., pp. 103-107.

Lyles RW, Lighthizer DR, Drakopoulos A, and Woods S (1986) "Efficacy of Jurisdiction-Wide Traffic Control Device Upgradings", Transportation Research Record 1068, Transportation Research Board, National Academies of Science, Washington, DC., pp. 34-41.

Lynam DA, Mackie AM, and Davies CH (1988) "Urban Safety Project: 1. Design and Implementation of Schemes", Research Report 153, Transport and Road Research Laboratory, Crowthorne, Berkshire, UK, 25pp.

Lyon C and Persaud B (2002) "A Pedestrian Collision Model For Urban 4-leg Signalized Intersections", Proceedings of the Canadian Multidisciplinary Road Safety Conference XII, June 10-13, 2001, London, Ontario.

Mackie AM, Ward HA, and Walker RT (1990) "Urban Safety Project: 3. Overall Evaluation of Area Wide Schemes", Research Report 263, Transport and Road Research Laboratory, Crowthorne, Berkshire, UK, 26pp.

Main MF (1984) "Four Underutilized Collision Reduction Measures", in Transport Risk Assessment: Proceedings of a Symposium on Risk in Transport, Third Symposium of Institute for Risk Research, edited by Sam Yagar, University of Waterloo Press, pp. 89-119.

McCoy PT, Ramanujam M, Moussavi M, and Ballard JL (1990) "Safety Comparison of Types of Parking on Urban Streets in Nebraska", Transportation Research Record 1270, Transportation Research Board, National Academies of Science, Washington, DC.

Merriam J (1993) "The Effect of Posted Speed Limit Reductions on Collision Rates", Region of Hamilton-Wentworth Traffic Department, unpublished.

Migletz J and Graham J (2002) "Long-Term Pavement Marking Practices", National Cooperative Highway Research Program Synthesis of Practice Report No. 306, Transportation Research Board, National Academies of Science, Washington, DC.

Ministry of Transportation for Ontario (1998) “The Science of Highway Safety: Network Evaluation and Safety Conscious Procedures”, Ministry of Transportation for Ontario, Traffic Office, Traffic Safety Management Section, Toronto, Ontario.

Mounce, J.M. (1981) “Driver Compliance with Stop-Sign Control at Low-Volume Intersections”, Transportation Research Record 808, Transportation Research Board, National Academies of Science, Washington, D.C., pp. XX

Ontario Provincial Police (1998) “Safe on Seven, 1998”, from the Community Based Traffic Safety – Best Practices Database, Transport Canada, http://www.tc.gc.ca/roadsafety/NPA/detail_e.asp?ID=42, accessed on June 4, 2002.

Owens RD (1967) “Effect of Rumble Strips at Rural Stop Locations on Traffic Operation”, Highway Research Record 170, Highway Research Board, National Academies of Science, Washington, DC, pp. 35-55.

Pant PD, Park Y, Neti SV, and Hossain AB (1999) “Comparative Study of Rural Stop-controlled and Beacon-controlled Intersections”, Transportation Research Record 1692, Transportation Research Board, National Academies of Science, Washington, DC, pp. 164-172.

Parker MR (1997) “Effects of Raising and Lowering Speed Limits on Selected Roadway Sections”, FHWA Report No. FHWA-RD-92-084. Federal Highway Administration, United States Department of Transportation, Washington, DC.

Parker MR and Zeeger CV (1989) “Traffic Conflict Technique for Safety and Operation Engineers Guide”, Report FHWA-IP-88-026, Federal Highway Administration, United States Department of Transportation, Washington, DC.

Pernia JC, Lu JJ, Xie X, Weng M, and Snyder D. (2002) “Development of Models to Quantify the Impacts of Signalization on Intersection Crashes”, Paper submitted for presentation at the 81st Annual Meeting of the Transportation Research Board, Washington, DC.

Persaud BN (1986) “Safety Migration, the Influence of Traffic Volumes, and Other Issues in Evaluating Safety Effectiveness – Some Findings on Conversion of Intersections to Multiway Stop Control”, Transportation Research Record 1068, Transportation Research Board, National Academies of Science, Washington, DC.

Poch M and Mannering F (1996) “Negative Binomial Analysis of Intersection-Accident Frequencies”, Journal of Transportation Engineering, American Society of Civil Engineers, Volume 122(2), pp. 105-113.

Polanis SF (1998) “Do 12“ Signal Lenses Reduce Angle Crashes?”, ITE Annual Meeting, Compendium of Papers CD-ROM, Toronto, Ontario.

Polanis SF (2002) “Right-angle Crashes and Late-night/Early-morning Flashing Operation: 19 Case Studies”, ITE Journal, Institute of Transportation Engineers, Washington, DC, pp.26-28.

Retting R, Greene MA, and Van Houten J (1997) “Use of Pavement Markings to Reduce Rear-end Conflicts at Commercial Driveway Locations”, Transportation Research Record 1605, Transportation Research Board, National Academies of Science, Washington DC, pp. 106-110.

Rimiller J, Ivan J, and Garrick N (2001) “Estimating Benefits from Specific Highway Safety Improvements: Phase III – Benefits of Lane Widening and Adding Left Turn Lanes”, Submitted for presentation at the 81st Annual Meeting of the Transportation Research Board, Washington, DC.

Royal Canadian Mounted Police (1999) “Highway 43 Safety Initiative, 1999”, from the Community Based Traffic Safety – Best Practices Database, Transport Canada, http://www.tc.gc.ca/roadsafety/NPA/detail_e.asp?ID=24, accessed on June 4, 2002.

Salman NK and Al-Maita KJ (1995) “Safety Evaluation at Three-Leg, Unsignalized Intersections by Traffic Conflict Technique”, Transportation Research Record 1485, Transportation Research Board, National Academies of Science, Washington DC, pp. 177-185.

