2016

Tool For Calculating User Delay Cost Associated With Urban Arterial Construction Zone

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Ghaffari Dolama, M. (2016). Tool For Calculating User Delay Cost Associated With Urban Arterial Construction Zone (Unpublished master's thesis). University of Calgary, Calgary, AB.
doi:10.11575/PRISM/28464
http://hdl.handle.net/11023/3523
master thesis

UNIVERSITY OF CALGARY

Tool For Calculating User Delay Cost Associated With Urban Arterial Construction Zone

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE

DEGREE OF MASTER OF SCIENCE

GRADUATE PROGRAM IN CIVIL ENGINEERING

CALGARY, ALBERTA

December, 2016

Abstract

Roadway construction work zone imposes travel delay on the road users. The monetary cost of the delay is called user delay cost (UDC). Limited work has been done on quantifying UDC in Canada and their focus were rural highways. If there is a realistic estimate of UDC, it could lead to less schedule overruns and more cost-effective work zone layouts. Considering interest of roadway agencies in quantifying UDC associated with urban arterial work zones, this research developed a probabilistic tool for monetizing UDC in urban setting using traffic microscopic simulation and Monte-Carlo simulation. Based on this tool, one hour of morning peak construction work on NB Crowchild Bridge created 169.2 hr of vehicles delay with average and 95 percentile UDC equal to $2,199 and $5,653, respectively. The application of the tool for selecting optimum work zone layout was demonstrated using the data from the rehabilitation of Bow Bridge in Calgary.

Keywords: user delay cost, user cost, contract incentives, accelerated construction methods, work zone, road occupancy cost, optimizing work zone layout

Acknowledgements

I would like to express my gratitude to the following individuals who unconditionally supported me during the most challenging period of my life:

- My parents who did their best to motivate me at the hard days of poor health.
- My amazing sister" Dr. Shabnam Ghaffari Dolama" whose professional advice and emotional support was one of the driving forces.
- My dedicated doctor" Dr. Paul MacMullan" and his medical team particularly "Alexandra Charlton" for saving my life.
- My exceptional supervisor "Dr. Lynne Cowe Falls" who had profound impact on my life. I found her a knowledgeable professional and an honorable personality. She is a person that I could trust to her impartial advice on different life challenges. I am proud to have her as my supervisor.
- Lovely Kerstin and Bruce Littlejohn who treated me as their daughter and created sense of family for me. I am glad for having them as friend.
- Dr. George Jergeas whom I benefited from both his professional knowledge and life experience, and honoring to be in the thesis committee.
- Dr. Lina Kattan for positively impacting this research by generously allowing to audit her courses, access to Transportation Lab and microscopic simulation software, and honoring to be in the thesis committee.
- Dr. Laleh Behjat, for being one of the most considerate professors that I have ever had and honoring to be in the thesis committee. Working with her as both her student and her Teaching Assistant is one of my pleasant experience at the University of Calgary
- Dr. Wirasinghe, Dr. De Barros, Dr. Grozic, and Dr. Wong, for their professional advice.
- Students at the Transportation Lab for generously sharing their professional knowledge and creating friendly environment in the Lab; particularly, Mr. Suman Mishra for his input at early stage of working with Synchro software and Ms. Mahsa Ghaffari for her kind manner.
- Mr. Carlo Andolfo "Project Management Graduate" for being an unconditional friend.

- Mr. Eric Torkia and Technology Partnerz Ltd. For their generous three months free access to Crystal Ball software which caused the research to finish as had been planned
- Dr. Anna-Lisa Ciccocioppo and Dr. Jan Crook for their life lessons, I am glad we have such amazing professionals in our university.
- Ms. Jadwiga Kroman " Manager of Bridges and Structures Transportation Infrastructure of the City of Calgary" for her motivating interest in the research, follow up with the progress, valuable industrial input, and facilitating access to the input data.
- Mr. Ekke Kok" Manager of Transportation Data Transportation Planning Department of the City of Calgary "and staff members for providing free and timely access to the traffic data.
- Mr. Stephen Nicholls "Sr. Traffic Signals Engineer of the City of Calgary" and staff members for providing signal timing data.
- Ms. Jillian Kohut, "MA Economy from the City of Calgary" and Mr. Majid Asefi "Manager Fleet Operations Fleet Services Corporate Services of The City of Calgary" for their inputs and facilitating access to some of the required input data.
- Mr. Michael T. Trueblood "Director of Traffic & Transportation from TERRA Associates, Inc., USA, for his input on SimTraffic software calibration.
- Any individual who had influence in the success of this research.

This work was funded through a **Queen Elizabeth II** scholarship and this support is gratefully acknowledged.

Dedication

This work is dedicated to any individual who makes positive change even with a kind smile.

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List of Symbols, Abbreviations and Nomenclature

Symbol	Definition
μi	Mean UDC of Model i
AAC	Added accident cost
AADT	Average annual daily traffic
AADTT	Annual average daily truck traffic
AASHTO	The American Association of State Highway and Transportation Officials
AB	Alberta
ABC	Accelerated bridge construction
AC	Accelerated construction
ADB	Asian Development Bank
AGC	Reduced or increased agency supervision cost
ANOVA	Analysis of variance
Ave	Avenue
AVOC	Added vehicle operation cost
B	Benefits associated with the improvement scenario
BC	British Colombia
BLS	Bureau of Labor Statistics
BPC	Monetary value of business travel time with passenger cars
BPCI	Intercity Business Travel Time With Passenger Car
BPCL	Local Business Travel Time With Passenger Car
C	Costs associated with the improvement scenario
C BPC	% of business travels in passenger cars
C BPCI	% of compensation cost per hour considered as the value of personal intercity travel time
C BPCL	% of total compensation cost per hour considered as the value of personal local travel time
C min	Minimum cycle length
C ppc	% of personal travels in passenger cars
C PPCI	% of median household income considered as the value of personal intercity travel time
C PPCL	% of median household income considered as the value of personal local travel time
CA	Contractor acceleration cost
CAD	Canadian dollar
CAD/ Kg	Canadian Dollar per Kilogram
CANSIM	Canadian Socio-Economic Information Management System (Statistics Canada)
CO3	Construction Congestion Cost System
CPI	Consumer price index
CPI -U	Consumer price index for all urban consumers in US
D	Disincentives
Delay pc	Total delay of passenger cars due to the work zone
Df	Multiplier to calculate I/D based on UC
DOT	Department of Transportation

Symbol Definition

DP	Cost of vehicle depreciation including all vehicles
Dr	Drive
EB	East bound
ESAL	Equivalent Single Axel Loading
EUC	Estimated user cost
F	Cost of freight inventory delay carried by trucks
FFS	Free flow speed
FHWA	Federal Highway Administration
FICEM	the Federation of Intra-American Cement Manufacturers
FinnRA	Finnish Road Administration
G EB	EB Green time
G WB	WB Green time
GDP	Gross domestic products
HDM-4	Highway Development and Management tools
HERS	Highway Economic Requirements Systems
HERS-ST	Highway Economic Requirements System–State Version
hr/voc	Hour per vehicle occupancy rate
I	Incentives
i	Indicator
I/D	Incentives and disincentives
ITE	Institute of Transportation Engineers
Kg	Kilogram
Km/h	Kilometer per hour
L	Left
LCC	Life cycle cost
LCCB	Life cycle cost benefit
m	Metre
MDOT	Michigan Department of Transportation
MTO	Ministry of Transportation, Ontario
MUC	Maximum user cost
NB	North bound
NOC	National occupational classification
NW	North west
ODA	The United Kingdom Overseas Development Administration
OST	Office of the Secretary of Transportation
p	p-value
PHF	Peak hour factor
PPC	Monetary value of personal travel time with passenger cars
PPCI	Personal intercity travel time with passenger car
PPCL	Personal local travel time with passenger car
QUEWZ	Queue and user cost evaluation of work zone
R	Right
RMF	Percentage of risk the agency is willing to share or recover from the contractor
ROIM	Discount factor
Symbol	Definition

s	Second
S.D.	Standard deviation
s/veh/voc	Second per vehicle per vehicle occupancy rate
Symbol	Definition
S15	15 percentile speed
S85	85 percentile speed
SB	South bound
sec	Second
SK	Saskatchewan
SNRA	Swedish National Road Administration
St	Street
SW	South west
T	Through
t	t-statistics
TCH	Trans-Canada Highway
TCV	Total critical lanes volume
TD	Monetary value of truck drivers travel time with truck
Tr	Trail
TRB	Transportation Research Board
UC	User cost
UDC	User delay cost
US	United States of America
USA	United States of America
USD	US dollar
USD/ KG	US Dollar per Kilogram
USDOT	The United States Department of Transportation
v/c	Volume per saturation flow rate
veh	Vehicle
voc	Vehicle occupancy rate
Voc BPCI	Vehicle occupancy of passenger cars in intercity business travel
Voc BPCL	Vehicle occupancy of passenger cars in local business travel
Voc PPCI	Vehicle occupancy of passenger cars in intercity personal travel
Voc PPCL	Vehicle occupancy of passenger cars in local personal travel
VOT	Value of travel time
vphpl	Vehicle per hour per lane
vs	Versus
WB	West bound
WIM	Weight In Motion
WZ	Work zone

Chapter One: Introduction

1.1. Research Background

The total cost of roadway construction includes direct and indirect costs (Chan et. al, 2008 and Gilchrist & Allouche, 2005). Direct cost comprises of engineering, construction, construction supervision, and administration costs, while indirect cost is the invisible cost paid by the road users, known as user cost. User cost has three main components: user delay cost (UDC), added vehicle operation cost (VOC), and added accident cost (AAC) as well as components that are harder to monetize, such as negative impacts on local communities and businesses, noise, and emissions (Chan et. al, 2008, Gilchrist & Allouche, 2005, Chien & Schonfeld, 2001, Daniels et. al , 1999, Sadasivam & Mallela, 2015). When a construction work zone is established in a segment of a roadway, travel time in that segment usually increases. It is due to either reduced posted speed or congestion. This extra travel time is commonly known as user delay. The monetary value of this delay time is a cost that is paid by road users and is referred to as user delay cost (UDC). Additionally, due to the increased travel time at the work zone and the nature of driving in this zone which is accompanied by extra decelerations & accelerations, vehicle operation cost (VOC) increases. This added vehicle operation cost (AVOC) is another indirect cost that is paid by the road users. Usually the work zone segment of a roadway is more susceptible to accidents compared to its normal operation. This added accident cost (AAC) is another component of the indirect cost of construction activity in a roadway.

Work zone instigated user costs in densely populated areas could outweigh the direct project costs (Sadasivam & Mallela, 2015) and therefore need to be considered in project alternative selection. This is the case in bridge rehabilitation projects in major urban arterials where there is high traffic demand with limited or no alternative river crossing as well as limited work zone space and layout options. The high value of user cost in such areas encourages governing agencies to attempt construction acceleration approaches among them; introducing contractual incentives and disincentives (I/D) or accelerated bridge construction (ABC) methods (Sadasivam & Mallela, 2015, Sadasivam & Mallela, 2016, Jia et. al, 2016). However, justifying construction acceleration approaches needs quantifying user cost. Different aspects of user cost have been researched so far, among them: Optimum work zone length to minimize UDC; Sensitivity of UDC to work zone layout, length and average annual daily traffic (AADT); Balancing economic impacts of work zone between stakeholders; Incorporating UDC in project life cycle analysis and creating UDC versus AADT look up tables (Chien & Schonfeld, 2001, Raymond et. al, 2000, Huen et. al, 2005, Huen et. al, 2006, Al Assar et. al, 2000). However, these researches focused on freeways and rural highways; additionally, user delay was calculated based on the recommendations of transportation codes. In contrast, this study focuses on developing tool for quantifying user delay cost and its associated risk in urban setting as well as incorporating microscopic traffic simulation to calculate user delay incurred during rehabilitation of bridges in major urban arterials. Considering the time frame of this research work and extent of user cost topic, this research will focus on quantifying user delay cost (UDC) aspect of user cost.

1.2. Research Objective

There has been extensive research on user cost, but those studies mainly focused on rural highways (Mallela & Sadasivam, 2011, Salem & Genaidy, 2008, Hawk, 2003, Walls & Smith, 1998). This study focuses on quantifying UDC in an urban setting with a focus on rehabilitation of urban bridges in major arterials. A decision support tool is developed to calculate hourly rate of road occupancy which is verified using the recorded travel data during the rehabilitation of

Crowchild Tr Bridge in Calgary, Canada. Additionally, application of the tool in selecting optimum work zone layout from UDC point of view is demonstrated using travel data during the rehabilitation of Trans-Canada Highway (TCH) Bridge on the Bow River (Bow Bridge) in Calgary, Canada.

1.3. Research Contributions

At the moment, there is no tool for quantifying road occupancy cost in urban setting in Calgary. The result of this research will provide the City of Calgary with a decision support tool to quantify road occupancy cost associated with its bridge rehabilitation activities and thereby the City could incentivize its contractors to finish projects in a timely manner and consequently, reduce congestion and economic costs to the City. Additionally, the developed tool could assist the City with selecting the most optimum work zone configuration and justifying accelerated bridge construction methods versus conventional construction methods.

1.4. Research Methodology

Microscopic simulation (microsimulation) models of traffic in study cordons of two cases _ Crowchild Tr Bridge and Bow Bridge_ under both normal operation and work zone conditions are built using SimTraffic software. These models are calibrated using recorded field travel time measurements of these bridges, then the added travel time of the traffic due to the construction work zone is calculated. Using the developed tool, probability distribution of hourly rate of road occupancy cost for each bridge is calculated. Additionally, in the case of Bow Bridge using the aforementioned process and statistical tools, the optimum work zone layout among three examined work zone layouts is analyzed. The research methodology has been elaborated in the following sections and a pictorial summary of that has been illustrated in Figure 1.1.

1.4.1 Traffic simulation model.

1.4.1.1 Crowchild Tr.

Using SimTraffic microsimulation software, the traffic flow on the Crowchild Tr Bridge and nearby routes with in a traffic flow cordon is simulated under normal operation of the bridge. The model is calibrated with the field travel time measurements in 2002, then with a similar process a second model is built. In this scenario, the bridge is modeled under work zone condition when access from both 10 Ave SW and Bow Tr WB onramps to Crowchild Bridge were closed and only one lane (the right lane) of the bridge was available for traffic (Wilson & Cowe Falls, 2003). This model is calibrated using the field travel time measurements which were recorded during the bridge rehabilitation period in 2002.

For every road user, the difference on their travel time under normal and work zone condition is the delay that is incurred due to the work zone establishment. In this research delay imposed on the following three categories of road users was studied (Walls & Smith, 1998):

Category 1 (Travelled through the work zone): This group travel through the work zone and pay all the associated costs as they have no other option. They have little effect on the other routes in the cordon and their delay time is the difference between their travel time through the bridge under normal condition and work zone condition. Depending on the capacity of the work zone compared to the traffic demand, their delay could be either the sole effect of reduced posted speed in the work zone or a combined effect of the reduced posted speed and queueing ahead of the work zone. This has been detailed in the following two paragraphs.

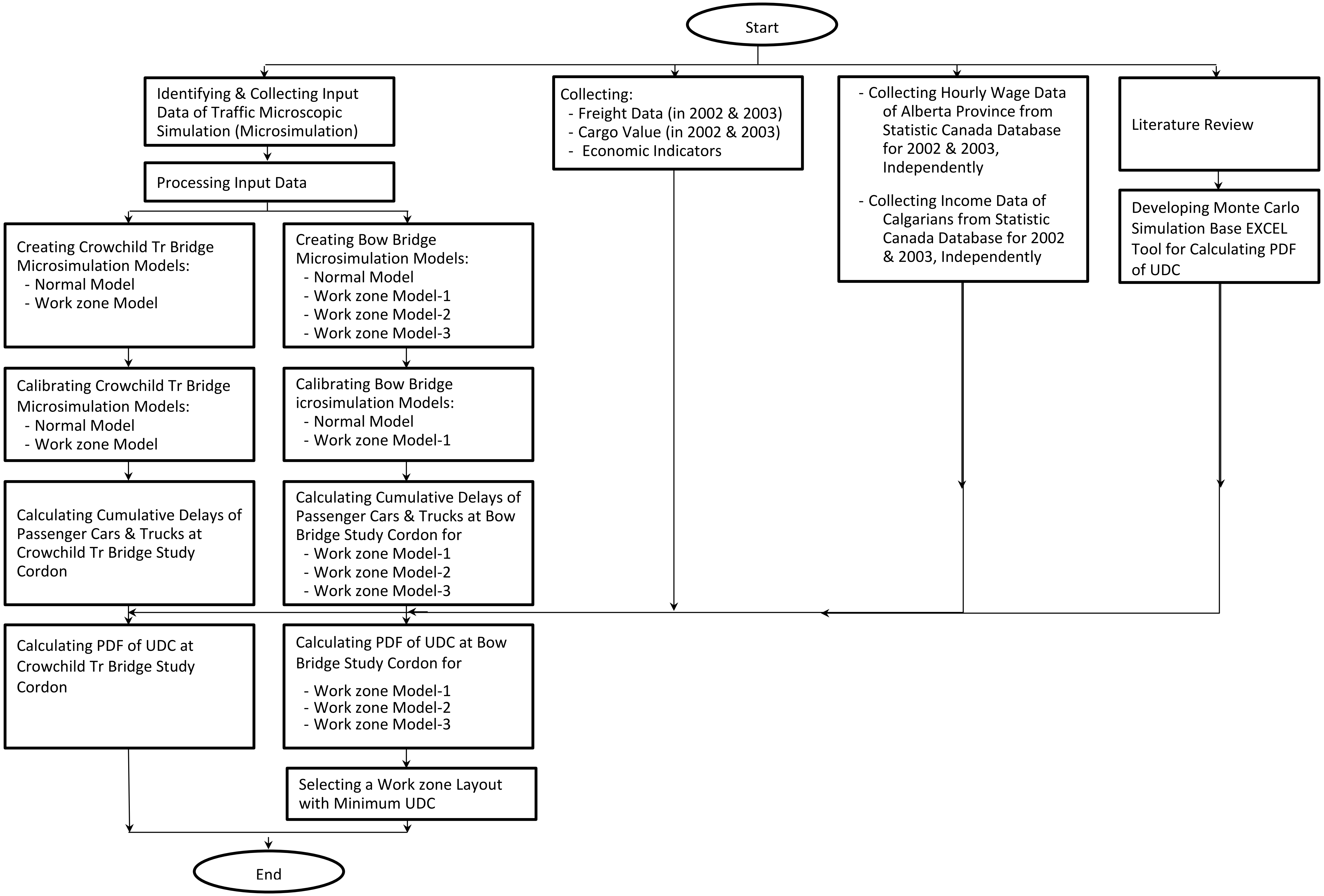

Figure 1.1 Research Methodology

When a work zone is established in a segment of a roadway, usually the posted speed at the work zone area is reduced. This results in the reduction in the roadway capacity. Additionally, depending on the work zone layout, an extra reduction in the roadway capacity could occur as well. If this reduced capacity is greater than the traffic demand at the work zone, then the traffic will travel under free flow condition but with slower speed and consequently longer travel time compared to the normal condition. In this case the excess travel time compared to the normal condition is the user delay time. This delay time is composed of time required for deceleration to the work zone speed and, extra time required to traverse the work zone due to the reduced posted speed at the work zone area, and time required for acceleration from the work zone speed to the approach speed (Mallela & Sadasivam, 2011).

In contrast, if the reduced capacity of the work zone is less than the traffic demand, then queue will form upstream of the work zone (force flow condition). Again, the excess travel time compared to the normal condition is the user delay time but the components of the travel time under force flow condition are different than the free flow condition. These components are deceleration time from approach speed to full stop, waiting time in queue, acceleration times (from full stop to work zone speed & from the work zone speed to approach speed), and crossing times (crossing the queue & crossing the work zone) (Mallela & Sadasivam, 2011).

Category 2 (Detoured): These road users fall into two subcategories based on opting or being forced to detour. We will call them: Willingly detoured and Forced to detour, respectively. Willingly detoured avoid the work zone by detouring to the other routes in the cordon. They reduce the impact of work zone on their travel time by invading routes of other vehicles in the cordon (Walls & Smith, 1998). In contrast, Forced to detour have to detour because their access to their desired route are blocked due to the work zone restrictions. In the case of Crowchild Tr Bridge rehabilitation, traffic which had to detour due to the closure of both 10 Ave SW and Bow Tr WB onramps belong to this subcategory.