Sayed T, Abdelwahab, and Nepomuceno J (1998) “Safety Evaluation of Alternative Signal Head Design”, Transportation Research Record 1635, Transportation Research Board, National Academies of Science, Washington, DC, pp. 140-146.

Sayed T and Rodriguez F (1999) “Accident Prediction Models for Urban Unsignalized Intersections in British Columbia”, Transportation Research Record 1665, Transportation Research Board, National Academies of Science, Washington, DC, pp.93-99.

Sayed T, Vahidi H, and Rodriguez F (1999) “Advance Warning Flashers – Do they Improve Safety?”, Transportation Research Record 1692, Transportation Research Board, National Academies of Science, Washington, DC, pp. 30-38.

Schrader MH (1999) “Study of Effectiveness of Selected School Zone Traffic Control Devices”, Transportation Research, Record 1692, Transportation Research Record, National Academies of Science, Washington, DC, pp. 24-29.

Shebeeb O. “Safety and Efficiency for Exclusive Left-turn Lanes at Signalized Intersections” ITE Journal, July 1995, pp. 52-59.

Stamatiadis N, Agent K, and Bizakis A (1997) "Guidelines for Left-turn Phasing Treatment", Transportation Research Record 1605, Transportation Research Board, National Academies of Science, Washington, DC, pp.1-7.

Stimpson WA, Kittleson WK, and Berg WD (1977) "Methods for Field Evaluation of Roadway Delineation Treatments", Transportation Research record 621, Transportation Research Board, National Academies of Science, Washington, DC, pp.

Tarrall MB and Dixon KK (1998) "Conflict Analysis for Double Left-turn Lanes with Protected-Plus-Permitted Signal Phases", Transportation Research Record 1635, Transportation Research Board, National Academies of Science, Washington, DC, pp. 105-112.

Thomas GB and Smith DJ (2001) "Effectiveness of Roadway Safety Improvements", Final Report, Center for Transportation Research and Education, Iowa State University, Ames, IA.

Tople AR (1998) "Development of South Dakota Accident Reduction Factors", Final Report, South Dakota Department of Transportation, Office of Research, Pierre, SD.

Transport Research Laboratory (2000), Monitoring of Local Authority Safety Schemes (MOLASSES) Database, Crowthorne, Berkshire, United Kingdom, located at http://www.trl.co.uk/molasses, and accessed on January 30, 2003.

Transportation Association of Canada (2001) "Visibility of Traffic Signal Displays for Aging and Colour Deficient Drivers" Final Report, unpublished.

Transportation Research Board (1998) "Managing Speed: Review of Current Practice for Setting and Enforcing Speed Limits", Transportation Research Board, Special Report 254, Washington, DC.

Tribbett L, McGowen P, and Mounce J (2000) "An Evaluation of Dynamic Curve Warning Systems in the Sacramento River Canyon", Final Report, Western Transportation Institute, Montana State University, prepared for the California Department of Transportation.

Ullman GL and Dudek CL (1987) "Effects of Reduced Speed limits in Rapidly Developing Urban Fringe Areas", Transportation Research Record 1114, Transportation Research Board, National Academies of Science, Washington, DC, pp. 45-53.

Upchurch J (1991) "Comparison of Left-turn Accident rates for Different Types of Left-turn Phasing", Transportation Research Record 1324, Transportation Research Board, National Academies of Science, Washington, DC, pp. 33-40.

Vancouver, City of (1999) “Results of the Speed hump Pilot Project”, Administrative Report to the Standing Committee of Council on Transportation and Traffic, CC File No. 5757, http://www.city.vancouver.bc.ca/ctyclerk/cclerk/991102/tt2.htm, accessed on June 5, 2002.

Van Houten R, Healy K, Malenfant, JEL, and Retting R (Unpublished) “The Use of Signs and Symbols to Increase the Efficacy of Pedestrian Activated Flashing Beacons at Crosswalks”

Vogt A (1999) “Crash Models for Rural Intersections: Four-lane by Two-lane Stop-controlled and Two-lane by Two-lane Signalized”, FHWA-RD-99-128, Federal Highway Administration, Office of Safety Research and Development, Washington, DC.

Vogt A and Bared J (1999) “Accident Models for Two-lane Rural Segments and Intersections”, TRR 1635, TRB, NAS, pp. 18-29.

Washington, University of (2002) “Child Pedestrian Injury Interventions Environmental Changes”, Health Injury Prevention Research Center, http://depts.washington.edu/hiprc/childinjury/topic/pedestrians/environment.html, accessed on June 4, 2002.

Waterloo, Region of (2001) “The Regional Municipality of Waterloo 1999 Collision Report”, Department Report to the Transportation and Environmental Services Committee, Report No. E-01-033, April 3, 2001, prepared by Allister McIlveen.

Wheeler AH (1992) “Advance Stop Line for Cyclists at Oxford, Newark, and Bristol”, Research Report 336, Transport and Road Research Laboratory, Department of Transportation, Crowthorne, Berkshire, UK, 22pp.

Willis PA, Scott PP, and Barnes JW (1984) “Road Edgelining and Accidents: An Experiment in South-west England”, TRRL Laboratory Report 1117, Transport and Road Research Laboratory, Crowthorne, Berkshire, UK, 17pp.

Wilson FR, Stevens AM, and Ambo A (2001) “Safety and Economic Benefits from Speed Reduction in the Province of New Brunswick”, Proceedings of the Canadian Multidisciplinary Road Safety Conference XII, Canadian Association of Road Safety Professionals, London, Ontario.

Winnipeg, City of (1991) "Maximum Speed Limits on Grant Avenue, on Kenaston Boulevard and on Taylor Avenue", Streets and Transportation Department, Report to the Highway Traffic Board, May 29, 1991.

Yagar S, and Van Aerde M (1984) “Reducing Accidents with 2-Way Left Turn Lanes”, Proceedings of the Canadian Multidisciplinary Road Safety Conference, London, Ontario, pp. 106-123.

Zador P, Stein HS, Wright P, and Hall J (1987) “Effects of Chevrons, Post-Mounted Delineators, and Raised Pavement Markers on Driver Behaviour at Roadway Curves”, Transportation Research Record 1114, Transportation Research Board, National Academies of Science, Washington, DC, pp 1-10.

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Appendix A: Evidence-based Road Safety

APPENDIX A – EVIDENCE-BASED ROAD SAFETY

WHAT IS EBRS?

EBRS is defined as:

> *The conscientious and judicious use of current best evidence in providing road safety for individuals, facilities, and transportation systems.*

The practice of EBRS is the integration of the best available information on global safety research with the experience and knowledge of the individual practitioner respecting community values and local policy. The end result of practicing EBRS is informed decision-making respecting road safety matters, where the safety effects of the selected actions and strategies are known, and are compatible with community values.

EBRS is not identifying and selecting the safest operational or control strategy. There must always be due regard for the impacts that these strategies have on other aspects of the community. For instance, the provision of protected left-turn phasing at a traffic signal, while considered a safer alternative than permissive phasing, also increases delay. If the additional delay is such that the level of service at the intersection becomes "unacceptable", the decision to implement the safer alternative may not be appropriate. EBRS may require the use of a permissive phasing. EBRS simply dictates that the competent practitioner will know what the safety consequences of this action are, and make the decision in light of this knowledge.

At the same time, EBRS is not the tacit acceptance of conclusions from poorly conducted research, simply because it is published, reported in a trade journal, or presented at a technical conference. External information and research should always be critically appraised by the practitioner to determine, the validity of the results, and the applicability of the results to the particular situation. For example, a practitioner in a large rural municipality is seeking information on the safety effects of all-way stop control. She finds a report on some well-conducted research from a nearby major metropolitan area that presents a CMF of 0.50. Despite having determined that the research has been well-conducted, the practitioner must also assess whether the results are applicable to her situation, as the research was conducted in an urban/metropolitan location, and her roads are rural.

Why do we need EBRS?

Road safety knowledge is dynamic and we must remind ourselves that keeping abreast of the conventional wisdom is an ongoing, life long, and self-directed process. Research into road safety matters is certainly a growing field, and better and more accurate relationships between interventions and their safety impacts will surely become available. In addition, community values are also dynamic and highly variable. The practice of EBRS means that practitioners need to be aware of these changes and continually reflect them in their daily practice.

The literature abounds with research and reports on the safety effects of various traffic operations and control strategies. The glut of information should not be mistaken for a wealth of knowledge. EBRS requires critical appraisal and judgement in the application of road safety research. The need for EBRS is demonstrated in the following scenario…

> *A municipal traffic engineer is asked by his elected officials to consider signalizing a two-way, stop-controlled urban intersection for safety reasons. The intersection has experienced an average of 4.5 crashes per year for the previous 36 month period and the elected representatives would like to see this safety record improved. The engineer refers to the current edition of the Transportation Association of Canada's Manual on Uniform Traffic Control Devices, which indicates that an average of 5 crashes per year in a 36 month period are required before signalization is "warranted". Therefore, the engineer reports back to the elected officials that signalization is not going to improve the crash record.*

Is the engineer correct in his conclusion? The information presented in the example is insufficient to answer the question. However, the logic behind the conclusion is certainly incorrect. The "accident" warrant requiring an average of five crashes per year over a 36 month period is not to be interpreted as signalized intersections typically have five crashes per year. In fact, at present the five crashes per year threshold is the matter of some scrutiny in both Canada and the United States.

The appropriate approach is for the engineer to develop, re-calibrate or adopt SPFs for unsignalized and signalized intersections and determine the crash count and severity for the prevailing and anticipated conditions. The SPFs will provide the best estimate of safety performance, which can then be evaluated in the context of local policy and community values to determine if signalization is warranted. Alternatively, the analyst can use CMFs for signalization that have been developed for different types of intersection geometry.

How do you practice EBRS?

There are two main concerns that need to be assessed in applying research results to a practical problem facing a practitioner:

1. *Has the research been conducted using sound methods such that the results can be considered valid? and*
2. *Are the results applicable to the particular situation?*

The traffic operations professional is practicing EBRS if (s)he is following these seven steps when dealing with road safety issues:

1. *Identify a problem or area of uncertainty*
2. *Formulate a relevant, focused question that needs to be answered*
3. *Find and appraise the evidence*
4. *Assess the applicability to your situation*
5. *Decide whether or not to take action*
6. *Evaluate the outcomes of your actions/inaction*
7. *Summarize/record the results*

Through this process EBRS considers the following things in a meaningful way:

- *Population: What is the population or site-specific conditions (signalized intersections, rural two-lane roads, etc.)?*
- *Problem: What is the problem (crash types, severity)?*
- *Interventions: What are the interventions being considered (including traffic operations and control strategies, geometric design changes, driver education, and "do nothing")?*
- *Outcomes: Is the intervention effective and do the safety and other impacts concur with operating practice and local policy?*

EBRS is achievable and a necessary component of informed decision-making on road safety matters. The following is an example of the appropriate use of EBRS in practice.

One of the elected officials, on behalf of her constituency, is seeking "back-to-back green arrows" for the left turn movement on the main street at a signalized intersection. The request cites "safety concerns" at this "dangerous location". The traffic operations (i.e., intersection and approach delay, volume to capacity ratio, etc.) under the present two-phase, and the proposed three-phase operation would be satisfactory. Therefore, the traffic operations staff are faced with deciding whether to recommend the left-turn phasing to improve safety.

The seven steps of the EBRS process are used as follows:

1. *Identify a problem or area of uncertainty*
   Does the intersection have a crash problem related to left-turns from the main street? Are left-turn-opposing through crashes over-represented? Are the crashes more severe than expected?