Category 3 (Hosts): These are the road users on the nearby routes in the cordon who are forced to share their routes with detoured group. Detoured group's presence in host group's routes increases traffic demand on these routes. If the new demand is higher than the capacity of these routes, the hosts will encounter forced delay which is the difference between the hosts' travel time under normal condition and their travel time under the work zone condition.

Since the scope of this research is developing a tool for calculating user delay cost rather than studying variation on route selection due to a construction work zone set up, this study will only focus on the effect of work zone on Forced to detour road users. For this group the incurred delay due to work zone is the difference on their travel time when they have to detour and the time they required to cross the bridge under the normal operation condition, equation 1.1.

$$Detour\ Delay = Detour\ Condition\ Travel\ Time - Normal\ Condition\ Travel\ Time \qquad 1.1$$

Additionally, considering the scope of this study, any potential change on the number of host road users due to the work zone set up will be disregarded.

1.4.1.2 Bow Bridge on TCH.

As mentioned in section 1.2, demonstrating application of the developed tool in selecting optimum work zone layout is another objective of this thesis. Since different work zone configurations could affect road users' delay differently, using the method introduced in section 1.4.1.1, effect of three different work zone configurations on user delay due to the rehabilitation of Bow Bridge on TCH in 2003 is studied. The following are the examined work zone configurations:

- Model 1: Closing one lane in each direction (work zone in 2003)
- Model 2: Complete detour of EB traffic via nearby major street (Bowness neighborhood)
- Model 3: Closing one bound and diverting traffic to the opposing bound (traffic light control)

In this case, travel times of road users in the Bow Bridge study cordon under the above three work zone configurations are simulated and the difference between the average simulated travel time under each work zone configuration and the average simulated travel time under normal operation of Bow Bridge is calculated. These values are excess travel time imposed on road users (Travelled through the work zone, Detoured, Hosts) due to the different work zone configurations. With capitalizing these excess travel times, the associated UDC under each work zone configuration could be determined.

1.4.2 Capitalizing user delay cost.

According to FHWA, UDC (user delay cost) is composed of three main components including: travel delay cost incurs to the road users including passenger cars in personal travel, passenger cars in business travel, and trucks, cost of freight inventory (carried by trucks) delay, and vehicle depreciation cost (Mallela & Sadasivam, 2011), See equation 1.2.

$$User\ Delay\ Cost\ (\$) = Travel\ delay\ cost + Freight\ inventory\ Delay\ Cost\ + Depreciation\ cost \quad 1.2$$

FHWA recommends a deterministic approach for calculating these components (Mallela & Sadasivam, 2011), which has been detailed in Section 5.2 of this thesis. However, as it will be elaborated in Chapter 5, the components of UDC are probabilistic in nature, thus in this research

UDC and its components will be calculated using the probabilistic tool developed in this study, the subject of Chapter 5. The following is an explanation of the UDC components and their quantifying method.

- Travel delay cost: After calculating the travel delay time for each category of the road users, the distribution of their travel delay cost due to the work zone set up will be calculated, equation 1.3.

$$Travel\ Delay\ Cost\ (\$) = \sum_i (\ Delay\ Time_i \times\ Travel\ Time\ Value_i \times\ Vehicle\ Occupancy_i) \qquad 1.3$$

Where:

- *i: {Passenger car in personal travel, Passenger car in business travel, Truck}*
- *Delay Time $_i$ (hr): Total travel delay incurred to vehicle type "i" due to the work zone (Section 1.4.1.1)*
- *Travel time value $_i$ ($/hr): Probabilistic distribution of the dollar value of one hour of travel time of vehicle type "i"*
- *Vehicle Occupancy $_i$ (person/veh): Average number of people in a vehicle type "i"*

In the available literature, travel time value is a function of vehicle classification (Mallela & Sadavisam, 2011, Walls & Smith, 1998). FHWA classifies vehicles to three categories of passenger cars, single unit trucks, and combination trucks. The value of travel time for each of these categories is different and is increasing in that order (Walls & Smith, 1998). In this study since the available vehicle classification is cars and trucks (traffic count data of the City of Calgary) and does not differentiate between single unit trucks and semi-trucks, only one category of travel time value for trucks will be used in capitalizing the user delay cost. Travel time value of people in passenger car in personal travel is estimated as percentage of their hourly income (Mallela & Sadavisam, 2011), the Calgary wide income distribution, extracted from Statistics Canada website is the base for calculating travel delay cost of road users in passenger car and personal travel. Travel time value of road users in passenger car and business travel is estimated as percentage of employer cost for employee compensation (Mallela & Sadavisam, 2011). This cost will be estimated from the hourly wages and benefits distribution of different occupations (excluding NOC 7 category of jobs) aggregated on provincial level which are extracted from Statistics Canada website. Travel time value of trucks is estimated equal to truck drivers' wage and benefits (Mallela & Sadavisam, 2011). This cost will be estimated from the hourly wages and benefits distribution of NOC 7 category of occupations aggregated on provincial level which are extracted from Statistics Canada website. Vehicle occupancy refers to the average number of occupants in a vehicle including the driver. The vehicle occupancy data from the City of Calgary Mobility report (The City of Calgary, 2012) will be used in this study.

- Freight inventory delay cost: This cost is the "hourly interest value of the monetary equivalent of merchandize carried by trucks through the work zone" multiplied by the "delay time" (AASHTO, 2010). In other words, cost of freight inventory delay carried by trucks is estimated equal to the market return on the cash equivalent of the delayed cargo during the delay period (Mallela & Sadavisam, 2011).

- Depreciation cost: Vehicle total depreciation is composed of mileage depreciation (usage component) and time depreciation (aging). The time depreciation of vehicles due to the work zone delay is part of the depreciation cost that is included in calculating UDC (Mallela & Sadavisam, 2011). Calculating time dependent depreciation requires calculating added vehicle operation cost (AVOC) which is out of the scope of this research, so depreciation will be excluded from UDC calculations.

Using the above input data and applying Monte-Carlo simulation technique to equation 1.2, the probability distribution of UDC will be calculated (Chapter 5). In order to assess the sensitivity of UDC to work zone layout for the Bow Bridge study cordon, the probabilistic distribution of UDC associated with each work zone layout is calculated. Then using ANOVA and t-test, the difference on the UDC in respect to the work zone layout is calculated and the least expensive option from UDC point of view is determined.

1.5. Scope Limitations

As mentioned in Section 1.3, the objective of this research is to develop a methodology for calculating user delay cost associated with bridge rehabilitation on major urban arterials. Traffic microscopic simulation (Chapter 3 and Chapter 4) is not the focus of this study rather it is used to demonstrate its application for the purpose of the developed methodology. The same applies to the Signal timing optimization wherever it was needed (Chapter 4). As such the signal optimization at this study is limited to individual signal optimization module of Synchro software rather than network signal optimization. In this case the software calculates the new cycle length and splits and maximum green times based on the new critical volumes of each approach, calculated using detour volumes. The phasing sequence, yellow and all red times, and all other features of the signals have been preserved. Obviously, in real world condition this optimization needs to be done with considering required details and its counter effects on the nearby streets.

Since calculation of time depreciation component of UDC requires calculating AVOC (added vehicle operation cost) which is out of the scope of this study, depreciation will be excluded from UDC calculations. Increased travel reliability cost is treated by some references as part of the work zone created user cost (Jia et al., 2016) since this cost category is not in the scope of this research, it would not be covered.

During the construction period some drivers might decide instead of using personal car to use public transit. This shift in travel mode could be subject of an independent study as a result this study would not addressed it. The same applies to Braess' paradox. It has been noticed closing some links in a congested networks could improve the overall performance of the network. This phenomena is known as Braess' paradox which has been first explained by Dietrich Braess, a mathematician at Ruhr University, Germany (Wikipedia, n.d.). Potential for such a paradox would not be addressed in this research. As it is mentioned traffic simulation is used to demonstrate its application for the purpose of the developed tool; as a result, in this study static traffic routing (Appendix A) will be applied.

1.6. Thesis Outline

This thesis is organized into six chapters: Chapter one is an introduction to the research work; Chapter 2 is a review of available literature on user cost (UC) with focus on user delay cost (UDC); Chapter 3 introduces the application of traffic microscopic simulation for calculating user delay due to one hour of construction work on a bridge during morning rush hour. Data of the rehabilitation of Crowchild Tr Bridge over the Bow River in 2002 is used as the case study; Chapter 4 presents a sensitivity analysis of user delay to work zone configuration. Using the data of the rehabilitation of Bow Bridge in TCH in 2003, user delay associated with three work zone configurations is calculated and work zone with the least delay is figured out. Chapter 5 demonstrates application of Monte- Carlo simulation technique for capitalizing UDC associated with bridge rehabilitation work zones. In this chapter the monetary value of UDC associated with case studies in Chapter 3 and Chapter 4 is presented; Chapter 6 is the concluding chapter which is comprised of the main findings and limitations of this research as well as recommendations for future research work.

Chapter Two: Literature Review

2.1 Introduction

As discussed in Chapter 1, during the past 25 years different aspects of user cost (UC) have been researched, among them:

- Monetizing work zone instigated user cost (Mallela & Sadavisam, 2011, AASHTO, 2010)
- Balancing economic impacts of work zone between stakeholders (Huen et al., 2006)
- Incorporating UDC in project life cycle cost benefit analysis and creating UDC versus AADT look up tables (Huen et. al, 2005, Al Assar et al., 2000)
- Optimum work zone length to minimize user delay cost (UDC) (Chien & Zhao, 2016, Chien & Schonfeld, 2001)
- Sensitivity of UDC to work zone layout and AADT (average annual daily traffic) (Raymond et al., 2000)

The most comprehensive reports on user cost and its components are: American Association of State Highway and Transportation Officials (AASHTO)'s book titled "User and Non-User Benefit Analysis for Highways" which is known as AASHTO's "red book" (AASHTO, 2010), and FHWA report No. FHWA-HOP-12-005 (Mallela & Sadavisam, 2011). The current research trend has focused on application of UC in justifying accelerated construction methods (AC) versus conventional methods with special focus on bridge construction and application of UC in defining contractual incentives and disincentives (I/D). I/D are contractual measures to encourage the contractor to finish construction works ahead of time (I) or on time (D).

However, the research works on UC and its applications are mostly focused on freeways and rural highways: there is a gap of knowledge when it comes to major urban arterial roads. Additionally, UC has been calculated using code based deterministic methods while it is a probabilistic parameter. These gaps will be addressed in this research by incorporating microscopic traffic simulation and Monte- Carlo simulation. Traffic microscopic simulation will address the effect of nearby controls on work zone traffic and Monte- Carlo simulation technique will cover the inherent uncertainty of the components of UC.

In this chapter, after reviewing available research works in Canada (Section 2.2), the most prominent studies in the USA and other parts of the world will be presented (Section 2.3). The chapter will conclude with a brief summary section (Section 2.4).

2.2 Research on User Cost in Canadian Context

Al Assar et al. (2000) provide a series of lookup tables for calculating UDC (composed of slowing delay and queue delay) due to rehabilitation and maintenance of road ways or bridges using OPAC 2000 package. In this study eight traffic control plans are defined and then, for each plan by varying AADT and keeping other factors (length of work zone, percentage of trucks, job duration per day, etc.) constant, user delay cost is calculated. Traffic plans are differentiated by the number of highway lanes, whether it is divided or undivided, and lane closure policy in the work zone. According to the findings of this study: UDC increases exponentially as soon as demand exceeds work zone capacity and queue forms and, threshold AADT value for queue formation depends on the traffic control plan (Al Assar et al., 2000).

UDC associated with traffic staging options during pavement resurfacing is studied by Raymond et al. in 2000. UDC in this study consists of slowing delay and queuing delay with a focus on four lane divided highways. Three staging alternatives are compared and associated UDC and total cost are calculated. These alternatives include traditional method of closing one lane in each direction, the traditional method with full time police presence, and detouring traffic to the opposite lanes of the highway and separating traffic from the opposing traffic with temporary traffic delineating posts. For each traffic staging alternatives, three work zone lengths (2 km, 5 km, and 8 km) and three categories of traffic volumes (low: 500 vphpl, medium: 750 vphpl, and high: 1000 vphpl) are studied. Based on the findings of this study, the traditional method of traffic staging is the most cost effective method as long as traffic demand is below work zone capacity. As soon as the queue forms, UDC associated with this method increases dramatically and results in this alternative not being a viable alternative. In case of queue formation, alternative three (detouring) is the economical option particularly for longer work zones. Additionally, the difference between alternative one and alternative two is the premium cost of police presence from cost wise and added safety due to the police presence (Raymond et al., 2000).

Huen et al. (2005) incorporate UC (user delay cost and added vehicle operation cost) in project life cycle cost analysis using OPAC2000. Eleven functional classes in Ontario, Canada are studied and UC, both during the facility construction and scheduled maintenance over an assumed 50 years life of the project is calculated. The main focus of this study is two lane highways with one lane closed due to the work zone; however, in the case of four lane highways, it is assumed one lane in each direction is closed. OPAC 2000 is used to create baseline deterioration curves of pavement and to calculate life cycle cost. The study found that UC is significant compared to the construction cost and recommended that UC be included in project alternative selection. In addition, facilities with higher AADT experience higher delay and UC, and four lane facilities experience less delay compared to the equivalent two lane facilities because they do not share a common lane through the work zone (Huen et al., 2005).

As a part of research work to improve mobility, safety and balance the economic impacts of work zone between key stakeholders, Huen et al. (2006) developed a prediction model to estimate UDC. This research is a partnership between MTO (Ministry of Transportation, Ontario), University of Waterloo, and University of Toronto. The prediction models are developed for two lane and four lane asphalt surfaced highways with provisions for multi lane highways, using Ontario data and equivalent single axel loading (ESAL). According to the findings of this research, UDC associated with four lane highways is lower than two lane highways due to lane closure strategies used in four lane highways in order to provide less restriction to traffic flow and roads in Northern Ontario have lower UDC than roads in Southern Ontario due to the lower population density in Northern Ontario. Additionally, a method was developed to calculate what percentage of the UDC should be allocated to the road agency. Calculated UDC for 90% ESALs and 110% ESALs and the resultant percentage increase was used as the percentage of UDC to be allocated to the road agency. Higher value of this percentage is translated as high sensitivity of UC to traffic volume and a justification to accelerate construction work (Huen et al. 2006).

2.3 Research on User Cost in International Context

There is considerable research on UC concept in international context particularly in partnership with FHWA (Federal Highway Administration, USA). USA is the leading country in research on UC. The following is a summary of the driving forces for this research:

- Strong interest from US Congress and USDOT in 1980s on maximizing road user's benefits from use of constrained public funds which resulted in considering user cost in highway investments (FHWA, 2000).

- Introduction of Special Experimental Program (SEP) No.14 in 1988 which aimed on introducing innovative contracting practices (cost plus time bidding, lane rental, and design-build contracting) in federal aided projects in order to minimize work zone instigated user impacts by shortening project delivery time without jeopardizing product quality (Mallela & Sadavisam, 2011).
- Work Zone Mobility and Safety Rule (effective since October 2007), this rule requires all state and local highway agencies to develop and implement policies and procedures for evaluating and managing work zone mobility and safety impacts on every individual project is financed in whole or in part with Federal-aid highway funds (Mallela & Sadavisam, 2011).
- Every Day Counts program (2010): The goal is to identify and deploy innovation (Accelerated construction methods, adaptive signal timing, alternative contracting, etc.) in order to shorten project delivery, enhance roadway safety, and protect the environment (Mallela & Sadavisam, 2011).

This interest and support both from congress and USDOT resulted in considerable research and publication on different aspects of UC. The most referenced works being the publications by FHWA, report. No. FHWA-HOP-12-005 (Mallela & Sadavisam, 2011) and AASTHO (AASHTO, 2010). The former includes the most detailed discussion of the components of UC (UDC, added vehicle occupation cost (AVOC), added accident cost (AAC)) and other work zone instigated social costs (emission cost, noise pollution, etc.) and is the most recent and the most comprehensive publication in this regard. However, similar to the other publications this reference applies a deterministic approach to calculate UC. The probabilistic approach developed in this research will be build up on the frame work of this reference. Since there is detailed discussion of this reference in Chapter 5, no further explanation on it will be presented in this chapter. The following is the comprehensive literature review of the outstanding international research on UC. Categorized in to developed tools (2.3.1) and UC applications (2.3.2) including: optimizing work zone layout, justifying accelerated construction (AC) methods, and contractual incentive and disincentive (I/D) calculation.

2.3.1 Tools.

Chien & Zhao provide a comprehensive list of tools applied by different State DOTs (Departments of Transportation) in US ranging from Excel based simple spreadsheets to traffic simulation tools, among them QUEWZ (Queue and User Cost Evaluation of Work Zone) (Chien & Zhao, 2016). QUEWZ was developed in 1984 and estimates the average speed in work zones, UC, UDC, and AVOC (Chien & Zhao, 2016, Chien & Schonfeld, 2001). Based on the assumed lane capacity and traffic volume the tool calculates the effects of different lane-closure strategies and the number of hours available for closures; however, the model does not consider the effect of detoured traffic (Chien & Zhao, 2016, Chien & Schonfeld, 2001).

CO3 (Construction Congestion Cost System) is an integrated set of tools that estimates the effect of alternative traffic maintenance contractual provisions on construction cost and road user cost. CO3 was developed from 1994 to 1997 by Michigan University under a research contract for MDOT (Michigan Department of Transportation) for the purpose of providing MDOT with a practical tool to reduce the effect of construction works on road users. The software input includes estimated distance and travel speed under work zone and normal conditions. Relationship between the work zone delay and percentage of vehicles that might divert to other routes or cancel their trips is also inputted to the software. Additionally, an hourly UC and cost of one mile of cancelled trip are other inputs. As output, CO3 provides detailed estimates of delays and associated UC and total cost (UC plus construction cost) for different lane closure alternatives. So, by comparing total cost of different lane closure alternatives one can choose a preferred construction and traffic maintenance method (Carr, R. I. 2000).

HDM-4 (Highway Development and Management) is a decision support tool for assessing economic and engineering viability of roadway projects. Its development was sponsored by the World Bank, the United Kingdom Overseas Development Administration (ODA), the Asian Development Bank (ADB), the Swedish National Road Administration (SNRA), the Federation of Intra-American Cement Manufacturers (FICEM), and the Finnish Road Administration (FinnRA). The study was coordinated by the International Study of Highway Development and Management tools (ISOHDM) Secretariat based within the Highways Management Research Group in the School of Civil Engineering at the University of Birmingham in the United Kingdom. This tool could be incorporated with highway management system at three levels including:

- Strategic level: At this level the software assists senior policy makers at long term budget planning or optimizing maintenance strategies of the whole network.
- Program level: At this level the software will assist in selecting candidate road sections of the whole network or subnetwork for maintenance under a particular budget. The time horizon is medium term (generally 5 years).
- Project level: At this level the software is used for alternative analysis through assessing physical, functional, and economic feasibility of specified project alternatives against the base case (doing nothing), then by maximizing NPV/ cost function, the most economic project alternative is introduced.

This software has four technical modules including:

- RUC (Which calculates Travel cost and VOC.)

- SEC (Which calculates AC and Environmental costs.)

- RD (Which calculates Road Deterioration cost.)

- MIE (Which calculates road way Maintenance and Improvement Effects.)

However, in this software, fixed unit costs are used for calculating user cost (RUC module) (Kerali et al., 1998).

HERS-ST (Highway Economic Requirements System–State Version) is a software developed by FHWA which estimates the future condition, performance, and user cost impacts result of highway investment. It also predicts the required investment to achieve a particular level of condition, performance, and user cost. HERS-ST is the state level version of HERS software which is used by FHWA since 1995 to provide estimates of the investment required to either maintain or improve the Nation's highway system.

HERS-ST input composed of current condition and performance data of the statewide highway sections including pavement condition, traffic volume, vehicle mix, and traffic capacity and financial study period. Using section-specific traffic growth forecasts, the software estimates future conditions and performance of each highway section at the end of study period and according to the engineering standards checks for deficiencies (pavement wear, volume to capacity ratio, etc.). Then using engineering practices, the software suggest a set of viable improvements to correct the deficiencies as well as the associated benefit cost ratio ($\frac{B}{C}$), equation 2.1.

$$\frac{B}{C} = \frac{User\ cost + Agency\ cost + Societal\ cost}{Initial\ improvement\ cost} \qquad 2.1$$

Where:

- B *Benefits associated with the improvement scenario*

- C	*Costs associated with the improvement scenario*
- $User\ cost$	*Change in travel time, crash, and vehicle operating costs*
- $Agency\ cost$	*Change in highway maintenance costs and the residual value of the projects*
- $Societal\ cost$	*Change in emissions*
- $Initial\ improvement\ cost$	*Project cost at time of implementation (right-of-way, acquisition, etc.)*

Using incremental cost benefit analysis the most cost effective improvement scenario for each highway section is identified by the software. The section improvements are prioritized based on their $\frac{B}{C}$, and the most cost effective options are selected for the statewide rehabilitation program, given the funding constraints, $\frac{B}{C}$ target, or performance objectives introduced by the user. The focus of this software is system level (statewide) transportation asset management rather than a project level one which is required in urban setting construction work zones impact analysis (FHWA, 2002).

2.3.2 User cost application

2.3.2.1 Work zone optimization.

Chein and Schonfeld in 2001 developed a mathematical model to optimize work zone length for four lanes highway with no intersections and interchanges. The objective function is minimizing total cost consisted of agency cost, UDC and accident cost and it is assumed that one lane in each direction is closed due to the work zone. The model inputs include differences in discharge rates and travel times with and without work zone, excess flow compared to the work zone capacity, accident rate, accident cost per vehicle hour, average and fixed agency cost per km of the work zone, and average and fixed maintenance time per km of the work zone. Based on their findings, longer work zone length causes higher UDC. In other words, using repeated shorter work zones instead of one long work zone will result in lower UDC but at the expense of increased agency cost. So, by using the developed model the optimum work zone length with minimum total cost could be decided. This model was developed for four lane highways with no interchanges or intersections (Chien & Schonfeld, 2001).

2.3.2.2 Justifying accelerated construction (AC) methods.

Wang & Goodrum (2005), developed UC tables for road construction projects in Kentucky to facilitate selection between conventional and accelerated construction (AC) methods as well as, selecting the type of AC. The tables give the value of work zone instigated UC (UDC component only) based on the ADT (average daily traffic), construction schedule (day time, overtime, and night time), type of terrain (mountainous, rolling, level), highway projects (four lane, six lane), trucks percentage (single units and combination separately), and speed (normal and work zone). The method this research uses for calculating the value of travel time differentiates it from early works and is, to some extent, similar to AASHTO (2010) and FHWA (2011) recommendations. In this study, three different travel time values are calculated: personal travel with passenger cars, business travel with passenger cars, and trucks. These values are calculated based on average income and wages of Kentucky counties which are scaled using HERS (Highway Economic Requirements Systems, Federal Highway Administration, 1998) recommended coefficients as shown in Table 2.1. The following is their method of calculating travel time value.

Table 2.1 Value of Travel Time Ranges as a Percent of National Wage Rate (HERS) (Wang & Goodrum, 2005)

Travel Type	Local	Intercity
Personal	35% - 60%	60% - 90%
Business	80% - 120%	80% - 120%
Truck	100%	100%

Personal travel time value: the average incomes of each county in Kentucky are acquired from the Kentucky Cabinet for Economic Development. The annual per capita income of each county in Kentucky is divided by 2080 (annual working hours according to Bureau of Labor statistics (BLS)) in order to calculate hourly per capita income of each county. The hourly income is converted to the study year [2004] income using inflation rate. Following HERS recommendation (Table 2.1), 80% of this value is considered as the hourly dollar value of personal travel time.

Business travel time value: Business travel time value of each county is calculated based on the average hourly wage of the county. To calculate average hourly wage of each county, the average hourly wage of Kentucky in 2003 ($15.15 in 2003 dollars) is extracted from BLS database, then using consumer price index (CPI) is converted to 2004 wages ($15.48). Using equation 2.2 the average hourly wage of each county is calculated. Following HERS recommendation (Table 2.1), 100% of this value is considered as the hourly dollar value of business travel time.

$$Average\ hourly\ wage\ of\ county_i = Average\ hourly\ wage\ of\ Kentucky \times \frac{Average\ per\ capita\ income\ of\ county_i}{Average\ per\ capita\ income\ of\ Kentucky} \quad 2.2$$

Where:

i: County indicator

Then counties are classified to four groups based on their hourly per capita income level (Table 3 of Wang & Goodrum, 2005). For each county group the average hourly value of travel time with passenger car (VOT) is calculated by equation 2.3.

$$VOT = business\ travel\ time\ value \times \%\ bussiness\ travel + personal\ travel\ time\ value \times \%\ personal\ travel \quad 2.3$$

Value of time for truck driver and cargo: The research uses Kentucky wide average value of time for truck drivers, which has already been established by the Kentucky Transportation

Cabinet. It is reasoned origin and destination of one-way truck trip is rarely at the same county, so statewide aggregated time value is more appropriate for this parameter. However, the report fails to include the dollar value of truck travel time used in the calculations of the UC tables.

Then using these travel time values for each county group, UC tables are developed (Wang & Goodrum, 2005). However, there is no more detail of how UC is calculated based on the parameters mentioned at the beginning of this section (ADT, construction schedule, type of terrain, etc.).

According to the researchers, the tables are intended to ease selecting AC method versus conventional methods. If the estimated user cost (EUC) using the tables is higher than maximum user cost (MUC) defined by the state transportation agencies, then AC method is selected. The researchers provide a decision flowchart for selecting the type of the acceleration based on the EUC/ MUC ratio for different type of the road construction works (Wang & Goodrum 2005). However, this flowchart seems mostly a detailed research proposal rather than a mature research work. As a strong point this research work is a transition between old methods (using a fix travel time value) and new methods (estimating travel time value based on the annual population statistics) of estimating UC and applying it in decision making process. However, it lacks clarification on details which leave the reader with question marks.

Thoft-Christensen in 2009 in a literature review paper, tries to justify the importance of performing life cycle cost benefit (LCCB) analysis rather than performing life cycle cost (LCCA) analysis in bridges management systems and emphasizes on inclusion of UC in total cost calculations while acknowledging the difficulty and uncertainty associated with estimating UC as a reason for not including it in LCCB and LCC analysis by practitioners. Thoft-Christensen recommends modelling UC stochastically, due to the stochastic nature of its influential parameters;

however, the paper does not provide any direction or framework of such modelling. The writer introduces UC concept in more detail and reemphasizes its importance and magnitude compare to agency cost by presenting a history of UC evolution through US state DOT's research works, and a research work in UK in 1999. The UC in this study is composed of UDC, AVOC, and AAC (Thoft-Christensen, 2009).

Sadasivam and Mallela in 2016 created ready-to-use reference charts for estimating UC associated with bridge work zone in US based on basic project characteristics, such as traffic volume, functional classification and available number of lanes. The charts are applicable for planning phase of a project. The aim is to help practitioners in selecting accelerated bridge construction (ABC) method versus conventional method during project alternative selection. In this study the term UC refers to the sum of UDC and AVOC and the source of their data is US National Bridge Inventory database. Using functional class (interstate, arterial), number of lanes at each direction (under normal & work zone condition), AADT (annual average daily traffic), AADTT (annual average daily truck traffic) and using the procedure defined at MicroBencost (McFarland et al., 1993) and Highway Capacity Manual (TRB, 2010), daily traffic delay due to the work zone was calculated. Delay time has been monetized using the following unit costs:

- UDC equal to \$21.88, \$23.06, and \$29.65 per hour for automobiles, single unit trucks, and combination trucks, respectively
- VOC equal to \$0.17/mile and \$0.82/mile for automobiles and trucks, respectively.

The results were categorized in the form of tables and graphs based on the functional class of a bridge, time of construction (day time versus night time), number of available lanes, and AADT, and UC that is extracted from each graph is adjusted to the percentage of AADTT using the provided adjustment tables. The adjusted user cost is in 2010 dollars which needs to be converted to the study year dollars using CPI -U (consumer price index for all urban consumers in US) (Sadasivam & Mallela, 2016).

Jia et al. (2016) developed a framework to compare total cost of ABC versus conventional bridge construction method for alternative analysis purposes. In this research, total work is defined as sum of construction cost (composed of labor, material, equipment, and contractor's overhead costs) and UC. In order to estimate ABC cost the researchers developed a regression construction cost model using the historical ABC construction cost data from all around USA since 1998 to 2013. These data were extracted from FHWA SharePoint database, which is an ongoing project developed under the National ABC 2 Project Exchange in the USA (Jia et al., 2016). For each bridge the final construction cost per square foot of the bridge deck was calculated and using regression analysis, this cost was defined as a function of AADT, number of bridge spans, bridge type (steel versus concrete), and bridge location (rural versus urban). This equation is a base for estimating construction cost of ABC alternative of any proposed bridge. Engineering, supervision, and other agency overhead costs are excluded from the comparison due to the limitation of the available data in this regard (Jia et al., 2016).

Four components are defined for UC of which two are different than what is commonly available in the literature:

- Value of travel time (UDC): Travel delay is estimated using QuickZone tool and monetized $16.64 / person/ hr.
- Travel reliability cost (TRC): This cost is defined as required buffer time due to the construction work multiplied by the hourly value of the buffer time which is $22.5/hr. Buffer time is extra time budgeted for a travel. The buffer time and its hourly value is calculated based on the recommendation of Second Strategic Highway Research

Program (SHRP 2). Since this cost category is not at the scope of this study, the further discussion on it is avoided and the reader is referenced to the source paper for further information.

- Accident cost (AAC): This cost is calculated with considering 30% increase in accidents rates due to construction using the specified method in the reference.
- Emission cost: This cost is calculated based on the recommendations of FHWA report, No. FHWA-HOP-12-005 (FHWA, 2010).

Using data from construction of one of the interstate bridges in US, conventional construction and ABC methods are compared. It is shown that, although the construction cost of ABC is higher than conventional methods its total cost is lower than conventional methods and based on the results is recommended alternative analysis to include total cost in comparison rather than only construction cost (Jia et al., 2016).

2.3.2.3 Contractual incentive/ disincentive (I/D) calculation.

FHWA’s Contract Administration Core Curriculum manual (2006) defines incentives (I) as “contractual provisions which compensates contractors for each day an identified critical work is completed earlier than the schedule” and disincentives (D) as “contractual provisions that charges the contractor for each day of the critical work is delayed beyond the schedule” (FHWA, 2006).

Sudarsana et. al in 2014 compare road user cost incurred due to road construction work zone with the minimum delay claims of the corresponding contracts in Indonesia. In this study UC consisted of UDC and AVOC. Based on Indonesian law, minimum delay claims per day is considered 0.1% of the contract price; in other words, construction contractors will be charged 0.1% of the contract price for every day of delay in project completion. The researchers use data for ten national road improvement projects in the Bally province, Indonesia, for 2012 fiscal year. Wilcoxon Sign Rank test and paired t-test are used as comparison tools and the study found that at 2.5% significance level there is significant difference between daily UC and the minimum daily delay claim value of the contract. For these ten links, the average value of the daily UC is 1.37% of the contract price while the minimum daily claim value of the contract is 0.1% of the contract price (Sudarsana et al., 2014).

Sadasivams & Mallela (2015) use UC in calculating contractual incentives and disincentives (I/D). Highway agencies often calculate I/D by applying a multiplier to UC, equation 2.4. The value of the multiplier could vary from 0.2 to 1.0 (Sadasivams & Mallela, 2015). For instance, California DOT guidelines typically recommends a multiplier equal to 0.5 while New Jersey DOT uses 0.25 as the multiplier (Sadasivams & Mallela, 2015).

$$I/D = Df \times UC \qquad 2.4$$

Where:

- I/D *Contractual incentives and disincentives*
- Df *Multiplier to calculate I/D based on UC*
- UC *User cost*

However, these researchers believe I/D calculation needs to incorporate the effect of discount factor and the risk that the agency is willing to share with the construction contractor. Based on prior research they defined a lower and upper bound to incentives and disincentives, equation 2.5.

$$CA \leq \frac{I}{D} \leq UC + \ AGC \qquad 2.5$$

Where:

- CA *Contractor's cost of acceleration*
- I *Contractual incentives*
- D *Contractual disincentives*
- AGC *Reduced or increased agency supervision cost*

The lower bound they defined is contractor's acceleration cost (CA) or extra cost incurred to the contractor due to the reducing project completion period. The upper limit they defined is UC plus or minus the estimated agency supervision cost (AGC) due to the early completion or delayed completion of the project, respectively. However, the value of UC needs to be discounted to the highway agency's investment dollars (Sadasivams & Mallela, 2015). The investment dollars are defined as the economic benefit that is created due to one dollar the agency spends in highway projects. For instance, the Massachusetts DOT spent an additional investment of \$ 1.75 million on accelerating construction for 42 bridges to produce a UC savings of \$ 136 million, therefore, the discount factor is 77.8. The researchers call this discount factor (ROIM) and to discount the UC to agency dollars, divide the UC with ROIM. Additionally, they believe the percentage of risk the agency is willing to share or recover from the contractor, referred to as RMF in this paper, should be incorporated in I and D calculation, in equation 2.6 and equation 2.7, respectively.

$$I = CA + RMF \times (\frac{UC}{ROIM} + AGC - CA) \qquad 2.6$$

$$D = RMF \times (\frac{UC}{ROIM} + AGC - CA) \qquad 2.7$$

Where:

- I *Contractual incentives*
- CA *Contractor's cost of acceleration*
- RMF *Percentage of risk the agency is willing to share with or recover from the contractor*
- UC *User cost*

- $ROIM$ *Discount factor*
- AGC *Reduced or increased agency supervision cost*
- D *Contractual disincentives*

The argument is that if the agency does not share any risk with the contractor (RMF = 0) there is no incentive for the contractor to accelerate the work and if RMF equals to 1, there is no saving or loss to the agency as the cost differentials (UC/ROIM + AGC - CA) is either paid to or recovered from the contractor through I/Ds. They hypothesized there is equilibrium to both agency and contractor with RMF between 0.4 and 0.6 (Sadasivams & Mallela, 2015). However, there is no further discussion on this hypothesis.

2.4 Summary

In this chapter research work on UC in Canada and other parts of the world with focus on the USA was presented. USA is the leading country in research on UC with the FHWA's 2011 report being the most comprehensive work on UC (Mallela, J. and Sadavisam, S., 2011). The current research trend focuses on application of UC in justifying AC methods and designing contractual I/D; however, both FHWA, 2011 report and other works use a deterministic approach for calculating UC while UC has a probabilistic nature. Additionally, most of the research work on UC focuses on freeways or rural roads while a lot of congestion occur in urban network as a result of construction. To address these shortcomings this research will apply traffic microscopic simulation to include the effect of nearby controls (traffic signals) in work zone instigated delay in urban setting (the subject of Chapter 3 and Chapter 4 of this thesis). Moreover, the probabilistic nature of UC will be addressed by incorporating Monte-Carlo simulation technique in Monetizing UC (subject of Chapter 5 of the thesis).

Chapter Three: Calculating User Delay Cost: The Case of Crowchild Bridge Rehabilitation

3.1 Introduction

In this chapter application of traffic microsimulation for calculating user delay due to one hour of construction work on a bridge during morning rush hour is presented. Data from the rehabilitation of Crowchild Tr Bridge in 2002 is used as the case study and traffic on Crowchild Tr study cordon will be micro-simulated both under normal operation of the bridge and work zone condition. The average simulated travel time of the road users in the study cordon under both conditions will be calculated and difference of these travel times is the delay that is imposed on the road users due to the work zone setup.

The following is the order of the contents of this chapter: Section 3.2 contains source of input data including; traffic count data, approach for resolving count data conflicts, vehicle classification, road data and signals timing; Section 3.3 covers field measurements, normal operation model, calibrating normal operation model, work zone configuration, work zone model and calibrating the work zone model; and in Section 3.4 the total delay imposed on road users in the study cordon including travelled through the work zone, detoured, and host users will be presented.

3.2 Crowchild Tr Input Data

3.2.1 Traffic volume.

The morning peak intersection count data was extracted from the data archive of the City of Calgary. The count data were in HTML format and were transformed to text format in order to be used with traffic simulation software (SimTraffic). A sample of the City's count data has been presented in Table 3.1. The study cordon composed of ten signalized and fifty five un-signalized intersections and depending on whether a three legs or four legs intersection, 21 or 28 count records were extracted for each intersection, respectively. Since the field travel time measurements of Crowchild Tr Bridge were taken in 2002, count data for 2002 was extracted and in cases where there was no 2002 count data, the count data of the closest year was used. Since Crowchild Tr Bridge study cordon has been located in a mature community, no considerable change in traffic volume in the span of two to three years is expected and such it was decided not to scale these count data to year 2002.

Table 3.1 Turning Movements Count Data (17 Ave SW & Crowchild Tr SW)

Traffic Count Reports

The City of Calgary
Transportation Department

Intersection Id: 9194 Study Name: 6 Hour Intersection Count Study Date: Wednesday, Aug 7, 2002

Verified: Yes Location: 17 AV SW & CROWCHILD TR SW

	North						South						East						West						Vehicle Totals
Period Beginning	North Left	North Straight	North Right	North Truck	North Ped	North Bike	South Left	South Straight	South Right	South Truck	South Ped	South Bike	East Left	East Straight	East Right	East Truck	East Ped	East Bike	West Left	West Straight	West Right	West Truck	West Ped	West Bike	
07:00:00	0	370	24	2	0	0	0	161	65	7	0	0	0	0	10	0	0	0	0	0	72	2	0	0	702
07:15:00	0	425	42	10	0	0	0	803	81	16	0	0	0	0	89	2	0	0	0	0	75	1	0	0	1515
07:30:00	0	463	52	13	0	0	0	816	95	19	0	0	0	0	118	2	0	0	0	0	87	2	0	0	1631
07:45:00	0	497	74	6	0	0	0	951	101	24	0	0	0	0	118	5	0	0	0	0	79	2	0	0	1820
08:00:00	0	452	50	21	0	0	0	792	81	17	0	0	0	0	109	1	0	0	0	0	64	1	0	0	1548
08:15:00	0	508	46	16	0	0	0	750	86	20	0	0	0	0	100	1	0	0	0	0	103	4	0	0	1593
08:30:00	0	447	57	17	0	0	0	756	91	21	0	0	0	0	95	0	0	0	0	0	71	5	0	0	1517
08:45:00	0	497	55	21	0	0	0	701	89	17	0	0	0	0	78	1	0	0	0	0	72	1	0	0	1492
TOTAL	0	3659	400	106	0	0	0	5730	689	141	0	0	0	0	717	12	0	0	0	0	623	18	0	0	11818
PEAK	0	1920	222	56	0	0	0	3309	363	80	0	0	0	0	445	9	0	0	0	0	333	9	0	0	6592
	PHF	0.94	Peak Total		2142		PHF	0.87	Peak Total		3672		PHF 0.94		Peak Total		445		PHF	0.81	Peak Total		333		
	Total Flow		11.8%				Total Flow		18.6%				Total Flow		2.1%				Total Flow		1.8%				
	Truck Flow		2.61%				Truck Flow		2.2%				Truck Flow		1.67%				Truck Flow		2.89%				
	Total Volume		4059				Total Volume		6419				Total Volume		717				Total Volume		623				

3.2.1.1 Conflicting count data.

Four types of conflicting data problems were observed in the Crowchild Tr study cordon including: count data for a nonexistent link, unbalanced volume, disappearing vehicles on a bridge, and generation nonexistent midblock vehicles. Methods for resolving the count data conflicts at locations presented in Table 3.2 have been detailed in the following paragraphs. As a general rule, resolving conflicts includes balancing volumes based on the volume of neighbouring intersections whose count data is consistent.