2. *Formulate a relevant, focused question that needs to be answered*
   What impact does a protected left-turn phase on the main street of a four-leg, urban, signalized intersection have on safety? (NOTE: Forming the question includes defining the treatment(s) being considered, and the site characteristics).

3. *Find and appraise the evidence*
   Using conventional literature searches, internet searches, material contained in this Synthesis, personal contact with other professionals, and other methods find information on the safety impacts of protected left-turn phasing. After assembling the available material, use the appraisal form in Appendix B to appraise the quality of the research and assess the reliability of the results.

4. *Assess the applicability to your situation*
   As much as possible determine if the reliable research identified in Step 3 was conducted on similar conditions to the situation of concern to you now. For instance, the current problem is at an urban, four-leg intersection – was the research conducted on similar intersections?

5. *Decide whether or not to take action*
   Will the protected left-turn phasing be effective in addressing the identified problem? If so are there other concerns that may preclude action (i.e., unacceptable environmental damage, violation of local policy, etc.)? Are there other equally effective options that need to be considered?

6. *Evaluate the outcomes of your actions/inaction*
   Monitor the location in the short- and long-terms. If a treatment was implemented, then the long-term evaluation of effectiveness will assist in building local CMFs and evidence-based knowledge.

7. *Summarize/record the results*
   Document the results of the project.

Cause and Effect

One of the key points of EBRS and the critical appraisal of research is the determination of cause-effect relationships. Much emphasis is placed on statistical significance when measuring the safety impacts of a particular traffic operations strategy. While statistical testing and inference are an important clue to a cause-effect relationship they cannot be used alone in determining cause-effect. A statistically significant relationship can only convey that “A” varies with “B”. It does not mean that “A” causes “B”.

Determination of cause and effect must take into consideration much more than the strength of association between a treatment and crash occurrence and severity. The first extensive and likely still the best set of tests to assess a cause-effect relationship was developed by Hill (1965). The tests, somewhat adapted for road safety use, are as follows:

1. Is there strength of the association?
   A statistical correlation should exist between the treatment and safety.

2. Is there consistency?
   The effects of the treatment if examined repeatedly by different persons, in different circumstances and times, should be consistent.

3. Is there specificity?
   The treatment should produce the desired effect, the absence of the treatment should not produce the desired effect (assuming that no other treatment has been applied).

4. Does the treatment and safety have a relationship in time?
   The presence of the treatment must precede the change in safety.

5. Is there a “dose-response” gradient?
   When application of a small amount of the treatment produces a safety impact, it is expected that a larger application would produce a larger impact. This test is useful in many situations but must be used with caution. For instance, there is a safety benefit associated with lane widening from 3.1 metres to 3.3 metres. In addition, we expect an even greater benefit if the lane is widened from 3.1 metres to 3.6 metres. However, there is a limit to the dose-response gradient somewhere around 3.7 metres. Lane widths greater than 3.7 metres provide no additional safety benefit, and in the instance of lanes wider than 4.5 metres may even show a safety disbenefit.

6. Is there logical plausibility?
   The treatment and the safety effect should be connected through a logical etiology. This test is somewhat reliant on the current conventional wisdom.

7. Is there coherence of the evidence?
   The results of the analysis should, in most cases, agree with what is generally known about the treatment and with the likely etiology.

8. Is there corroborating experimental evidence?
   Rarely possible in road safety, although potentially important human performance studies may be used to further support causation. Driving simulators, surveys, and human performance studies that measure crash surrogates with a definitive link to crash causation or increased crash severity are examples of this type of experimental evidence.

9. Can we find applicable analogy?
   If the mechanism through which the treatment acts is analogous to some similar treatment. For instance, the province of Ontario implemented "Community Safety Zones" (CSZs) , which are sections of road that are identified by municipal by-law, where the fines for moving violations are doubled. The CSZs are identified on the street by rectangular black-and-white, regulatory signs posted at the roadside. The effect of CSZs on travel speed is negligible [Forbes, 2002]. The lack of a cause-and-effect between CSZs and speed is further supported by the analogy to speed limits and travel speed, where changes in the posted speed limit (communicated to the motorist through a rectangular black-and-white regulatory sign posted at the roadside) also fail to illicit a change in travel speed.

The application of these tests, and not just identifying a statistical relationship, are the appropriate method of assessing a cause-effect relationship. Of course, there are instances where not all of the tests will point the researcher in the same direction. In instances of paradoxical results among the above tests the practitioner must use his/her discretion in interpreting and applying the research to practical situations.

The Current State of EBRS

EBRS is scientific, and therefore knowledge-based. What has become apparent in the preparation of this Synthesis is that the knowledge base with respect to the safety impacts of traffic operations and control strategies is underdeveloped. This places the practitioner in the uncomfortable position of having to make decisions based on incomplete information. Until sufficient good quality research has been conducted and made available to the practitioner, the practitioner will likely have to rely upon prejudice, hunch, opinion, and guesswork (PHOG).

Appendix B: Critical Reviews

APPENDIX B – CRITICAL REVIEWS

Critically Reviewing Literature

Research into traffic safety is continuous and evolving. The conventional wisdom respecting the safety implications of traffic operations strategies is certain to be advanced over time. As this publication is static and represents what we know about the safety impacts of traffic operations at this time, there is a need for practitioners to supplement the knowledge contained herein with new information as it becomes available. Much of this information is bound to be gleaned from reports, articles, and presentations and must be critically appraised by the practitioner to determine the quality of the research, and the applicability of the findings to his/her situation.

In order to assist the practitioner in this regard, a worksheet for critically evaluating road safety research is provided.

There may be a willingness on the part of the practitioner to accept the published report because information is needed to support a position, and it is easy to use published material in this respect. The practitioner is cautioned against accepting the written work because it supports the position and it is convenient. Much of the literature that permeates the work place is not peer-reviewed, or peer-reviewed by individuals who are not well-versed with road safety matters.