Table 3.2 Types of Conflicts in Crowchild Study Cordon

Location \ Conflict	Non Existent link	Unbalanced Volume	Disappearance Vehicles	Generating Midblock Vehicles
Crowchild NB & 17 Ave on Ramp	√	√		
14 St NW & Memorial Dr.		√	√	
Crowchild SB Kensington – Memorial EB off Ramp		√		√
Crowchild SB Memorial EB off – 10 Ave SW off Ramp		√	√	

Figure 3.1 provides an example calculation using data of “17 Ave SW & Crowchild” intersection on August 7, 2002 where, 445 vehicles were travelling from 17 Ave Westbound (WB) to Crowchild Northbound (NB) (Table 3.1), whereas there is no ramp from 17 Ave SW WB to Crowchild NB. Additionally, this number is in contradiction with 94 vehicles turning from 17Ave SW WB to Richmond Rd from where some of them could turn to Crowchild NB (Figure 3.1). It is worth mentioning that traffic count at all of these intersections (“Richmond Rd & 17 Ave SW”, “24 St SW & 17 Ave SW”, “Crowchild Tr & 17 Ave SW”) were performed on the same day, August 7, 2002.

In this case, the cumulative number of vehicles (570 veh) which are turning from both Richmond Rd NB and either direction of 17 Ave SW to Crowchild NB (333+37+200= 570) has been used in the analysis instead of 445 vehicles based on the count data, which have no ramp to enter to Crowchild NB (Figure 3.1). Additionally, based on the 2004 count data for “Crowchild Tr & Bow Tr” intersection, the total number of vehicles on Crowchild Tr NB shortly before Bow Tr EB off ramp equals 4,185 (2800 T + 1385 R) vehicles. There is no midblock vehicle source between this point and 17 Ave SW to Crowchild Tr NB, therefore, these vehicles are either turning from 17 Ave SW off ramp to Crowchild Tr NB (570 veh) or are traveling on Crowchild Tr NB where this ramp (17 Ave SW ramp) meets Crowchild Tr NB (3,309). As it is seen the sum of these vehicles (570+3,309= 3,879 veh) is less than the reported 4,185.

Figure 3.1 Traffic volume on 17 Ave SW & Crowchild Tr intersection (Google)

These two traffic counts have been done in two different years (2004 vs 2002). The count data on “Bow Tr & Crowchild Tr” intersection does not have the discrepancy of the count data of “17 Ave SW & Crowchild Tr” intersection, and the total volume of vehicles based on the count data of “Bow Tr & Crowchild Tr” intersection is larger than total volume of vehicles based on “17 Ave SW & Crowchild Tr” intersection. So, as a conservative approach the count data on “17 Ave

SW & Crowchild Tr” intersection is scaled up to be equal to the cumulative volume of vehicles on Crowchild Tr NB at “Bow Tr & Crowchild Tr” intersection (4,185 veh= 615R+3,570 T).

Another area of data conflict was “14 St NW & Memorial Dr NW” intersection. Based on the count data there are 243 through vehicles on NB 14 St NW; however, the total number of vehicles upstream of the intersection “14 St SW & 6 Ave SW” is 1,313. Additionally, the total number of vehicles NB of the downstream intersection “14St NW & Kensington Rd NW” is 1,127 (89 R, 934 T, 104 L). Since vehicles could not disappear on the bridge and considering the total volume of vehicles in the downstream intersection, the count data was discarded and vehicle volumes of different movements at this intersection calculated based on the four neighbouring intersections (“14 St NW & Kensington Rd NW”, “14 St SW & 6 Ave SW”, “Memorial Dr NW & Crowchild Tr”, “Memorial Dr NW & 10A St NW”).

These adjustments will prevent the software from unrealistically generating or omitting midblock vehicles in order to balance the count data which occurs when the traffic volume downstream of an intersection is higher than that of the intersection. If there is no space in the link for the generated vehicles to enter the network, they will be denied by the software to enter the link and the waiting time of these denied entry vehicles will be added to the total travel time of the vehicles that are already traveling on that specific link. So, the travel time will be unrealistically long. This is what would be modeled on 14 St NW between Memorial Dr NW and Kensington Rd NW if the aforementioned balancing had not been performed.

Similar normalization was carried out on Crowchild Tr SB between Kensington Rd NW and the off ramp of 10 Ave SW as presented in Table 3.3 (rows labeled “**After**”). Based on the count data 827 vehicles are added to Crowchild Tr SB traffic shortly after its intersection with Kensington Rd NW and before the Memorial Dr EB off ramp. In reality there is no such a midblock traffic source: in fact the midblock traffic is almost zero. Using these raw count data causes an unrealistic delay and increased travel time in SB segment of Crowchild Tr between Kensington Rd NW and Memorial Dr EB off ramp.

A similar problem occurs on the Crowchild Tr Bridge SB between Memorial Dr EB onramp and 10 Ave SW off ramp where vehicles could not be added or disappear on the bridge. So, the traffic between Kensington Rd NW intersection and 10 Ave SW off ramp was normalized based on the count data of 10 Ave SW off ramp. Normalizing traffic volume based on the count data of 10 Ave SW off ramp rather than "Kensington Rd NW & Crowchild Tr" intersection results in higher traffic volumes which are on the safe side. The traffic volumes before and after normalization are presented in Table 3.3 and Figure 3.2.

Table 3.3 Traffic Volume on Crowchild Tr SB Before and After Normalization

Location		Diverge (veh)	Through (veh)	Merge (veh)	Total (veh)	Count Year	Conflict Type
Immediately After Kensington	Before		426+2,602+184		3,212	2002	Generating Midblock Vehicles
	After		**486+2,972+210**		**3,669**		
Memorial EB off ramp	Before	740	3,299		4,039	2005	
	After	**672**	**2,997**		**3,669**		
Memorial EB on ramp	Before		3,299	223	3,522	2005	Disappearance of Vehicles on Bridge
	After		**2,997**	**203**	**3,200**		
10 Ave SW off ramp	Before	702	2,498		3,200	2002	
	After	**702**	**2,498**		**3,200**		

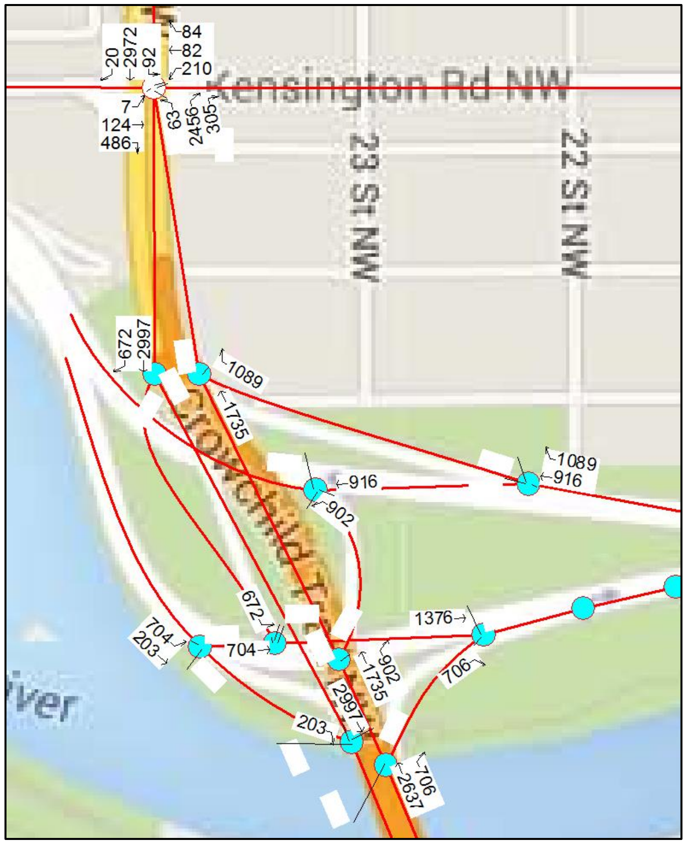

Figure 3.2 Traffic volume on Crowchild Tr SB after normalization (Google)

3.2.2 Vehicle classification.

The City's count data classifies vehicles as passenger cars or trucks which was applied in Synchro software wherever it was available, where it was not available: the City of Calgary recommended 2%, 5%, or 7% as trucks percentage on side streets, main streets, and industrial areas, respectively (Personal communication, 2015).

3.2.3 Other input data.

Other input data including road data and traffic data has been extracted either from local survey, Google map, or in consultion with the City of Calgary. A complete list of these data with their source is presented in Table 3.4.

Table 3.4 Input Data and Their Source

Data	Source	Data	Source
Roadway Alignment	Google Map	Traffic Control	The City of Calgary (Signal Timing Plan)
Speed Limit	Local Survey	Intersection Turn Counts	The City of Calgary (Count Data)
Allowable & Restricted Movements	Local Survey	Saturation Flow Rate	The City of Calgary
No of Lanes & Lanes' Width	Local Survey	PHF	The City of Calgary

3.3 Model Calibration

As mentioned in section 3.1, traffic on Crowchild Tr and neighbouring streets were modeled under two conditions; normal operation condition and work zone condition. The calibration process of both models is presented in the following sections. Travel time along the NB of Crowchild Tr between 17 Ave SW and Memorial Dr EB was recorded during both the Crowchild Bridge rehabilitation (30^{th} May to 20^{th} June, 2002) and after rehabilitation (2^{nd} November to 22^{nd} November, 2002), Table 3.5.

The measurements were carried out by two drivers with the exact the same driving characteristics, using the same vehicle each time and driving in the same lane (ie. no lane change was allowed). Measurements were taken using a steering wheel mounted stopwatch which were activated when passing under 17 Ave SW (NB travel) and stopped under Memorial Dr NW (NB travel). The opposite start stop locations where used for SB travel (ie. start at Memorial Dr NW and stop at 17 Ave SW). This research is reported in Wilson and Cowe Falls, 2003. As a matter of fact there is distribution of drivers on any roadway and using measurements based one or two specific drivers might not be the exact reflection of the average behavior on the study cordon.

Table 3.5 Field Measurement on Crowchild Tr NB Between 17 Ave SW & Memorial Dr

Condition	Date	Time of Day	Travel Time (s)	Average (s)	Standard Deviation (s)
Normal	04-Nov-2002	8:38 a.m.	81	79.57	2.44
	06-Nov-2002	8:42 a.m.	77		
	08-Nov-2002	8:40 a.m.	77		
	13-Nov-2002	8:34 a.m.	78		
	18-Nov-2002	8:35 a.m.	82		
	20-Nov-2002	8:42 a.m.	79		
	22-Nov-2002	8:37 a.m.	83		
Work Zone	03-Jun-2002	8:42 a.m.	189	189	6
	04-Jun-2002	8:18 a.m.	192		
	19-Jun-2002	8:25 a.m.	195		
	20-Jun-2002	8:18 a.m.	180		

Since travel time was the only available measurement, calibration of the model was attempted using travel time. There are two approaches to calibrate SimTraffic microsimulation models; adjusting global parameters, adjusting local parameters (Trueblood, 2013, Husch & Albeck, 2003). Global parameters are parameters that when changed affect the whole network and include driver parameters and vehicle parameters (Trueblood, 2013, Husch & Albeck, 2003). Driver parameters can be used to change drivers' reaction rate or to make the driver population more or less aggressive (Husch & Albeck, 2003). Vehicle parameters are used to change the vehicles characteristics including the percentage of cars, trucks, and buses in the fleet, the length and the width of the vehicles, and the acceleration rate and the maximum speed of the vehicles. Usually, the software developer recommends using the default vehicle parameters (Husch & Albeck, 2003) unless detail information on the vehicle fleet exists or unusual vehicle characteristics are being modeled such as mining vehicles in a closed network. Therefore, Driver parameters are the parameters used for global calibration purposes. Ten types of drivers are defined in this software, ranging from driver type one who is the most cautious and courteous driver to driver type ten who is the most risky and aggressive type of driver. The rest of the driver types fall between these two groups.

Thirteen driver parameters have been defined for each driver type in the software in order to characterize their driving behavior. The most prominent ones which are used for calibration purposes are speed factor and headway factor. Speed factor for each driver type is multiplied with the free flow speed (FFS) of each street. The product is the FFS which is used by the software for that specific type of the driver on that specific street. Headway factor is technically a measure of distance that each driver type will keep from the vehicle ahead of them. This factor (headway factor) is multiplied by the headway which is defined in Synchro software and for each driver type is defined for three speeds: 0 Km/h, 50 Km/h, and 80 Km/h. The software uses interpolation to calculate the headway factor for other speeds. A detailed definition of the remaining 11 factors could be found in SimTraffic manual (Husch & Albeck, 2003).

As mentioned previously, local parameters are the other means of model calibration in SimTraffic software, as adjusting these parameters would affect at the street level rather than at the network level. These parameters could be headway, FFS, etc. of each movement of the street that is studied. SimTraffic uses distribution for FFS, so, not every driver will have the same speed limit equal to the FFS of the study approach.

3.3.1 Calibrating Normal Condition Model.

Prior to calibration of the normal condition model for Crowchild Tr, the model was seeded for 60 minutes, then ten 60 minutes intervals were simulated. Based on the results of these ten simulations, the average travel time on Crowchild Tr NB between 17 Ave SW and Memorial Dr

EB is 106 s (Table 3.6). The result is significantly different than the field measurements; therefore, the model needs to be calibrated. The following is the steps have been taken to calibrate the model. They have been summarized in Table 3.7.

Table 3.6 Average Travel Time on Crowchild Tr NB without Any Calibration

Seed no.	90	91	92	93	94	95	96	97	98	99	Mean	S.D.	T statistic	P value
Travel time (s)	104.7	105.8	107.3	106.2	107.0	103.2	106.5	101.8	108.0	104.2	105.5	1.96	24.14	.0001

In the first trial FFS along Crowchild Tr was increased 5 Km/h which did not cause a considerable change on travel time. In the next trial, the speed factor was increased 20% and headway factor was decreased 25% which resulted in reducing travel time to 86.3 s. Still this result was not statistically equal ($t= 8.3$, $p<0.001$) to the field measurements (79 s) in Table 3.5, therefore, the speed factor was increased another 20% and again the result was not satisfactory. As a result, the speed factor increment was fixed at 20%. As the main delay to vehicles was incurred in the link between 17 Ave SW and off ramp from Crowchild NB to Memorial Dr EB, calibration in local level on this specific link was tried. This link is composed of two parts:

- Part 1: Between 17 Ave EB off ramp to Crowchild Tr NB and end of acceleration lane on Crowchild Tr NB
- Part 2: Immediately after part 1 up to Crowchild NB off ramp to Bow Tr EB

At the first trial headway on part 1 was reduced to 0.8, there was no meaningful reduction on simulated travel time. At the second trial, headway on the whole link was reduced to 0.8 and average simulated travel time was 86.2 s: in other word no practical change on travel time. Consequently, no local calibration was considered and the model was only calibrated on the global level. The average simulated travel time of the final model (20% increase on drivers' speed factor & 25% reduction on drivers' headway factor) is presented in Table 3.8.

Table 3.7 Simulated Travel Time on Crowchild Tr NB between 17 Ave SW & Memorial Dr Versus Global and Local Calibration Parameters (Normal Condition)

Trial No.	Speed Factor	Headway Factor	Local Calibration	Node No. [local calibration]	No. of Runs	Travel Time (s)
1	1.00	1.00	N/A	N/A	1	106.9
	1.00	1.00	N/A	N/A	10	105.5
2*	1.20	0.75	N/A	N/A	1	86.7
	1.20	**0.75**	**N/A**	**N/A**	**10**	**86.3**
3	1.40	0.75	N/A	N/A	1	91.0
4	1.20	0.75	Headway=0.8	1	1	85.5
5	1.20	0.75	Headway=0.8	1 & 53	10	86.2
Field Measurements						79.6

* Bold indicates final model.

Table 3.8 Average Travel Time on Crowchild NB Final Model, Normal Condition

Seed No.	1	2	3	4	5	6	7	8	9	10	Mean	S.D.	T statistic	P value
Travel Time (s)	85.6	87.7	85.6	86.4	86.6	85.6	87.1	85.3	86.5	86.7	86.3	0.77	8.30	0.0001

Based on the t test results, there is significant difference at the 5% significance level between simulated average travel time (86.3 s) and average field measurements (79.6 s). The difference can be explained as the field measurements were done between 8:34 am to 8:42 am, which is shortly after the peak hour, so slightly shorter travel time can be expected than during the peak hour travel time. Moreover, considering human error for pressing on and off the stop watch, there are chances of a few seconds of difference. In addition, the filed measurements were carried out by the same two drivers in the same cars while preserving the same driving pattern during the measurements, and as such, the measurements were representative of one driver group out of the ten driver groups of the simulation model. Given all the factors, the model at the current state is considered as a calibrated model.

3.3.2 Calibrating Work Zone Condition Model.

3.3.2.1 Work zone configuration and traffic volume.

Crowchild Bridge rehabilitation in 2002 was composed of three stages; Stage one included rehabilitation of onramps from 10 Ave SW and Bow Tr WB to Crowchild NB; Stage two followed, concentrated on rehabilitating the main bridge over the Bow River located between 10 Ave SW onramp and Memorial Dr EB off ramp and; Stage three rehabilitation of Memorial Dr EB off ramp. During stages one and two both onramps from 10 Ave SW and Bow Tr WB to Crowchild Tr NB were closed (Wilson & Cowe Falls, 2003). Additionally, based on Wilson & Cowe Falls, 2003 and available pictures during the rehabilitation of the main bridge, two lanes of the Crowchild Tr Bridge NB were closed and only one lane was open. The width of the open lane was three meters and the posted speed on the work zone was 50 Km/h. The open lane was the right side lane.

As previously mentioned, during Stage one and two of the rehabilitation, traffic from 10 Ave SW and Bow Tr WB onramps to Crowchild Tr NB was closed and therefore, this reduced the total traffic volume on NB of Crowchild Tr Bridge to 2,800 vehicles. In normal circumstances 52% of the total vehicles entering NB of Crowchild Tr Bridge drive straight up to Kensington Rd NW intersection. 21% of the total vehicles will turn to Memorial Dr EB and the remaining 27% will turn to Memorial Dr WB. The same percentages are used in order to estimate what number of the total vehicles on the bridge during the construction period (2,800 veh) will choose either destinations, Table 3.9 and Figure 3.3.

Table 3.9 Traffic Volume on Crowchild Tr NB During Normal and Work Zone Condition

Location	Condition	Diverge (veh)	Through (veh)	Merge (veh)	Total (veh)
Start of the Work Zone	Normal		2,800	543	3,343
	Work zone		**2,800**	**0**	**2,800**
Memorial Dr EB off ramp	Normal	706	2,637		3,343
	Work zone	**591**	**2,209**		**2,800**
Memorial Dr WB off ramp	Normal	902	1,735		2,637
	Work zone	**755**	**1,453**		**2,209**
Memorial Dr WB on ramp	Normal		1,753	1,089	2,842
	Work zone		**1,453**	**1,371**	**2,824**
Kensington Intersection	Normal	63+305	2,456		2,842
	Work zone	**63+305**	**2,456**		**2,842**

* Bold indicates work zone condition.

Figure 3.3 Traffic volume on Crowchild Tr NB during work zone condition

3.3.2.2 Detour assumptions.

During the first and second phases of the Crowchild Tr Bridge rehabilitation, both 10 Ave SW and Bow Trail WB onramps to Crowchild Tr NB were closed (Wilson & Cowe Falls, 2003). Information about the detour path of the vehicles that used to access Crowchild Tr NB via these ramps was not available and as a result, the following assumptions were made based on common sense.

- Vehicles which used to access to Crowchild Tr NB through 10 Ave SW onramp will detour from 10 Ave & 14 St intersection (286 veh) and will continue on 14 St NW in order to access to the north bank of the Bow River.

- Vehicles which used to access to Crowchild Tr NB through Bow Tr WB onramp will use 14 St NW in order to access the north bank of Bow River (257 veh). These vehicles will start their journey to 14 St from the 6 Ave SW & 14 St NB onramp.
- Based on the count data, 248 vehicles on Bow Tr EB turn to WB of 10 Ave SW. These vehicles could have three potential destinations including; Crowchild Tr NB, Bow Tr WB, and limited facilities at the end of 10 Ave SW. There is no information on the percentage of vehicles that will choose either of these destinations. Considering the communities on Bow Tr between 26 St SW and Crowchild Tr, chances of these vehicles to be destined on Bow Trail west bound is very low. There is limited number of businesses on WB of 10 Ave SW shortly after its intersection with Bow Tr, so, it is expected that majority of these vehicles will turn to Crowchild Tr NB in normal circumstances. Moreover, this number (248 Veh) is very close to the count data of the vehicles which use 10 St SW onramp to access Crowchild Tr NB (286 Veh). During construction, some of these detoured vehicles (used to access Crowchild Tr NB via 10 Ave SW onramps) might use other routes like Sarcee Tr in order to access to the north bank of Bow River or even might ignore the trip if either it is not necessary or there is a better replacement. With the lack of reliable data it is conservatively assumed that all of these vehicles (286 veh) are traveling to Crowchild Tr NB. Additionally, it is assumed they will detour via 10 Ave SW &14 St intersection (248 veh. out of 286 veh)

Based on the count data, in normal circumstances, 706 of vehicles on Crowchild Tr NB turn to Memorial Dr EB, 902 turn to Memorial Dr WB, and 1,735 travel straight toward Kensington Rd NW. In order to estimate the negative effect of detouring vehicles (due to the 10

Ave SW and Bow Tr WB onramps closure) on Memorial Dr travel time (between 14 St NW & Crowchild Tr) the following assumptions are made.