Worksheet for Evaluating an Article About
Road Safety Countermeasures

Document Title: ________________________________
Author: ________________________________
Date/Year: ________________________________
Source: ________________________________

Study Methodology

1. What population are the results applicable to?

Setting:	[ ] Urban	[ ] Rural		
Intersection:	[ ] Signal	[ ] All-way stop	[ ] Two-way stop	[ ] Roundabout
Road Segment:	[ ] Two-lane	[ ] Four-lane	[ ] Freeway	[ ] Divided

2. Is the allocation of sites to treatment and control randomized?
   [ ] Yes [ ] No [ ] Can't Tell

3. Are time-trends in crash data properly accounted for?
   [ ] Yes [ ] No [ ] Can't Tell

4. Is regression-to-the-mean properly accounted for?
   [ ] Yes [ ] No [ ] Can't Tell

5. Are appropriate statistical methods used?
   [ ] Yes [ ] No [ ] Can't Tell

6. Does the study design properly account for extraneous influences on safety? (i.e., changes traffic volume, environmental conditions, etc.)
   [ ] Yes [ ] No [ ] Can't Tell

7. Are the groups similar at the start of the trial?
   - Baseline prognostic factors (geometry, traffic control, other known confounders) balanced?
   - If different, are these adjusted for?

     [ ] Yes [ ] No [ ] Can't Tell

8. Aside from the subject countermeasure(s), are the groups treated equally?
   - Are there any co-interventions? or contaminants?
   - Did the treatment undergo any changes during the study? (i.e., vandalism, modifications, tweaking)

     [ ] Yes [ ] No [ ] Can't Tell

9. Overall, are the results of the study valid?

   [ ] Yes [ ] No [ ] Can't Tell

Results

1. How large is the treatment effect?
   a. Applies to all crashes or target crashes?
   b. CMF or SPF?
   c. Effect on crash severity?

2. How precise is the estimate of the treatment effect?

Usefulness of Results

1. Can the results be applied to my jurisdiction?
   a. Roads are similar for driving laws and populations, design guidelines, and other important factors?
   b. Lack of any compelling reason why the results would not apply?

   [ ] Yes [ ] No [ ] Can't Tell

2. Are the expected safety benefits worth the potential negative impacts?
    a. Cost, environmental damage, social impacts, etc.?
        [ ] Yes [ ] No [ ] Can't Tell

3. Is the treatment consistent with the system and local policies?
    [ ] Yes [ ] No [ ] Can't Tell

4. Are other treatments available?
    [ ] Yes [ ] No [ ] Can't Tell

Appendix C: Conducting and Authoring Research

APPENDIX C – CONDUCTING AND AUTHORING RESEARCH

In the course of preparing this Synthesis, it became evident that much of the literature that comprises the conventional wisdom is comprised of articles and reports that fail to meet the minimum standards of quality demanded by the community of science. In some instances the substandard work is a result of the study design (i.e., the methods and materials), in others it is simply the incomplete presentation of the results. In either event, the glut of poorly conducted research or inappropriate reporting is at best limiting the advancement of road safety knowledge, and at worst is misleading practitioners.

Conducting Road Safety Research

The proper conduct of road safety research requires forethought and planning. To build the road safety knowledge-base it is essential that the safety efficacy of traffic operations and control strategies be accurately determined. The pace of advancement is significantly reduced if the community relies solely on academics and researchers to undertake this activity. The traffic operations practitioner is almost continuously making changes to the system, whether safety-related or not, and as such has access to a vast storehouse of data. The use of this data to advance the knowledge in road safety should not be limited by inappropriate research methods.

This section provides a discussion on the need for standardized research methods in road safety, suggests some general principles to assist in standardization, and some assistance for the practitioner in conducting safety research.

Proposed Road Safety Research Standards

Road safety is a subset of the public health and injury prevention field. However, the process by which products are approved for use in road safety versus a pharmaceutical product are vastly different. While both protected left turn phasing, and influenza vaccinations have a direct impact on human health (although in different populations), seldom are the two interventions treated similarly. The evaluations of crash countermeasures seldom have the same degree of rigour as initial testing of pharmaceutical products, food additives, and other products intended to protect public health.

Pilot studies are popular fare among practicing traffic operations professionals. New equipment, designs, and features such as red light cameras, traffic calming, and strong yellow-green sheeting are often implemented on a ‘pilot basis’ in municipalities. How ever good intentioned the pilot study, it’s usefulness is often severely hampered by a failure to consider the measures of effectiveness, and the evaluation methodology *a priori*.

As an example, the City of Ottawa and the Region of Ottawa-Carleton implemented traffic calming within the City of Ottawa on a trial basis with the intention of evaluating it's effectiveness. However, the evaluation methodology and metrics were not scripted at the initiation of the study. When the evaluation was undertaken, and the methodology determined, it was found that much of the data required from the 'before' period was not recorded [Hemsing and Forbes, 2000]. The failure to careful script an evaluation methodology at the start of the pilot study limited the conclusions that could be definitively drawn from the pilot study.

Organizations such as the Transportation Association of Canada who produce the standards, guidelines, and practices that govern the day-to-day decisions made by practitioners would certainly benefit from more rigourous road safety research. The Transportation Association of Canada's Manual of Uniform Traffic Control Devices (MUTC) would be greatly enhanced by making safety an explicit consideration in the selection and application of different traffic control devices. Furthermore, if the MUTCD could integrate numerical guidance and "best evidence" respecting safety, then practitioners' decisions would be more informed. In order to achieve these ends, there is a need for standardization on what is considered good practice in conducting road safety research and evaluation of traffic operations and control strategies.

Even if the guideline publishers do not adopt some minimum standards (although it is strongly recommended that they do), the traffic operations practitioner can use these guidelines in conducting their own evaluations and developing a better understanding of the safety implications of various actions.