- The percentage of the detoured vehicles (286 veh + 257 veh) which will travel Memorial Dr EB, Memorial Dr WB, and Crowchild Tr NB is equal to the percentage of right turn, left turn and through movements on Crowchild Tr NB & Memorial Dr interchange before the rehabilitation (27%, 21%, 52%, respectively).
- Those detoured vehicles whose destination are Memorial Dr WB, will take 14 St NW off-ramp to Memorial Dr WB and continue their trip straight to Parkdale Blvd.
- Those detoured vehicles whose destination is Memorial Dr EB will continue straight on 14 St NB and will turn right in the 14st NW & Kensington Rd intersection.
- Those detoured vehicles, whose destination is Crowchild Tr NB, will travel Memorial Dr WB then will enter Memorial Dr WB onramp to Crowchild Tr NB. After that they will continue on Crowchild Tr NB up to the Kensington Rd & Crowchild Tr intersection. In normal condition, at this point some will go straight on Crowchild Tr NB, some will turn left to Kensington Rd WB and some will turn right to Kensington Rd EB. There is no information about the percentage of these turning vehicles. During detouring conditions part of these right turning vehicles might drive straight on 14 St NW up to 14 St NW& Kensington Rd intersection and then turn to their desired direction (Kensington Rd EB or WB). There is no information about the percentage of these potential route selections; as a result, this potential route selection is discarded and it is assumed all of these vehicle will turn to Memorial Dr WB and will head to “Kensington Rd NW & Crowchild Tr” intersection.

3.3.2.3 Calibrating work zone condition model.

In order to calibrate the work zone condition model of Crowchild Tr, the model was seeded for 60 minutes. Then five 60 minutes intervals were simulated. Based on these five simulation results, the average travel time on Crowchild Tr NB between 17 Ave SW and Memorial Dr EB is 401.9 s (Table 3.10). The result is significantly different than the field measurements (189 s).

Table 3.10 Average Travel Time on Crowchild NB without Any Calibration (Work Zone)

Seed no.	1	2	3	4	5	Mean	S.D.	T statistic	P value
Travel Time (s)	398.5	403.8	402.6	409.5	394.9	401.9	5.53	55.3	0.0001

Again, since travel time was the only available measurement, calibration of the attempted model was tried for travel time. As a first trial global parameters were changed. From the experience of the normal condition model, drivers' speed factor was increased 20% and their headway factor was reduced 25%. This resulted in average travel time of 325.5 s along Crowchild Tr NB from 17 Ave SW to Memorial Dr EB off ramp (Table 3.10).

Considering the fact that SimTraffic uses FFS distribution rather than a unique value, in the next step FFS along Crowchild Tr NB was increased 5 Km/hr. Additionally, since in work zones and before entering such zones, vehicles generally travel closer to each other, headway on NB was reduced 10% which reduced the average travel time to 290.5 s. The result was still significantly different than field measurements. The headway was decreased in 5% steps and average simulated travel time was measured and compared with the average travel time based on the field measurement (189 s). Based on these simulations 25% reduction on headway of vehicles traveling on NB Crowchild Tr between 17 Ave SW and off ramp to Memorial Dr EB would result in average travel time which is statistically equal to the average travel time based on the field measurements at 95% confidence level (Table 3.11, Trial No. 6). The detailed results for this model are available in Table 3.12.

Table 3.11 Simulated Travel Time on Crowchild Tr NB between 17 Ave SW & Memorial Dr NW Versus Global and Local Calibration Parameters (Work Zone Condition)

Trial No.	Speed Factor	Headway Factor	Local Calibration	Node/s [Local Calibration]	No of Runs	Travel Time (s)
1	1.00	1.00	N/A	N/A	5	401.8
2	1.20	0.75	N/A	N/A	8	325.5
3	1.20	0.75	FFS: 5km Increase Headway: 10% reduction	52,1,53,93,5,39,54 On Crowchild NB	1	285.1
					10	290.5
4	1.20	0.75	FFS: 5km Increase Headway: 15% reduction	52,1,53,93,5,39,54 On Crowchild NB	1	258.5
					10	268.75
5	1.20	0.75	FFS: 5km Increase Headway: 20% reduction	52,1,53,93,5,39,54 On Crowchild NB	10	227.16
6	**1.20**	**0.75**	**FFS: 5km Increase Headway: 25% reduction**	**52,1,53,93,5,39,54 On Crowchild NB**	**10**	**144.6**
Field Measurements	---	---	---	---	---	189.0

* Bold indicates the final model

Table 3.12 Average Travel Time on Crowchild NB Final Model, Work Zone Condition

Seed no.	3	4	5	6	7	8	9	10	11	12	Mean	S.D.	T statistic	P value
Travel Time (s)	93.8	184.5	142.4	122.1	109.4	136.7	131.4	220.6	193.8	111.5	144.6	41.43	2.08	0.059

3.4 Analysis of Results and Summary

In this study, the total delay due to one hour of construction work on NB of Crowchild Tr during morning peak has been analyzed. The methodology, assumptions, and calibration of the models have been discussed so far. As a quick review, two traffic microsimulation models were built using SimTraffic software (Version 6): one is simulating traffic on Crowchild Tr and other streets of the study cordon during normal condition (no work zone); the other model is simulating traffic on Crowchild Tr and other streets of the same study cordon under work zone condition. During the work zone condition two lanes (left & middle) of Crowchild Tr NB are closed.

In order to calculate total delay due to the construction work, the travel time per vehicle on the targeted streets during the normal condition and work zone condition is calculated, then in those targeted streets, the cumulative travel time (average travel time per vehicle × number of vehicles) under normal and work zone condition is calculated. The difference of these two cumulative travel times is the total delay per vehicle occupancy rate (voc) which is incurred due to the one hour of construction work in morning peak.

In order to calculate the average travel time per vehicle, each model (normal and work zone condition) were run ten times with different seeding numbers. Then the average of these ten runs (for the calibrated models) was considered as the average travel time per vehicle (Table 3.13).

The delay for vehicles traveling in the following three streets were calculated. The first one encompasses the work zone and the last two are assumed to be the detour path.

- Crowchild Tr NB between 17 Ave SW and Kensington Rd NW.
- 14 St NB between 6 Ave SW onramp and Kensington Rd NW.
- Memorial Dr WB between 14 St NW and Crowchild Tr.

Based on the results of this study, the total delay incurred to vehicles due to the one hour of construction work in morning peak is 169.2 (hr/voc). It is composed of 8.8 (hr/voc) of trucks delay and 160.4 (hr/voc) of cars delay (Table 3.13). This is total delay per vehicle occupancy of vehicles that are traveling through the work zone, vehicles which detoured, and host vehicles on 14 St NB and Memorial Dr NB.

It is worth mentioning an individual unit in SimTraffic software is vehicles rather than a vehicle occupant. So, the software calculates average travel time for a vehicle regardless of its occupancy rate (voc), which is why travel delay is presented in the unit of (hr/voc), rather than hr. The effect of vehicle occupancy rate in travel delay and UDC will be elaborated in Chapter 5.

In Chapter 4 using the method introduced in this chapter, sensitivity of user delay to work zone layout will be assessed. Three work zone layouts will be analyzed and travel delay associated with each work zone will be calculated.

Table 3.13 Total Delay of Vehicles on Crowchild NB, 14St NB, and Memorial Dr. WB Due to One Hour of Construction Work on Crowchild Tr NB

Description	# Route	Normal (Average 10 run)			Construction (Average 10 run)			% Trucks	Total Delay (hr/ voc)		
		Travel Time (s/voc)	Volume (veh)	Travel Time × Volume (hr/voc)	Travel Time (s/voc)	Volume (veh)	Travel Time × Volume (hr/voc)		Cumulative	Trucks	Cars
CrowChild [17 Ave Sw - Bow EB Tr off Ramp]	21	38.4	4,185	44.6	53	4,185	61.6	0.04	16.97	0.68	16.29
CrowChild [Bow EB Tr off Ramp - 10 Ave SW on Ramp]	21	28.6	2,800	22.2	71.7	2,800	55.8	0.04	33.52	1.34	32.18
CrowChild [10 Ave SW on Ramp - Memorial Dr. EB off ramp]	21	14.9	3,343	13.8	16.3	2,800	12.7	0.04	-1.16	-0.05	-1.11
CrowChild [Memorial Dr. EB off ramp - Memorial Dr. WB off ramp]	21	4.4	2,637	3.2	3.7	2,209	2.3	0.04	-0.95	-0.04	-0.91
CrowChild [Memorial Dr. WB off ramp - Memorial Dr. WB on ramp]	22	11.6	1,735	5.6	12.5	1,453	5.0	0.04	-0.55	-0.02	-0.52
CrowChild [Memorial Dr. WB on ramp - Kensington]	23	149.8	2,824	117.5	256.7	2,824	201.4	0.06	83.86	5.03	78.83
14 St [6 Ave on ramp - 14 st off ramp]	100	28	1,313	10.2	26.9	1,856	13.9	0.05	3.66	0.18	3.47
14 St [14 st off ramp- Kensington]	102	48.2	1,127	15.1	49.6	1,274	17.6	0.03	2.46	0.07	2.39
Off ramp from 14 st to Memorial Dr. West	120	22.2	186	1.1	22.9	583	3.7	0.05	2.56	0.13	2.43
Memorial Dr. WB [off ramp from 14 st acceleraton lane length]	121	18.3	186	0.9	18.6	583	3.0	0.05	2.07	0.10	1.96
Memorial Dr. WB [14 st off ramp acceleraton lane to Crowchild NB ramp]	103	42.2	889	10.4	63.3	1,553	27.3	0.05	16.88	0.84	16.03
Memorial Dr. WB [Split from EB to before Crowchild NB ramp]	107	15.2	1,089	4.6	16.8	2,402	11.2	0.05	6.61	0.33	6.28
Memorial Dr. WB [Crowchild NB ramp]	104	27.6	1,089	8.3	29.7	1,371	11.3	0.05	2.96	0.15	2.81
Memorial Dr. WB [Memorial Dr WB after Crowchild NB ramp]	105	8.5	916	2.2	8.6	1,031	2.5	0.05	0.30	0.02	0.29
Total									169.2	8.8	160.4

Chapter Four: Work Zone Generated User Delay Cost: The Case of Bow Bridge Rehabilitation

4.1. Introduction

In Chapter 3 user delay due to one hour of construction work in morning peak hour was presented. In that case the work zone composed of closing all but one Northbound (NB) lanes of Crowchild Bridge. Using this method, the effect of three different work zone configurations on user delay due to the rehabilitation of the Bow Bridge in 2003 is studied in order to evaluate how different work zone configurations could affect road user delay differently.

Construction work on Bow Bridge interrupted access from "Trans Canada Highway (TCH) & Home Rd" intersection to "TCH & Sarcee Tr" interchange along TCH. In this study, travel times between these two points under three different work zone configurations are simulated and the difference between resultant average simulated travel time and the average simulated travel time under normal condition on this segment of TCH is calculated . These values are excess travel time imposed on road users due to the different work zone configurations which is then converted to the associated user delay cost (UDC). The examined work zone configurations are as follows (Figure 4.1):

- Model 1: Closing one lane in each direction (work zone in 2003, Section 4.3)
- Model 2: Complete detour of Westbound (WB) traffic via nearby major street (Bowness neighbourhood, Section 4.5)
- Model 3: Closing one direction and diverting traffic to the opposing direction (traffic light control, Section 4.6)

In this chapter the source of input data and potential conflicts are presented as well as resolution mechanisms (Section 4.2), then each of the three work zone configurations (Sections

4.3, 4.5 and 4.6) as well as normal operation model of Bow Bridge traffic study cordon (section 4.4) are detailed. Additionally, in these sections the required average simulated travel time in order to get from "TCH & Home Rd" intersection to "TCH & Sarcee Tr" interchange for the associated work zone configuration is presented. The same applies to Section 4.4 (Normal operation model). In Section 4.7 user delay associated with these three work zone configurations are compared. Capitalizing user delay will be discussed in chapter 5.

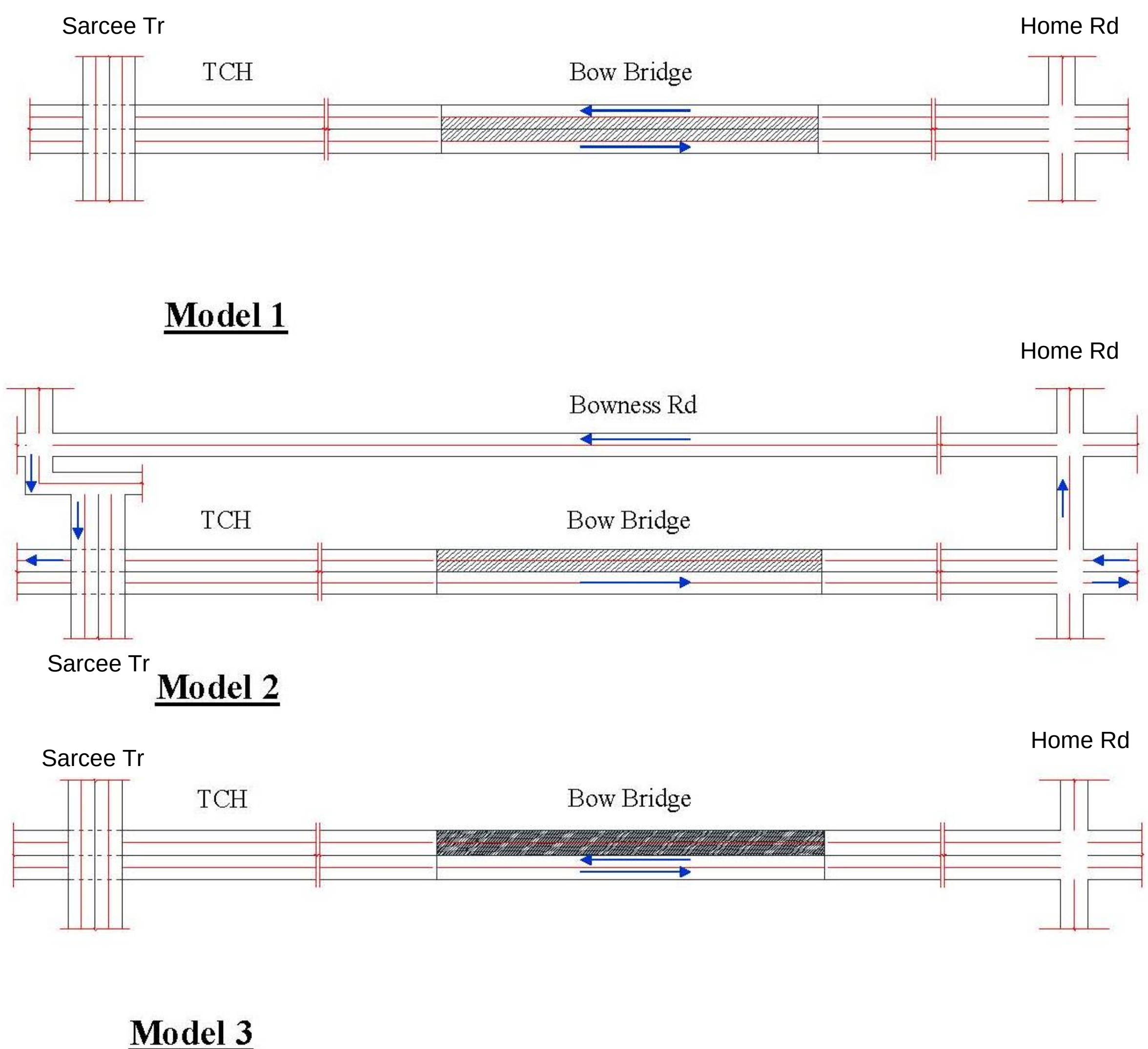

Figure 4.1 Studied work zone configurations

4.2. Input Data

4.2.1. Traffic volume.

The morning peak intersection count data was extracted from the data archive of the City of Calgary. Since the field travel time measurements of Bow Bridge were taken in 2003, count data for 2003 was extracted and in cases where there was no 2003 count data, the count data of the closest year was used. Since Bow Bridge study cordon has been located in a mature community, no considerable change in traffic volume in the span of two to three years is expected and such it was decided not to scale these count data to year 2002.

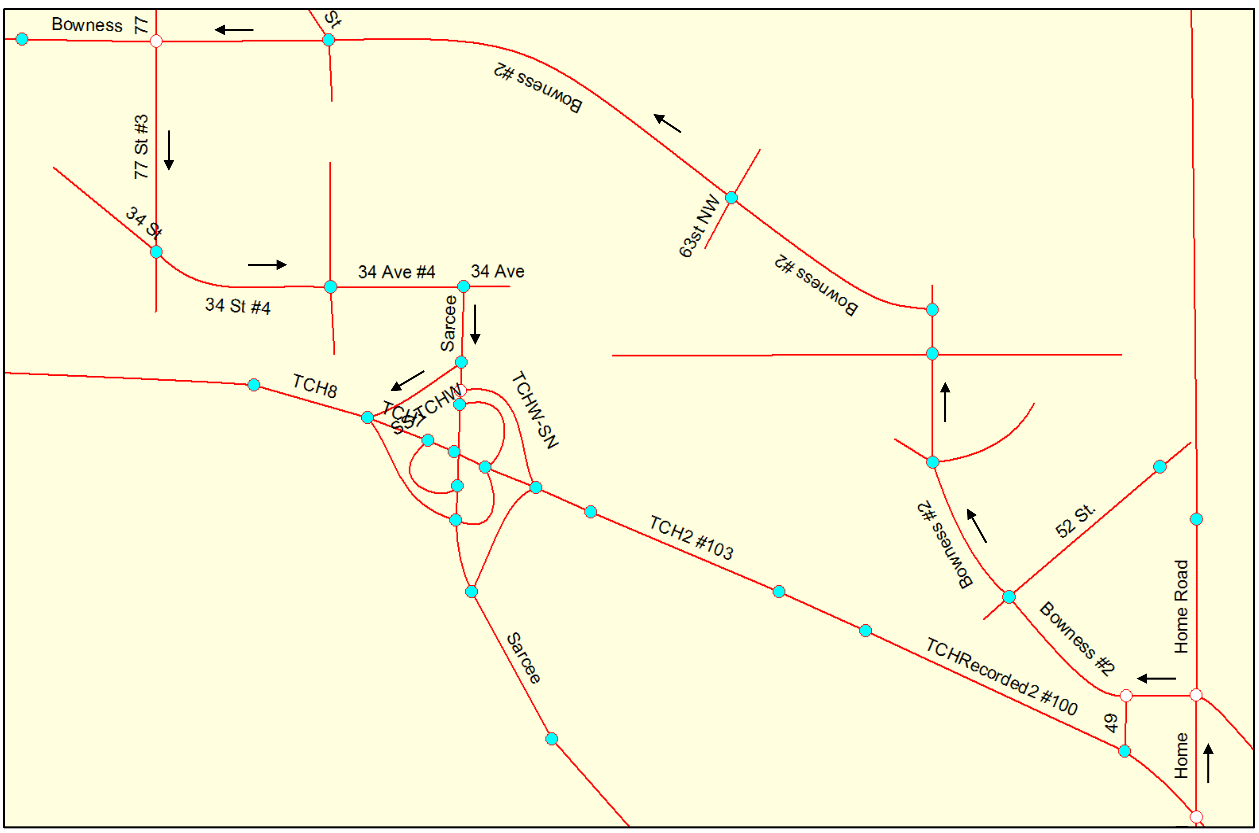

Figure 4.2 Bow Bridge study cordon and detour path

4.2.1.1.Conflicting count data.