As a starting point for standardization, the profession may look to the health care industry and their standards for research titled "Good Laboratory Practice" (GLP) which is:

> *A quality system concerned with the organisational process and the conditions under which non-clinical health and environmental safety are planned, performed, monitored, recorded, archived, and reported.*[7]

GLP is intended to cover work performed in the laboratory and in the field as it relates to pre-clinical studies in the pharmaceutical and cosmetic industries. Nonetheless, there is clearly a parallel use in road safety. The profession has developed sound, scientific methods to be used in conducting road safety research. Yet these methods are routinely ignored, either because the researcher is not aware of the proper methods, the researcher is not experienced in the proper methods, and/or the data required to implement best practices is not available.

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[7] "Principles on Good Laboratory Practice" Organization for Economic Co-operation and Development. Environment Directorate, Chemicals Group and Management Committee, Revised 1997.

A lack of data is likely a local consideration and can be remedied by local researchers and practitioners. The ignorance of or inexperience with best practices is a more global concern. It would certainly benefit the road safety community if Canadian national and provincial organizations were to develop and promote a set of best practices for road safety research that is similar to and follows the same basic principles of GLP.

The transportation profession has taken enormous strides towards becoming more scientific in the approach to road safety. Many of the shortcomings and pitfalls associated with the observational studies to evaluate road safety strategies have been recognized and methods developed to deal with them. It is certainly beyond the scope of this document to detail the generic process for conducting road safety research. It suffices to reiterate that sound research is based upon careful forethought respecting the study design and scientific methods.

To gain more insight into the statistical and scientific methods that can be used in road safety research, the reader is directed to the following resources:

- *Hauer E (1997) Observational Before-After Studies in Road Safety – Estimating the Effect of Highway and Traffic Engineering Measures on Road Safety. Elsevier Science Inc., New York, USA, 289pp.*

- *Persaud B (2001) "Statistical Methods in Highway Safety Analysis", NCHRP Synthesis 295, Transportation Research Board, National Academy Press, Washington, DC, 75pp.*

- *"Statistics for Transportation Researchers", NCHRP Report 20-45, Transportation Research Board, National Academies Press, Washington, DC.*

- *Organisation for Economic Co-operation and Development (1997) "Road Safety Principles and Models: Review of Descriptive, Predictive, Risk and Accident Consequence Models", OECD IRRD No. 892483, Paris, France.*

Assistance for the Practitioner

Recognize from the outset that evaluation is not an afterthought, or an activity to be tacked on to the end of a project. A proper and thorough evaluation is anticipated from the start, carefully planned, and ideally documented in a study protocol. A study protocol is a "document that describes the objective(s), design, methodology, statistical considerations, and organization of a trial."[8] The protocol is an essential element of the documentation that is required before an evaluation begins.

[8] Guideline for Good Clinical Practice. International Conference on Harmonisation of Technical Requirements for Registration of Pharmaceuticals for Human Use. May 1996.

In larger jurisdictions, there is no sound reason why the practitioner should not be using "best practices" to conduct safety research. There is certainly a need to do so to assist in making informed technical decisions on the safety implications of day-to-day matters. Furthermore, the data that is required to undertake the appropriate analysis is usually contained in existing files and records. The arguments of insufficient staff time or inadequate knowledge of the appropriate methods are weak, and would not pass the test of *due diligence*.

By suggesting that road safety research be conducted using the best practices, it is recognized that the data and the expertise to do so is not always available to practitioners from smaller jurisdictions. This should not preclude action/evaluation. It is very likely that research and evaluations will continue to be conducted using methods that are not considered "best practice". This is acceptable, as long as the limitations of the evaluation are recognized, documented, and considered by the traffic operations professional when applying the results in practice.

One of the most common sources of error found in road safety studies is regression-to-the-mean (RTTM). It is prevalent in road safety studies and bears mention here to assist practitioners in designing and conducting road safety research. RTTM potential occurs when sites that are selected for treatment are done so because of an abnormally high collision frequency or rate. This is usually the case, and why RTTM is a frequent issue.

RTTM is briefly explained as follows:

The long-term average crash frequency (all things remaining stable) is the true measure of safety at a location. The annual crash count is a short-term measure that is generally used to approximate the long-term average. However, we know that crash counts are subject to some random variation from year-to-year. Therefore, if sites are selected for treatment because of the abnormally high annual crash count, one has to ask – is the high crash count representative of the long-term average, or is it a random fluctuation?

In some instances the short-term crash count is representative of the long-term average, in other instances the short-term count is randomly high. In the case that the count is randomly high, we would expect that the next set of crash counts would be more representative of the long-term average. That is to say, the next set of counts would be closer to the a long-term average (i.e., lower). Hence, if the abnormally high crash count is used as the "before" data, and the short-term crash count is not reflective of the long-term average, then a safety benefit would be exhibited even if no treatment was applied. This tendency for short-term high crash frequencies to produce lower (more average) crash counts in the subsequent observation periods is known as RTTM.

It is common practice for road safety researchers to use a three to five year crash history to account for random variation in annual crash counts. This is an important step in minimizing RTTM effects, but is insufficient to eliminate them.

Hauer (1997) has proposed the Empirical Bayes (EB) method of dealing with the RTTM bias. In brief, under EB the actual crash count of the location is tempered by the mean crash count for similar locations to produce a better long-term estimate of the safety performance of the individual location. Application of the EB procedure requires a relatively sizeable dataset from a number of similar locations and some statistical expertise. In many instances practitioners may not be equipped to apply the EB methods.

These practitioners have available at least three other options for dealing with RTTM. They are:

- *Random site selection and allocation;*
- *Random allocation of sites with aberrant crash records to the treatment and control groups; and*
- *Application of a correction factor.*

These alternatives methods can greatly simplify the safety analysis. However, there are some ethical and legal considerations associated with the first two approaches.

If a treatment is to be applied to improve safety, ethics and due diligence demand that the treatment be applied at the locations where the treatment would do the most "good". The random selection of sites will result in some sites with "good" crash records being treated, and some sites with "poor" crash records being untreated. If there is a good faith belief that the treatment will yield a safety benefit – then this approach is inappropriate.