One of the cases of conflicting data was observed at “TCH & 49 St NW” intersection. While eastbound (EB) midblock traffic between this intersection and the downstream intersection (TCH & Home Rd) is zero, the recorded EB through movement volume (1,757 veh) is about 26% lower than the same volume at TCH & Home Rd intersection (2,365 veh). Moreover, it is lower than the EB traffic volume of the upstream interchange (TCH & Sarcee Tr, 2,267 veh), but there is no ability for midblock traffic to exit between these two intersections. As a result, the EB traffic volume at this intersection was normalized based on the traffic volume of the closest intersection (TCH & Home Rd)

*Normalized right turn= 271 * 2,365 / 1757= 365 veh*

*Normalized through movement= 1757 * 2,365 / 1757= 2,365 veh*

No change was made to the WB traffic volume due to the midblock traffic generated by the market place located at the WB of TCH between Home Rd and 49 St NW.

Additionally, the EB traffic volume on TCH & Sarcee Tr intersection was slightly adjusted based on the traffic volume on EB of TCH & Home Rd intersection. This will prevent the simulation software from unrealistically generating midblock vehicles in order to balance the count data. In real world, there is no midblock traffic on EB of TCH between Sarcee Tr and Home Rd intersections. As previously mentioned, a problem occurs when the software generates midblock vehicles and there is no space in the link for the generated vehicles to enter the network. These vehicles will be denied by the software to enter the link and the waiting time of these denied entry vehicles is added to the total travel time of the vehicles that are already traveling on that specific link and the total travel time will be unrealistically long.

For instance, in Model 1 the simulated average travel time on EB of TCH between 49 St NW and Home Rd based on the unadjusted volumes was 576 s. This link is only 225 m long and such a long travel time is excessive. After the abovementioned adjustment, the simulated average travel time on this link reduced to 95.1 s, both of these travel times are before calibration of the traffic simulation model (section 4.3.1).

4.2.2. Vehicle classification.

In Bow Bridge study cordon, TCH has truck traffic, but as mentioned in Section 3.2.2, the City's count data classifies vehicles to passenger cars and trucks, but does not clarifies the type of the trucks (single unit truck or semi-truck). Based on count data at the TCH & Home Rd intersection 4.2% of the morning peak hour traffic in both directions are trucks.

Field measurement of vehicle classification at the intersection of TCH & 49 St NW from June 4, 2003 to June 12, 2003 was conducted and as this intersection is very close to TCH & Home Rd intersection, these measurements could be a good estimate of vehicle classification in TCH & Home Rd intersection. The average percentage of different classes of vehicles in the morning hours (8:30 am to 12.00 pm) based on the field measurements is available at Table 4.1.

Table 4.1 Average Vehicle Classification Based on the Field Observation, June 4 to 12, 2003

Vehicle Classification	Passenger Car	Light Trucks/ Buses	Semi-trucks	Recreational Vehicles
% of Traffic Stream	92.8	3.7	2.5	1.0

According to this table 6.2% of the vehicles are trucks and recreational vehicles forms 1% of the traffic (note: recreational vehicles in the software have been introduced as single unit trucks (SU). The percentage of trucks and passenger cars in the field are close to the count data. The vehicle classification based on the field measurements is more in detail and closer to the software vehicle classification (which classifies vehicles to two types of passenger cars, SU trucks, light trucks, double trucks, buses, and car pools). Consequently, average vehicle classification based on the field observation has been used as TCH vehicle classification in this analysis.

4.2.3. Other input data.

Other input data including road data and traffic data has been extracted either from local survey, Google Map, or in consultation with the City of Calgary. A complete list of these data with their sources are available in Table 4.2

Table 4.2 Input Data and Their Source

Data	Source	Data	Source
Roadway Alignment	Google Map	Traffic Control	The City of Calgary (Signal Timing Plan)
Speed Limit	Local Survey	Intersection Turn Counts	The City of Calgary (Count Data)
Allowable & Restricted Movements	Local Survey	Saturation Flow Rate	The City of Calgary
No of Lanes & Lanes' Width	Local Survey	PHF	The City of Calgary

4.3. Closing One Lane In Each Direction (Model 1)

In this scenario one lane in each direction of TCH on Bow Bridge is closed for the rehabilitation work. There are field measurements of travel time during the bridge rehabilitation. where the recorder was stationed on WB of TCH at its intersection with 49 St NW. The measured travel times represents the time that vehicles needed to travel between TCH & Home Rd intersection and start of the work zone. The average travel time measurements between 8:00 am to 8:56 am are presented in, Table 4.3. There is no information on the work zone speed limit and the available lane width during the rehabilitation period therefore the posted speed on Crowchild Bridge rehabilitation work zone was used (50 Km/h). Similarly, there is no information on lane width so no change of the lane width was made in the model.

Table 4.3 Field Measurement on TCH Between Home Rd and Bow Bridge, 2003

Direction	Date	Travel Time (s)	Comments	Average (S)	Standard Deviation (S)
Eastbound	4-Jun-2003	161		133.57	24.33
	4-Jun-2003	113			
	4-Jun-2003	147			
	4-Jun-2003	114			
	6-Jun-2003	124			
	12-Jun-2003	167			
	12-Jun-2003	109			
Westbound	4-Jun-2003	151		136	10.24
	4-Jun-2003	235	Major Outlier		
	4-Jun-2003	129			
	4-Jun-2003	125			
	6-Jun-2003	131			
	12-Jun-2003	134			
	12-Jun-2003	146			

4.3.1. Calibrating Model 1.

The calibration concept in SimTraffic microsimulation software has been detailed in Chapter 3, section 3.2 of this thesis. As a result, this chapter will avoid repeating those concepts and the final results will be presented for Bow Bridge study cordon.

In order to start calibrating Bow Bridge model under the Model 1 work zone condition, the model was seeded for 60 minutes, then ten 60 minutes intervals were simulated. Based on the results of these ten simulations, the average travel time of vehicles on EB & WB between Bow Bridge & Home Rd is 95.1 s and 72.1 s, respectively. These results are significantly different than the field measurements (EB: T=4.29, p=0.0009, WB: T=16.8, p=0.0001). Comparing the field measurements with the simulation results, it appears simulated travel time by the software, regardless of the movement direction, is less than the measured travel time. This means that in the real world vehicles in the study cordon move with lower speed and larger headway than the simulated speed and headway.

In normal condition, the posted speed on WB of TCH after TCH & Home Rd intersection is 80 Km/h while the posted speed on the EB of that segment of TCH is 50 Km/h. Additionally, as there is a traffic light ahead of the vehicles that are moving on the EB of this segment (traffic light at TCH & Home Rd intersection), it is expected that travel time on the WB of this segment will be shorter than the travel time of the EB. This is what actually simulation result demonstrates (72.1 s versus 95.1 s). But, based on the field measurements travel time on both directions of this segment (between Home Rd and Bow Bridge) during the construction work was almost the same (136 s versus 133.6 s). This means that due to the construction work, vehicles on the WB were traveling with average speed less than the posted speed (80 Km/h). So, in local calibration level free flow speed (FFS) on WB of this segment of TCH needs to be reduced. In contrast, those vehicles traveling on the EB have already left the bottle neck of the work zone and they will travel on FFS.

Similar to the Crowchild Bridge model, calibration on both global and local level were carried out. On global level, drivers' speed factor was decreased step by step and their headway factor was gradually increased. In the final model drivers' speed factor was decreased 35% and their headway factor was increased 25%. As it was mentioned in the previous chapter, SimTraffic software uses distribution for drivers' FFS, so not every driver has the speed limit equal to FFS. As a result, the work zone FFS was reduced 10 Km/h and FFS on WB of TCH between Home Rd and the work zone was reduced 30 Km/ h. The last scenario (reduction on FFS of this segment of TCH) is expected. Due to the presence of the work zone ahead, if the WB traffic demand is higher than the work zone capacity, vehicles on the WB of TCH, shortly after leaving TCH & Home Rd intersection, will face the work zone bottle neck and their speed will gradually drop to a range close to the speed limit of the work zone. The field measurements demonstrate this phenomenon. The simulated travel time values on EB & WB of TCH between Home Rd and Bow Bridge for average of ten simulated travel times for 60 minutes of seeding with different seed numbers and 60 minutes of simulation after calibration are presented in Table 4.4.

Table 4.4 Average Simulated Travel Time on EB & WB of TCH Between Home Rd & Bow Bridge, 2003 (Model 1, Closing one Lane in Each Direction)

Seed No.	20	21	22	23	24	25	26	27	28	29	Mean	S.D.	T Statistic	P Value
EB Travel Time (s)	137.6	140.6	136.6	137.4	130.6	116.7	135.6	134.5	136.1	122.5	132.8	7.5	0.1	0.9276
WB Travel Time (s)	124.5	128.5	130.2	127.4	127.3	128.6	131.7	129.5	129.1	124.7	128.2	2.3	2.4	0.0320

As seen in Table 4.4 the average simulated travel time on the EB is statistically equal to the average field travel time measurements at 95% significant level, but there is a 7.8 s (5.7%) difference between the average simulated travel time on WB and the average field travel time measurements. This is a local effect and further change on the local parameters to make the simulated average travel time on WB statistically equal to the average field travel time measurements at 95% significant level seemed unrealistic, mentioning that this effort did not cause considerable change on WB travel time. Additionally, there are chances of potential human errors on field measurements which may account for the 5.7% difference including:

- The observer positioned in the halfway between the origin and destination points
- Reaction time required by the observer to start and stop the stopwatch,
- Potential tiredness of the observer.

Consequently, the model is considered calibrated at this stage.

4.3.2. Analysis result (Model 1).

In case of closing one lane in each bound of TCH during the rehabilitation period (Figure 4.1, Model 1), vehicles driving on the segment of TCH between Home Rd & Sarcee Tr are the ones who will be mostly affected by the work zone. As a result, the average simulated travel time on both EB and WB of this segment of TCH based on ten runs of the calibrated model with different seeding numbers (first row of Table 4.4) has been calculated. The average simulated travel times on this segment of TCH are 485.6 (s/veh/voc) and 244.9 (s/veh/voc) for EB and WB movements, respectively, (3rd column of Table 4.5). As explained in Section 3.4, the software calculates average travel time for vehicles regardless of their occupancy rate (voc). That is why in travel time calculations (s/veh/voc) unit is used interchangeably with (s) unit.

Table 4.5 Travel Time on TCH Between Home Rd & Sarcee Tr Due to One Hour of Construction Work on Bow Bridge During Morning Peak, Model 1

Description		# Route	Construction			
			Travel Time (s/veh/ voc)	Volume (veh)	% Trucks	Travel Time × Volume (hr/voc)
TCH EB	[Home Rd - 49 St NW]	100	54.6	2,365	3	35.9
	[49 St NW- Bow Bridge]	100	80.2	2,730	2	60.8
	[Bow Bridge]	11	38.0	2,730	5	28.8
	[Bow Bridge - End of acceleration lane]	11	134.7	2,730	5	102.1
	[End of acceleration lane - Sarcee Tr NB off ramp]	11	68.7	2,730	5	52.1
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	11	48.3	2,289	5	30.7
	[Sarcee Tr NB on ramp - Sarcee Tr]	11	24.2	2,302	5	15.5
	[Sarcee Tr- Sarcee Tr SB on ramp]	11	14.7	2,302	5	9.4
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	11	22.2	2,184	5	13.5
	Total EB		485.6			348.8
TCH WB	[Home Rd - 49 St NW]	100	30.2	765	10	6.4
	[49 St NW- Bow Bridge]	100	98.0	855	7	23.3
	[Bow Bridge]	11	35.3	855	5	8.4
	[Bow Bridge - End of acceleration lane]	11	39.1	855	5	9.3
	[End of acceleration lane - Sarcee Tr NB off ramp]	11	10.6	817	5	2.4
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	11	9.2	569	5	1.5
	[Sarcee Tr NB on ramp - Sarcee Tr]	11	6.5	1,112	5	2.0
	[Sarcee Tr- Sarcee Tr SB on ramp]	11	5.5	1,112	5	1.7
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	11	10.5	1,112	5	3.2
	Total WB		244.9			58.2

4.4. Normal Condition Model

The available travel time measurements on Bow Bridge cordon are limited to the rehabilitation period (Model 1). Generally speaking, during the rehabilitation period if traffic demand is less than work zone capacity, vehicles will travel under free flow condition before the work zone, with reduced speed through the work zone and finally will leave the work zone under free flow condition. In this case the only influential factor on their travel condition will be downstream factors, like a downstream traffic light. However, if traffic demand is more than the work zone capacity, congestion will occur before the work zone and the work zone will act as a bottle neck. In this case vehicles will travel through the work zone with reduced speed and will leave the work zone with free flow condition. Again the flow condition after the work zone will be influenced only by the downstream factors. As a conclusion, it could be said that the traffic condition after the work zone is only affected by the downstream factors.

As discussed, there is no field travel time measurement of normal operating condition of Bow Bridge study cordon in year 2003, but there are travel time measurements on EB of a segment of TCH immediately after the work zone and before Home Rd. Therefore the travel time measurements on EB of TCH immediately after the work zone will be considered as the best estimate of the travel time on this segment of TCH under normal operating condition. In this case with changing global parameters the model was calibrated to present the same travel time on EB of TCH. With 35% decrease on drivers' speed factor and 25% increase on drivers' headway factor at 95% confidence level, the average simulated travel time on EB of this segment of TCH, is statistically equal to its average field travel time measurements, Table 4.6 (EB).

Table 4.6 Average Simulated Travel Time on EB of TCH between Home Rd & Bow Bridge, 2003 (Normal Condition)

Seed No.	1	2	3	4	5	6	7	8	9	10	Mean	S.D.	T Statistic	P Value
EB Travel Time (s)	138	136	116.9	121.9	116.2	137.9	137	136	120.4	138.6	129.9	9.7	0.435	0.6698
WB Travel Time (s)	74.5	73.7	74.6	75.5	75.2	74.3	73.6	74.9	74.8	74.9	74.6	0.6	N/A	N/A

4.4.1. Analysis result (normal condition model).

Using the calibrated normal condition model, the average simulated travel time on both EB and WB of the segment of TCH between Home Rd & Sarcee Tr are calculated. The model was run ten times with different seeding numbers (1^{st} row of table 4.6). Based on the result of these runs the average simulated travel times on EB and WB of this segment of TCH are 234.8 (s/veh/voc) and 174.9 (s/veh/voc), respectively, (3^{rd} column of Table 4.7).

Table 4.7 Travel Time on TCH between Home Rd & Sarcee Tr During Morning Peak, Normal Condition Model

Description		# Route	Construction			
			Travel Time (s/veh/voc)	Volume (veh)	% Heavy vehicles	Travel Time × Volume (hr/voc)
TCH EB	[Home Rd - 49 St NW]	100	52.7	2,365	3	34.6
	[49 St NW- Bow Bridge]	100	80.6	2,730	2	61.1
	[Bow Bridge]	11	19.1	2,730	5	14.5
	[Bow Bridge - End of acceleration lane]	11	38.2	2,730	5	29.0
	[End of acceleration lane - Sarcee Tr NB off ramp]	11	12.3	2,730	5	9.3
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	11	9.3	2,289	5	5.9
	[Sarcee Tr NB on ramp - Sarcee Tr]	11	6.5	2,302	5	4.2
	[Sarcee Tr- Sarcee Tr SB on ramp]	11	5.2	2,302	5	3.3
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	11	10.9	2,184	5	6.6
	Total EB		234.8			168.5
TCH WB	[Home Rd - 49 St NW]	100	21.6	765	10	4.6
	[49 St NW- Bow Bridge]	100	53.1	855	7	12.6
	[Bow Bridge]	11	18.1	855	5	4.3
	[Bow Bridge - End of acceleration lane]	11	38.9	855	5	9.2
	[End of acceleration lane - Sarcee Tr NB off ramp]	11	10.9	817	5	2.5
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	11	9.4	569	5	1.5
	[Sarcee Tr NB on ramp - Sarcee Tr]	11	6.6	1,112	5	2.0
	[Sarcee Tr- Sarcee Tr SB on ramp]	11	5.6	1,112	5	1.7
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	11	10.7	1,112	5	3.3
	Total WB		174.9			41.7

4.5. Complete Detour of EB Traffic Via Bowness Neighbourhood (Model 2)

In this section after introducing detour path, volume adjustment in the study cordon due to detouring will be explained (subsection 4.5.1), then the simulated average travel time of the detoured traffic will be presented (Model 2 – Option 1). As it will be detailed in subsections 4.5.2 to 4.5.5, because of congestion on Home Rd NB which results in blockage of Bowness Rd EB and 49 St NW, three other scenarios have been studied including; Closure of left turn from TCH (EB) to 49 St NW (Model 2 – Option 2), basic optimizing signals affecting Traffic on Home Rd (Model 2 - Option 3), and combination of basic optimizing signals affecting Traffic on Home Rd as well as closure of left turn from TCH (EB) to 49 St NW (Model 2 - Option 4). Travel time associated with each option will be detailed in the respective subsection. The final detour option with associated user delay will be presented on Section 4.5.6.

In the detour scenario, it is assumed that during the rehabilitation period WB of TCH at the work zone area is completely closed for construction purposes, but EB operates normally, Figure 4.1 Model 2. Traffic which used to take TCH WB in order to travel from "TCH & Home Rd" intersection to "TCH & Sarcee Tr" interchange will use "Home Rd NB, Bowness Rd WB, 77 St NW SB, 34 Ave NW EB, and finally Sarcee Tr SB" in order to do this trip, Figure 4.2. When the rehabilitation work on WB of the Bow Bridge is finished, the rehabilitation work on the EB will start. During the construction work on EB of the bridge, the EB traffic will be temporarily detoured to WB of the bridge in the work zone area. This traffic will be back to EB after crossing the work zone. During the entire rehabilitation period, WB traffic will detour through Bowness neighbourhood via the described detour path.

4.5.1. Volume adjustment (Model 2).

Sources of vehicle on WB of TCH before Bow Bridge are (Figure 4.3):

- Right turns from SB of 49 St NW (85 veh)
- Right turn from SB of Home Rd (100 veh)
- Through vehicles on WB of TCH (653 veh)
- Left turn from NB of Home Rd (12 veh)

Twenty five of the vehicles from the last three origins will turn to NB of 49 St NW, but the rest will cross the bridge. There is no information about what percentage of these 25 vehicles comes from which of the abovementioned origins. With the lack of precise data, as a best guess it is assumed that the ratio of vehicles from each origin is proportional to its traffic volume, Table 4.8. On this basis, four of these vehicles are right turns from SB of Home Rd and 21 are through vehicles on WB of TCH. Based on the common scenes the following assumptions were made as well.

- During the detour scenario all traffic on WB of TCH apart from these 25 vehicles will detour to NB of Home Rd at intersection of TCH & Home Rd and will continue the detour path mentioned in Section 4.5. The total is 740 vehicles (632 [TCH, WB] + 96 [Home Rd, SB] + 12 [Home Rd, NB]).
- The turning vehicles from SB of 49 St NW to TCH WB (85 veh) will left turn from "49 St NW & Bowness Rd" intersection to WB of Bowness Rd and will continue the previously defined detour path.
- The total of detoured vehicles to Bowness Rd WB from 49 St NW & Bowness Rd intersection onward is 825 vehicles. These vehicles will be added to traffic demand on the detour path.
- In the normal operating condition, 244 vehicles turn from TCH WB to Sarcee trail. Twenty-four of these 244 turning vehicles used to right turn from TCH WB to Sarcee Tr

NB and advance to 34 Ave NW. The remaining 220 vehicles used to left turn from TCH WB to Sarcee Tr SB. In the detour scenario it is assumed there is no turning movement from TCH WB to Sarcee Tr because these turning vehicles are supposed to detour through Bowness Rd.

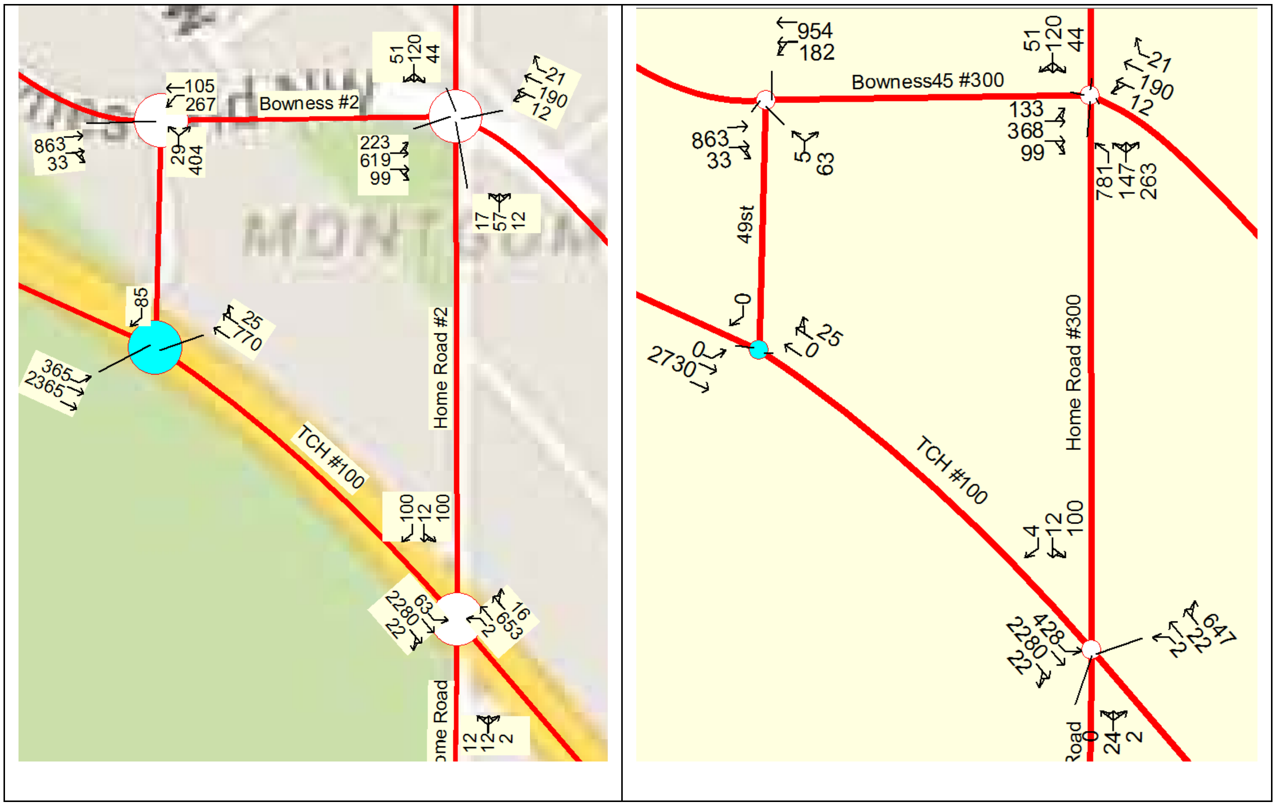

Figure 4.3 Traffic volume on TCH & Bowness Rd before detour (left) & after detour (right), Model 2- Options2 & Model 2- Options 4

Table 4.8 Estimating Origin of Right Turning Vehicles from EB of TCH to NB of 49 St NW

Origin	No. of Vehicles
WB of TCH (through)	$20.9 = \dfrac{25 \times 653}{563 + 100 + 12} \rightarrow say\ 21$
SB of Home Rd (right)	$3.7 = \dfrac{25 \times 100}{563 + 100 + 12} \rightarrow say\ 4$
NB of Home Rd (left)	$0.4 = \dfrac{25 \times 12}{563 + 100 + 12} \rightarrow say\ 0$

- Additionally, it is assumed 24 of these 244 vehicles will separate from the detoured vehicles at "34 Ave NW & Sarcee Tr" intersection and will continue through movements on 34 Ave NW. Those 220 (previously left turning) vehicles will leave the detoured vehicles at off-ramp of Sarcee Tr SB to TCH WB and will continue through movements on Sarcee Tr SB. The remaining detoured vehicles will leave Sarcee Tr SB to TCH WB through the off-ramp which connects these two highways.
- Parking on NB of Home Rd between TCH and Bowness Rd is prohibited.
- Left turn from both lanes of Home Rd to Bowness Rd is permitted.
- Parking along 77 St NW and 34 Ave NW in detour path is prohibited. So apart from segment of Bowness Rd between 52 St NW and Bow Crescent NW always 2 lanes will be available for detouring vehicles.
- Left turn from both lanes of Bowness Rd to 77 St NW is permitted.

4.5.2. Travel time, detouring EB traffic via Bowness area (Model 2 - Option 1).

The detour scenario was simulated with the above assumptions. Since this is a hypothetical scenario, there is no field measurement for calibration and the drivers' characteristics based on the normal operating condition model was applied to this scenario as well. The average simulated travel time per vehicle along the detour path for ten 60 minutes simulation was extracted and presented in Table 4.9. Based on these simulations, the average simulated travel time per vehicle on Home Rd NB between TCH and Bowness Rd is 14,098 s (3.92 hr), Table 4.9.

Apparently, the traffic signals that affecting traffic flow on Home Rd including; "TCH & Home Rd", "Bowness Rd & Home Rd" and "Bowness Rd & 49St NW" have been designed for the normal operating condition not the detour condition. They need to be adjusted based on the detour volumes. These adjustments might reduce travel time on Home Rd and its negative effect on the segment of Bowness Rd EB between 49 St NW and Home Rd. Due to overcrowding at Bowness Rd & Home Rd intersection (with actuated signal) and lack of space on Bowness Rd EB, right turning vehicles (from 49 St NB to Bowness Rd EB) could not do the turn and form a tail back which extended to TCH EB.

Table 4.9 Travel Time on Detour Path During Morning Peak, Model 2 - Option 1

Arterial No.	#2	#3	#4	#5	#15
Route on Detour Path	Home Rd NB + Bowness Rd WB	77 St NW SB	34 Ave NW EB	Sarcee Tr SB	Off-ramp Sarcee Tr SB to TCH WB
Travel Time (s/veh/voc)	14,098.0	49.6	72.7	24.6	38.8

As a result, two approaches were followed and their effect, individually and cumulatively, on travel time on Home Rd NB and detour path were studied. The first approach was closing the left turn from TCH EB to 49 St NW (Model 2 - Option 2). In this case, the left turning vehicles (365 vehicles) need to advance on TCH EB up to "TCH & Home Rd" intersection and access to Bowness Rd through Home Rd NB. This needed another adjustment in the traffic volume of the affected intersections which has been detailed in Section 4.5.3. The second approach was optimizing signal timing of the three intersections which affect traffic on Home Rd NB. Since signal optimization is not in the scope of this study, the optimization work was limited to the isolated signal optimization module of synchro software (Sections 4.5.4 and 4.5.5).

4.5.3. Travel time, detouring EB traffic via Bowness area & closing left turn from TCH EB to 49 St NB (Model 2 - Option 2).

As mentioned, the first approach for reducing congestion on Home Rd NB was closing left turn from TCH EB to 49 St NW (Model 2 - Option 2). This will create more green time for currently detouring vehicles on both Home Rd NB and Bowness Rd WB, "Bowness Rd & Home Rd" intersection, particularly if the majority of these left turning vehicles (from TCH EB to 49 St NW) are destined to Bowness Rd EB. In this case, the left turning vehicles from TCH to 49 St NW (365 vehicles) need to advance on TCH up to "TCH & Home Rd" intersection then turn to Home Rd NB in order to access to Bowness Rd. Now the main question is what percentage of these vehicles will turn to either direction of Bowness Rd (EB or WB).

There is no data available for this option; however, the percentage vehicles making right turn or a left turn at "49 St NW & Bowness Rd" intersection is available, (see Table 4.10). In the absence of precise data it is assumed in the case of detouring through Home Rd NB, the same percentages of these 365 vehicles will turn either to WB (left turn) or EB (right turn) of Bowness Rd at "Bowness Rd & Home Rd " intersection. Based on this assumption, 24 of these vehicles will turn to WB of Bowness Rd and 341 of them will turn to EB of Bowness Rd. The EB traveling vehicles (341 veh), in normal circumstances will advance on Bowness Rd EB up to its intersection with Home Rd, then some of them will turn NB of Home Rd, and the rest will continue through movement on Bowness Rd EB. It is not expected any of these vehicles will turn to Home Rd SB as it is faster to do such a trip through the TCH & Home Rd intersection rather than "49 St NW, Bowness Rd, and Home Rd". In order to decide the percentage of left turn and through movements on Bowness Rd EB at "Bowness Rd & Home Rd" intersection a similar process for deciding turning counts of EB & WB of Bowness Rd was applied, (see Table 4.11). In other words, it is assumed the same percentage of turning movements at Bowness Rd EB & Home Rd intersection on normal circumstances applies to these 341 vehicles.

Table 4.10 Estimating Number of Vehicles on WB of TCH Turning to EB & WB of Bowness Rd

Direction	Turning % at 49 St NW & Bowness Rd Intersection	No of TCH WB Vehicles Turns to EB & WB of Bowness Rd
Bowness Rd WB (left turn)	$\frac{29}{29+404} = 6.7\%$	$6.7\% \times 365 = 24$
Bowness Rd EB (right turn)	$\frac{404}{29+404} = 93.3\%$	$93.3\% \times 365 = 341$
Total	100%	365

Table 4.11 Estimated Vehicles Number on TCH WB Destined Home Rd NB & Bowness Rd EB

Destination	% Turning at Home Rd & Bowness Rd EB Intersection	No of TCH WB Vehicles
Bowness Rd EB (through)	$\frac{619}{223+619} = 73.5\%$	$73.5\% \times 341 = 251$
Home Rd NB (left turn)	$\frac{223}{223+619} = 26.5\%$	$26.5\% \times 341 = 90$
Total	100%	341

To sum up, in this scenario, after closure of left turn from TCH to 49 ST NW, all of 365 left turning vehicles will continue on TCH EB, then they will turn to NB of Home Rd. They will travel on Home Rd NB up to Bowness Rd & Home Rd intersection where 24 of them will turn to Bowness Rd WB and 251 of them will turn on to Bowness Rd EB: the remaining (90 veh) will continue through movement on Home Rd NB. Based on these calculations the adjusted traffic volumes on this part of the study cordon are as shown in Figure 4.4.

Traffic flow in the study cordon was simulated with this data and the average simulated travel time per vehicle along the detour path for ten 60 minutes simulations was calculated presented in Table 4.12. Based on these simulation results, the average simulated travel time per vehicle on Home Rd NB between TCH and Bowness Rd is 14,500.4 s (4.03 hr). As it is seen, solely closing left turn from TCH to 49 St NW does not reduce travel time of detouring vehicles. As a result, optimizing signal timing is necessary.

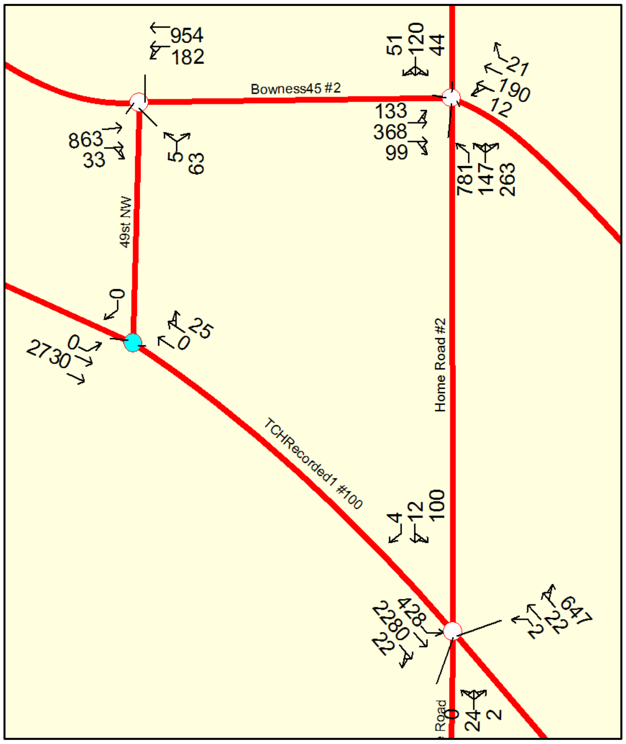

Figure 4.4 Traffic volume after closure of left turn from TCH to 49 St NW (Model 3)

Table 4.12 Travel Time on Detour Path During Morning Peak, Model 3 - Option 2

Arterial No.	#300				#15
Route on Detour Path	Home Rd NB + Bowness Rd WB	77 St NW SB	34 Ave NW EB	Sarcee Tr SB	Off-ramp Sarcee Tr SB to TCH WB
Travel Time (s)	14,500.4	74.5	116.5	24.6	38.8

4.5.4. Travel time, detouring EB traffic via Bowness area & optimizing traffic signals (Model 2- Option 3).

In case of the Model 2 - Option 3, left turn from TCH to 49 St NW is permitted in order to study the sole effect of signal timing optimization on detour travel time within the scope limitation of the research mentioned in Chapter 1, Section 1.5. The signal timing for each signal has been optimized individually _ "TCH & Home Rd" intersection, "Bowness Rd & Home Rd" intersection, "Bowness Rd & 49 St NW" intersection_ and the original and new timings are as given in Table 4.13 and Table 4.14, respectively.

Using the new signal timings, traffic was simulated in the study cordon and the average simulated travel time per vehicle along the detour path for ten 60 minutes simulations was calculated (see Table 4.15). Based on these simulation results, the average simulated travel time per vehicle on Home Rd NB between TCH and Bowness Rd is 9,148 s (2.54 hr). As it is seen, basic signal optimization has reduced travel time on the Home Rd part of the detour path 1.38 hr. which shows the importance of signal optimization on this case.

Table 4.13 Original Signal Timing, Model 2 - Option 3

Intersection	Cycle length (s)	Parameter			
TCH & Home Rd	157.1	Phase			
		Split (s)	96.5	50.5	10.1
		Green max (s)	90	43	7
Bowness Rd & Home Rd	80	Phase			
		Split (s)	49.5	30.5	
		Green max (s)	44	24.5	
Bowness Rd & 49 St NW	80	Phase			
		Split (s)	40.5	26.5	13
		Green max (s)	35	21	10

Table 4.14 Optimized Signal Timing, Model 2 - Option 3

Intersection	Cycle length (s)	Parameter			
TCH & Home Rd	150	Phase			
		Split (s)	89.5	50.5	10
		Green max (s)	83	43	7
Bowness Rd & Home Rd	110	Phase			
		Split (s)	50	60	
		Green max (s)	44.5	54	
Bowness Rd & 49 St NW	150	Phase			
		Split (s)	117.5	24.5	8
		Green max (s)	112	19	5

Table 4.15 Travel Time on Detour Path During Morning Peak, Model 2 - Option 3

Arterial No.	#300				#15
Route on Detour Path	Home Rd NB + Bowness Rd WB	77 St NW SB	34 Ave NW EB	Sarcee Tr SB	Off-ramp Sarcee Tr SB to TCH WB
Travel Time (s)	9,148.0	73.2	111.4	24.5	37.4

4.5.5. Travel time, detouring EB traffic via Bowness area & closing left turn from TCH EB to 49 St NB & optimizing traffic signals (Model 2 - Option 4).

In this case, the simultaneous effect of closing left turn from TCH EB to 49 St NW NB and optimizing the three sets of signals is studied. Majority of these left turning vehicles from TCH EB to 49 St NB are destined to Bowness Rd EB and Home Rd NB, (see Table 4.11). If they have a new route which consumes less green time of the two traffic lights on Bowness Rd (intersections with 49 St NW and Home Rd), then this saved green time could be allocated to the vehicles which are detouring through Bowness Rd WB. With this concept, signal timing for each of the following intersections was individually optimized; “TCH & Home Rd” intersection, “ Bowness Rd & Home Rd” intersection, “ Bowness Rd & 49 St NW” intersection. The new timings are as presented in Table 4.16. The original timing is summarized in Table 4.13.

Table 4.16 Optimized Signal Timing, Model 2- Option 4

Intersection	Cycle length (s)	Parameter			
TCH & Home Rd	120	Phase			
		Split (s)	59.5	50.5	10
		Green max (s)	53	43	7
Bowness Rd & Home Rd	150	Phase			
		Split (s)	50	100	
		Green max (s)	44.5	94	
Bowness Rd & 49 St NW	80	Phase			
		Split (s)	40.5	26.5	13
		Green max (s)	35	21	10

Using the new signal timings, traffic was simulated at the study cordon and the average simulated travel time per vehicle along the detour path for ten 60 minutes simulations was calculated and are presented in table 4.17. Based on these simulation results, the average simulated travel time per vehicle on Home Rd NB between TCH and Bowness Rd is 0.56 hr (2,248.8 s). As shown combination of basic signal optimization and closing left turn from TCH to 49 St NW reduces travel time on Home Rd NB and consequently detour path dramatically compared to the condition without taking any of these measures, 0.62 hr versus 3.92 hr. As a result, Model 2 - Option 4 will be considered as the final detour option at this study.

Table 4.17 Travel Time on Detour Path During Morning Peak, Model 2 - Option 4

Arterial No.	#2	#3	#4	#5	#15
Route on Detour Path	Home Rd NB + Bowness Rd WB	77 St NW SB	34 Ave NW EB	Sarcee Tr SB	Off-ramp Sarcee Tr SB to TCH WB
Travel Time (s)	2,248.8	74	116	24.6	38.3

4.5.6. Analysis result (Model 2 - Option 4).

In final detour scenario (Model 2- Option 4) left turn from TCH EB to 49 St NW is blocked and signal timing of the following intersections is optimized based on the detour volumes; "TCH & Home Rd", "Bowness Rd & Home Rd" and "Bowness Rd & 49 St NW". In this scenario, WB traffic of TCH which have to detour as well as host traffic on WB of the detour path will experience the highest level of delay due to Bow Bridge rehabilitation. These vehicles are referred to as Group1 from this point onward. In addition to Group1, work zone generated delay will incur to traffic on a couple of other routes; however, its amount is negligible compare to the delay experienced by Group1. The following is the list of those routes:

- EB traffic on Bow Bridge during rehabilitating EB of the bridge (extra delay of this traffic due to weaving from EB to WB and WB to EB in order to cross the work zone).
- TCH WB due to change on timing of the signal at "TCH & Home Rd" intersection.

- Increased travel time on SB of Home Rd (upstream of its intersection with Bowness Rd) due to increased number of vehicles using “Bowness Rd & Home Rd” intersection.
- Increased travel time of Home Rd NB (upstream of its intersection with TCH)

Because of a lack of precise traffic data and work zone information, these lesser affected routes are not considered in the user delay calculations. Not mentioning that including them at this point would not affect the final result (choosing the most cost effective work zone set up from UDC (user delay cost) point of view). However, in case of availability of suitable data it is recommended UDC associated with these routes to be included in total cost calculations as well.

It can be assumed that, in the case of detour, attractiveness of Bowness Rd could decrease as drivers cancel or replace their trip through the detour path. Percentage of traffic that might decide to do so is out of the scope of this study; as a result, the effect of these drivers on reducing user delay has not been included in the calculations of neither Model 2 (detour) nor Model 1 and Model 3. Considering all of the above information, the total user delay associated with this work zone is composed of 736.8 (hr/voc) of passenger cars and 25.2 (hr/voc) of truck delay, as summarized in Table 4.18.