The above ethical/legal problem is overcome by the second method of selecting sites with "poor" crash records and then randomly allocating them to treatment and control groups. This eliminates the concern that sites with "good" crash records will be treated, but maintains the concern of sites with "poor" safety records being left untreated. In this instance, the decision to leave some sites untreated is acceptable if:

- *There is a good faith belief that the treatment is ineffective (or no more effective then what is currently in place); or*
- *Limited resources do not permit treating all sites.*

The application of a RTTM correction factor is a method that was developed by Abbess et al (1981), and is simpler than the EB methods. The correction factor method also requires a relatively large dataset of locations that are similar to the treated location. However, the mathematics are more manageable for the practitioner. The correction factor is calculated by Equation C1.

$$R = \frac{(N_t + N)n}{(n_t - n)N} - 1 \qquad [C1]$$

Where:
R = Regression effect (as a decimal)
n = Number of years of crash data for the site
N = Number of crashes at the site in "n" years
$N_t = a^2/(v-a)$
$n_t = a/(v-a)$
a = mean or average crash rate for a group of similar intersections
v = variance of mean crash rate for the group

For example, if a two-way stop-controlled intersection with a crash record of 12 crashes in 3 years were treated with an all-direction stop and the crashes were reduced to 5 crashes in a 3 year period, how much of the safety benefit can be attributed to RTTM? In order to use the correction factor, the analyst must assemble data from a group of similarly two-way stop controlled intersections. Furthermore, it is found that the mean crash frequency for the group is 2 crashes/year, with a variance of 0.2. Therefore:

N = 12
n = 3
N_t = (2*2)/0.2-2) = -2.22
n_t = 2/(0.2-2) = -1.11

$$R = \frac{(-2.22 + 12)3}{(-1.11 - 3)12} - 1 \qquad = 0.29$$

The RTTM effect is estimated to be 29%. Therefore, the CMF is calculated as follows:

$$\text{CMF} = \frac{\text{5 crashes/year "after"}}{\text{12 crashes/year "before" x (1-.029) correction for RTTM}} = 0.59$$

Without the correction for RTTM the CMF would have been overestimated to be 0.42.

Authoring a Scientific Paper

It is not the intent of this section of the report to provide a comprehensive discussion on all of the important elements of report writing. Grammar and the language of the paper are certainly beyond the scope of this document. What is intended is for the reader to get an understanding of the essential elements of a research paper, and some suggestions for organizing the content.

A scientific paper, to be of value to the road safety community, either researcher or practitioner, should provide enough information for readers to assess observations, repeat the studies (if desired), and evaluate the intellectual processes.

The essential elements of a quality scientific paper are undisputed:

- *Introduction: What is the problem or issue being addressed?*
- *Materials and Methods: How was the problem studied?*
- *Results: What was found?*
- *Discussion: What do the findings mean?*

The introduction generally includes a review of the literature on the subject matter. Which, as discussed in Appendix A is an essential element of determining causation. This element of papers reporting on road safety is usually present.

In many instances it is the "Materials and Methods" section that is incomplete. In particular the method of site selection, and the limitations associated with a non-random selection of sites (which is often the case) are missing elements of road safety documentation. It has been clearly established that studying treatments at sites that were selected because of an abnormally high crash frequency will overestimate treatment effectiveness because of regression to the mean effects. Appraisal and application of research results that fail to report on methods and materials in a meaningful way severely hampers the road safety effort of practitioners. In a worst-case scenario, erroneous results that cannot be deciphered because of a lack of information on research methods may be used to make decisions respecting road safety.

Recognizing that many practitioners will seldom be able to use the most current and rigourous methods, an important part of the documentation is a discussion on the limitations of the research methods used.

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Appendix D: How to Use Safety Performance Functions

APPENDIX D – HOW TO USE SAFETY PERFORMANCE FUNCTIONS

The current direction in safety research and evaluation is to make use of crash prediction equations, also known as safety performance functions (SPFs), to estimate the long-term crash frequency of a facility. SPFs may take many forms, the most basic uses traffic volumes as an independent variable, and crash frequency as the dependent variable. Equations D1 and D2 below, are examples of basic SPFs for road segments and intersections, respectively.

$$N = a\ ADT^{b} \qquad [D1]$$

$$N = a\ ADT_m^{\ b}\ ADT_s^{\ c} \qquad [D2]$$

where: N = Crashes/year/km (Equation D1); crashes/year (Equation D2)
ADT = Average daily traffic
ADT_m = Average daily traffic for the main road
ADT_s = Average daily traffic for the side road
a, b, c = constants derived from regression

The SPFs are developed through an examination of crash and volume records for a category of roads or intersections. For instance, three-leg, all-way stop controlled intersections in an urban setting may be a category of intersection for which an SPF is developed. Using appropriate regression techniques, data that is available from all intersections in this category can be used to determine the expected crash frequency for this group of facilities.

Combining SPF results with Crash Records

If the crash record of a site or facility is unavailable, then the output from the SPF is the best estimate of the long-term crash count. For instance, if a new signalized intersection is proposed where one does not exist now, then the SPF for signalized intersections and traffic volume projections can be used to estimate the crash count. However, if the crash record is available, then it must be considered in determining the expected long-term crash count of the facility.

The expected frequency as determined by the SPF is an estimate of the safety of the facility, but in order to provide a better estimate of safety for an existing site, the results of the SPF calculation must be tempered with the actual crash record of the site. Consider the following, the SPF for two-lane rural, arterial roads indicates that the expected crash frequency for Arterial "X" is 1.2 crashes/km/yr; the actual crash record of Arterial "X" over the past three years shows 2.6 crashes/km/yr. Which is the correct estimate of the long-term safety? Neither – in order to provide the best estimate of safety the two pieces of information have to be combined.

The SPF produces the expected average crash frequency for facilities of the type specified; the crash record of the specific facility is an indication of the safety for the location. By combining these two pieces of information, we arrive at the best estimate of the safety of the facility. In order to join the two numbers, consideration must be given to the:

- Reliability of the SPF (how well does it predict crash frequency?); and
- Number of years of crash data available for the site.

Using both of these factors one can determine the weight to placed on the actual crash record and the weight to be placed on the predicted or expected crash count. In the regression calibration process the mean and the variance of the regression estimate can be used to determine the overdispersion parameter “k”. Further explanation of “k” and statistical modelling surrounding it are left to others [Hauer, 1997]. Nonetheless, “k” is a measure of the reliability of the SPF, and by knowing it and the crash record of the site we can estimate the safety by combining the results of the SPF and the crash record as follows:

$$EC = w\ OP + (1-w)\ N/n \tag{D3}$$

$$w = k / \{ k + (n\ OP)\} \tag{D4}$$

where:
EC = Expected number of crashes/year
OP = output from SPF
k = statistical measure of overdispersion associated with the SPF
N = Actual number of crashes
n = number of years of crash data

As the number of years of crash data from the site under analysis increases, the weight placed on the measured crash count from motor vehicle crash reports also increases.

Example

Here is an example of how to use this methodology. An unsignalized intersection has the following characteristics:

$$SPF = 0.0044\ ADT_m^{0.64}\ ADT_s^{0.17} \tag{D5}$$

where: $k = 0.766$
$ADT_m = 8,000$
$ADT_s = 4,000$
Crash record = 12 crashes in the last 5 years (2.4 crashes/year)

If it has been proposed that the intersection be signalized, the safety impacts of signalization are determined as follows:

Unsignalized: $SPF = 0.0044\ (8000)^{0.64}\ (4000)^{0.17} = 5.7$ crashes/year
$w = 0.766/(0.766+(5 \times 5.7) = 0.026$
Crashes/year = $(0.026 * 5.7) + (1-.026)*12/5 = 2.5$ crashes/year

If the SPF for a signalized intersection is:

$$SPF = 0.044\ ADT_m^{0.34}\ ADT_s^{0.16} \qquad [D6]$$

where: $k = 0.812$

Then the expected crash frequency under signalization is:

$$SPF = 0.044\ (8000)^{0.34}\ (4000)^{0.16} = 3.5 \text{ crashes/year}$$

As there is no crash record associated with this intersection being signalized, the output from the SPF is the best estimate of the crash record for this intersection.

As a result of the above analysis, it can be expected that signalizing this particular intersection would result in a 40% increase in total crashes (2.5 crashes/year while unsignalized, and 3.5 crashes/year expected under signal control).

Integrating Crash Severity

In the above example, the increase in crash frequency may be associated with a change in the type of crashes, and perhaps the crash severity. If separate SPFs have not been developed for different crash severities, then it is necessary for the analyst to determine the distribution of crash severity through other means.

The typical procedure would be to determine the average distribution of crash severities for the facilities under examination. Again using the above example, it is determined that the average crash severity distributions for unsignalized and signalized intersections are as follows:

Crash Severity	Proportion of all Crashes (%)	
	Unsignalized	Signalized
PDO	73.9	77.0
Injury	25.0	22.7
Fatal	1.1	0.3

Therefore, from the above example the frequency of crashes by severity are:

Unsignalized:
PDO = 2.5*0.739 = 1.85 crashes/year
Injury = 2.5*0.25 = 0.63 crashes/year
Fatal = 2.5*0.011 = 0.03 crashes/year

Signalized:
PDO = 3.5*0.77 = 2.70 crashes/year
Injury = 3.5*0.227 = 0.79 crashes/year
Fatal = 3.5*0.003 = 0.01 crashes/year

The results of this more detailed analysis indicates that the proposed traffic signal, although it increases the total number of crashes, will reduce the incidence of fatal crashes. Accepted societal values of different crash severities can be used to compare desirability of signalizing.

Calibrating SPFs for Different Jurisdictions

SPFs that are developed for one jurisdiction are not necessarily directly applicable to all jurisdictions. Local differences in environment, crash reporting, design standards, and drivers licensing are just a few examples of conditions that will affect the transferability of SPFs. The current procedure recommended for calibrating SPFs for local use is described by Harwood et al (2000).

Essentially, a calibration factor should be developed for each SPF, that is a multiplier to be inserted into the SPF.

$$N = C_f\,[a\ ADT_{main}^{\ b}\ ADT_{side}^{\ c}] \qquad \text{[D7]}$$

where:
N = Number of crashes
C_f = Calibration factor for intersection type
ADT_{main}, ADT_{side} = Average daily traffic
a, b, c = constants developed by regression

The most basic form of calibration factor for intersections is determined by following the procedure:

- Identify a random sample of intersections that correspond to the SPF available (ex., urban, three-leg, unsignalized intersections). Larger sample sizes will produce more reliable calibration factors, but at a greater cost.

- Using the ADTs for the intersecting roads and the borrowed SPF, calculate the total number of crashes ($N_{expected}$) expected at all of the intersections in the sample.

- Calculate the sum of all crashes (N_{actual}) that occurred at all of the sample intersections.

- Calculate the calibration factor (C_f) by dividing $N_{expected}$ by N_{actual}.

It is important that the sample be representative of the different geometric and traffic conditions that occur at the selected intersection type. Stratification of the sample may assist in this regard. Harwood et al (2000) suggest that the sample sizes contain a minimum of 100 intersections for stop-controlled intersections, and 25 intersections for signal-controlled intersections.

Calibration using more advanced methods, or additional data is possible, but is not explained herein. Similarly the calibration of SPFs for road segments is not explained herein. Analysts who want to learn more about calibration and transferability of SPFs are referred to Harwood et al (2000), and should monitor the research being conducted by the United States Federal Highway Administration for the Interactive Highway Safety Design Module (IHSDM).