Table 4.18 Total Delay of Vehicles at Detour Scenario (Model 2) Due to One Hour of Construction Work on Bow Bridge During Morning Peak

Description		# Route	Construction				Normal				User Delay (hr/ voc)	
			Travel Time (s/voc/veh)	Volume (Veh)	% Trucks	Travel Time × Volume (hr/voc)	Travel Time (s/voc/veh)	Volume (Veh)	% Trucks	Travel Time × Volume (hr/voc)	Cars	Trucks
TCH EB	[Home Rd - 49 St NW]	100	0.0	0	0	0.0	21.6	795	10	4.8	-4.3	-0.5
	[49 st NW- Bow Bridge]	100	0.0	0	0	0.0	53.1	855	7	12.6	-11.7	-0.9
	[Bow Bridge]	11	0.0	0	0	0.0	18.1	855	5	4.3	-4.1	-0.2
	[Bow Bridge - End of acceleration lane]	11	0.0	0	0	0.0	38.9	855	5	9.2	-8.8	-0.5
	[End of acceleration lane - Sarcee Tr NB off ramp]	11	0.0	0	0	0.0	10.9	855	5	2.6	-2.5	-0.1
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	11	0.0	0	0	0.0	9.4	569	5	1.5	-1.4	-0.1
	[Sarcee Tr NB on ramp - Sarcee Tr]	11	0.0	0	0	0.0	6.6	1,112	5	2.0	-1.9	-0.1
	[Sarcee Tr- Sarcee Tr SB on ramp]	11	0.0	0	0	0.0	5.6	1,112	5	1.7	-1.6	-0.1
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	11	0.0	0	0	0.0	10.7	1,112	5	3.3	-3.1	-0.2
Home Rd	NB [TCH - Bowness Rd]	300	1,068.6	1,191	4	353.5	84.4	86	12	2.0	338.9	12.7
Bowness Rd	WB [Home Rd - 49 St]	300	69.6	1,136	5	22.0	26.5	372	9	2.7	18.4	0.8
	WB [49 St NW - 52 St NW]	300	64.9	1,107	5	20.0	31.1	282	11	2.4	16.8	0.7
	WB [52 St NW - Bend Node]	300	4.8	1,097	5	1.5	35.5	442	7	4.4	93.9	2.7
	WB [Bend Node - Bow Crescent]	300	285.2	1,257	3	99.6						
	NB [Bow Crescent - 33 Ave NW]	300	120.7	1,033	4	34.6	28.1	208	9	1.6	31.7	1.3
	NB [33 Ave NW - 60 St NW]	300	14.0	1,052	5	4.1	12.5	228	12	0.8	3.2	0.1
	WB [60 St NW - 63 St NW]	300	65.2	1,076	7	19.5	64.6	674	10	12.1	7.2	0.2
	WB [63 St NW- 73 St NW]	300	234.9	1,133	4	73.9	90	308	8	7.7	63.6	2.6
	WB [73 St NW - 77 St NW]	300	320.0	982	3	87.3	63.6	157	6	2.8	81.6	2.9
77 St NW	SB [Bowness Rd - 34 Ave NW]	300	74.0	1,020	3	21.0	68	195	2	3.7	16.8	0.5
34 Ave NW	EB [77 St NW - 73 St NW]	300	63.8	1,336	3	23.7	55.4	511	2	7.9	15.4	0.5
	EB [73 St NW - Sarcee Tr]	300	52.2	1,183	4	17.2	29.8	358	5	3.0	13.7	0.5
Sarcee Tr	SB [34 Ave NW - TCH WB off ramp]	300	24.6	1,140	4	7.8	23.8	351	5	2.3	5.3	0.2
	SB [TCH WB off ramp- TCH WB On ramp]	6	7.0	561	4	1.1	11.3	341	5	1.1	0.0	0.0
	Sarcee Tr SB off Ramp to TCH WB	15	38.3	579	3	6.2	31.1	10	5	0.1	5.9	0.2
	Sarcee Tr NB on Ramp from TCH WB	16	0.0	0	0	0.0	62.7	244	5	4.2	-4.0	-0.2
	NB [- TCH WB on ramp - TCH WB off ramp]	6	6.9	142	5	0.3	7.4	166	5	0.3	-0.1	0.0
	NB [- TCH WB off ramp - 34 Ave NW]	5	15.1	315	5	1.3	15.4	339	5	1.5	-0.1	0.0
49 St NW	NB	20	45.0	68	2	0.9	39.2	433	2	4.7	-3.8	-0.1
	SB	20	0.0	0	0	0.0	22.1	85	2	0.5	-0.5	0.0
Bowness	EB [49 St NW - Home Rd]	300	67.4	600	3	11.2	29.2	941	3	7.6	3.5	0.1
TCH	EB [49 St NW - Home Rd]	100	139.4	2,730	3	105.7	52.7	2,365	3	34.6	69.0	2.1
Total			2,781.6			912.1	1059.3			150.1	**736.8**	**25.2**

4.6. Closing One Direction & Diverting Traffic To Opposing Direction (Model 3)

In this hypothetical scenario it is assumed that WB is completely closed for rehabilitation purposes and only EB of the bridge is available for traveling through the work zone. A traffic light is installed in either side of the work zone which divides the green time between the opposing traffic. In other words, when the signal is green for EB traffic, the WB traffic will queue well ahead of the work zone and let the EB traffic to cross the work zone. There will be enough all red time for the EB vehicles to clear this shared segment of TCH before the signal turns green for the traffic on WB. When the signal turns green for the WB traffic, they will cross the work zone through the EB lanes while the opposing traffic (EB) is queuing at west side of the work zone. Again there will be enough all red time for the WB traffic to clear the common segment of TCH. This cycle will be repeated through the rehabilitation of WB. After finishing the rehabilitation of WB lanes, the traffic will be diverted to these lanes and repair work on EB of the work zone will start. The same traffic light concept will be applied again.

4.6.1. Signal timing.

In this case a signal is installed in either side of the work zone and operates as interconnected (clustered) signals: when one is green the other will be red (see Figure 4.5). Critical lane method is used to calculate minimum cycle length and phase split of the signals with the input data has been shown in Table 4.19. As shown the total critical volume is almost at the range of the saturation flow rate. Even assuming v/ c = 1 and PHF=1, the minimum signal timing is 2’629 s (43.8 minutes), see Table 4.21, which is not viable signal timing. As a result, sensitivity of signal cycle length to the percentage of the total critical volume was done (the results are shown in Table 4.21) and it can be seen that going to shorter cycle lengths will lead to more waste of green time as well as longer vehicles queue and waiting time in order to cross the work zone.

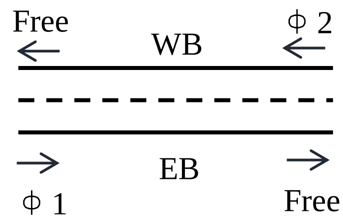

Figure 4.5 Phasing plan of traffic lights at the work zone (Model 3)

Table 4.19 Input Data for Calculating Morning Peak Signal Timing (Model 3)

Parameter	Value	Source
Critical Volume EB	1,365 veh/ h/ lane	The City of Calgary
Critical Volume WB	428 veh/ h/ lane	The City of Calgary
Total Critical Volume	1,793 veh/ h/ lane	Row1 + Row 2 of Table 4.19
S (saturation flow rate)	1,850 veh/ h/ lane	The City of Calgary
PHF	1	The City of Calgary
V/C	1	Assumption
Speed	50 km/h	Assumption
W (work zone length)	240 m	200 m Bridge & 20 m Weaving in each direction
Yellow	3.5 s	Refer to note "*"
All Red	37 s	Refer to note "**"

Note*

Change interval (yellow time) was calculated using ITE methodology (Ross et al., 2011). In these calculations 85 percentile speed of approaching vehicles (S_{85}) is not known; as a result, sensitivity of change interval to S_{85} was performed, Table 4.20. Other assumptions for calculating change interval are as the following.

- Drivers' perception time: 1 s
- Grade of approach: 0%
- Deceleration rate of vehicles: 3.048 m/s^2 (10 ft/s^2)

Table 4.20 Sensitivity of Change Interval to S_{85} (Model 3)

S_{85} (km/h)	45	50	55	60	65	70	75	80
Yellow (s)	3.1	3.3	3.5	3.7	4.0	4.2	4.4	4.6

Although the FFS before the work zone is 80 Km/h in both directions and considering the congestion before the work zone during the morning peak hour and all standard sign posts before a work zone, it is expected approaching vehicles to have speeds close to the work zone speed limit (50 Km/h) rather than FFS (80 Km/h). Additionally, based on the primary simulation results the average approaching speed of vehicles on TCH before the work zone is 38 km/h and 26 Km/h for WB and EB, respectively. Considering theses information as well as change interval of "TCH & Home

Rd" intersection (3.5 s with FFS = 50 km/h on both EB and WB approaches), the yellow time equal to 3.5 s has been chosen for signals at the work zone.

Note**

All red time was calculated using ITE methodology, as well (Ross et al., 2011). In this case the work zone has been modeled as a wide intersection (240 m) then the time required for a vehicle to clear the intersection has been calculated. The bridge width based on the google map measurements is around 200 m, with considering 20 m from each side as the minimum weaving distance the total work zone length (intersection width) will be 240 m. ITE uses 15 percentile speed of approaching vehicles (S_{15}) for calculating all red time. This speed is not known; however, based on the primary simulation results the average speed at the work zone is 24 Km/h and 25 Km/h on EB and WB, respectively. So, the all red time was calculated using 24 Km/h speed in lieu of S_{15}. Based on these calculations required all red time is 36.9 s. All red time equal to 37s was used in the simulation software. The simulation result shows that this amount of all red time allows all vehicles to clear the work zone immediately before the opposing traffic enters the work zone.

Table 4.21 Morning Peak Signal Timing (Model 3)

% Total Critical Volume	C_{min} (s)	$Split_{WB}$ (s)	$Split_{EB}$ (s)	G_{WB} (s)	G_{EB} (s)
100%	2,629 (43.8')	648.5	1980.5	608	1,940
95%	1,022 (17.0')	265.5	756.5	225	716
90%	634 (10.6')	172.5	461.5	132	421
85%	460 (7.7')	130.5	329.5	90	289
80%	**361 (6.0')**	**107.5**	**253.5**	**67**	**213**
75%	297 (5.0')	92.5	204.5	52	164

Looking at Table 4.21 cycle lengths associated with 75% and 80% of the total critical volume (TCV) are more practical. In these cases 73% and 78% of the cycle length, respectively will be used by the traffic to cross the work zone and the rest will be wasted on yellow and all red. There is no considerable difference between these two cases; however, the waiting time for WB traffic in case of 75% TCV is 0.8 minutes shorter than 80% TCV (4.1' vs. 4.9'). The waiting times for EB traffic in both cases are close (2.2 minutes versus 2.5 minutes respectively for 75% TCV and 80% TCV). In this study the simulation is continued with cycle length equal 361 s, Figure 4.6.

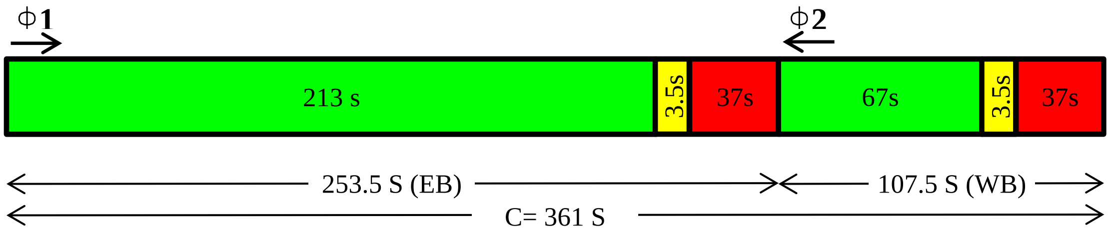

Figure 4.6 Signal Timing at Work Zone Based on Critical Lane Volume Method (Model 3)

4.6.2. Closure of left turn from TCH to 49 St NW and vice versa (Model 3).

Based on the results of the simulation, left turning vehicles from TCH EB to 49 St NW interrupted TCH WB traffic flow created aggravating queue on this direction which was long enough to block "TCH & Home Rd" intersection. So in this scenario left turn from TCH EB to 49 St NW NB is blocked and these vehicles need to do this turning movement through Home Rd NB. Based on this route change, adjustment on the traffic volume of "TCH & Home Rd" intersection, " Bowness Rd & Home Rd" intersection, "49 St NW & Bowness Rd" and " TCH & 49 St NW" intersections were carried out, (see Figure 4.7). The details are similar to section 4.5.3.

Due to traffic light control on TCH WB shortly before the work zone, vehicles queue up to cross the work zone and the lineup blocks right turns from 49 St NW to TCH WB which consequently causes a long queue on 49 St NW SB.

- This queue in turn prevents left turn from Bowness Rd WB to 49 St NW SB.
- Queue on Bowness Rd WB blocks left turn from Home Rd NB to Bowness RD WB.
- Queue on Home Rd NB prevents left turn from TCH EB to Home Rd NB. This in turn causes long queue on TCH EB which blocks shared lanes on the work zone.

To prevent this domino effect, right turn from 49 St NW to TCH WB is blocked and these right turning vehicles are required to detour through Bowness Rd EB, Home Rd SB to TCH WB, Figure 4.7.

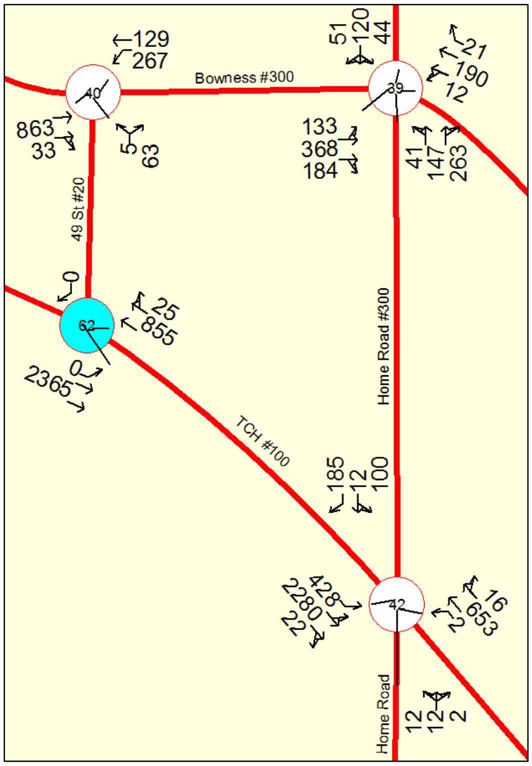

Figure 4.7 Traffic volume after closure of left turn from TCH to 49 St NW and right turn from 49 St NW to TCH (Model 3)

Using the above detouring and signal timing, traffic on the study cordon was simulated. Based on the average result of ten, 60 minutes simulations delay on TCH WB between Home Rd and work zone is 1,497.3 s /veh/voc [1,571.9 (WZ) -74.6 (normal)] and delay on TCH EB between off ramp to Sarcee Tr SB and work zone is 72.9 s /veh/voc [174.5 (WZ) -101.6 (normal)]. This results in 410.9 (hr/voc) delay on TCH (between Home Rd & Sarcee Tr SB) due to one hour of repair work in morning peak. After these changes, TCH EB works smoothly, however, queue on TCH WB still spills beyond “TCH & Home Rd” intersection. This queue causes some of the TCH WB traveling vehicles to wait for two cycle lengths of the traffic light and cross this intersection during the third cycle.

Using shorter traffic light cycles at the work zone, increases the cumulative delay of vehicles due to the work zone; on the other hands, increasing the cycle lengths of the traffic lights of the work zone will increase the queue length and will exacerbate the blockage of “TCH & Home Rd” intersection. It seems the main problem relays on dividing the green time between TCH EB and TCH WB traffic. So, seven different signal splits with the same cycle length (361 s) and the same yellow + all red (3.5 s + 37 s) were simulated, (see Table 4.22 and Figure 4.8). Based on the simulation results (average of ten 60 minutes run) , the signal split with 103.5 s of WB green time and 176.5 s of EB green time resulted in the lowest delay along TCH between Home Rd and off ramp to Sarcee Tr SB and the rest of the calculations are used this signal timing (see Figure 4.9).

Table 4.22 Sensitivity of Total Delay on TCH to Signal Split, Average of 10 Runs (Model 3)

Cycle Length (s)	G_{WB} (s)	G_{EB} (s)	Split $_{WB}$ (s)	Split $_{EB}$ (s)	TCH Total Delay (hr/ voc)	WB Queue Spilled Beyond Home Rd
361	140.0	140.0	180.5	180.5	669.5	No
	125.0	155.0	165.5	195.5	573.0	No
	115.0	165.0	155.5	205.5	547.6	**No**
	103.5	**176.5**	**144.0**	**217.0**	**271.1**	**No**
	97.0	183.0	137.5	223.5	283.5	No
	92.0	188.0	1323.5	228.5	310.3	No
	67	213	107.5	253.5	410.9	41% Time

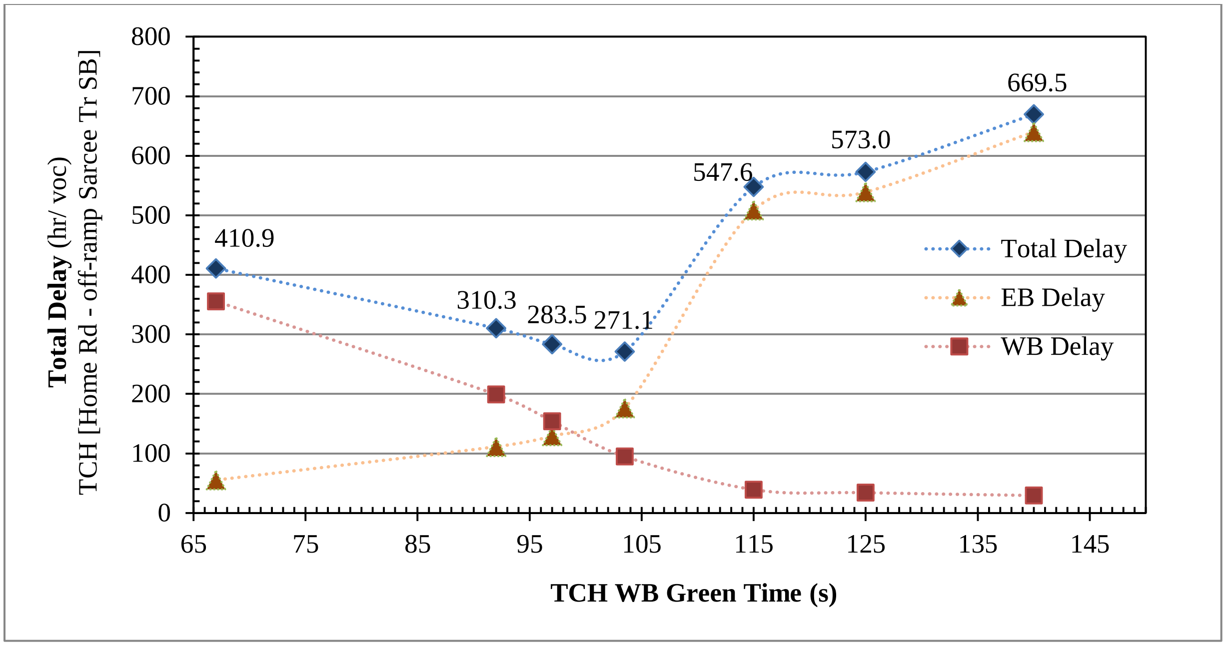

Figure 4.8 Sensitivity of total delay on TCH to signal split, average of 10 runs (Model 3)

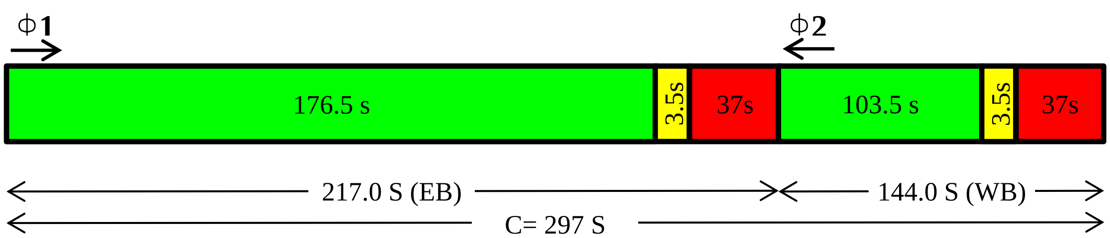

Figure 4.9 Final signal timing at work zone (Model 3)

4.6.3. Analysis result (Model 3)

In this hypothetical scenario (Model 3), TCH WB is closed for rehabilitation purposes at Bow Bridge work zone. The open lanes on EB of the work zone are shared with the opposing traffic (WB traffic). Interconnected traffic signals are set up at either side of the work zone in order to share the open lanes between the EB and WB traffic at the work zone. At this study minimum required signal cycle length of the traffic lights was calculated using critical lanes volume (TCV) method. Since the total critical lanes volume is close to saturation flow rate, this calculation resulted in 43.8 minutes of minimum cycle length which is not a viable cycle length. Consequently, sensitivity of the cycle length to the percentage of TCV was investigated, (see Table 4.21). Cycle length associated with 80% of the TCV was selected for the work zone (361 s).

Due to the blockage of "TCH & Home Rd" intersection in case of using the cycle split calculated based on the TCV method, sensitivity analysis of total delay along TCH to signal split with considering cycle length equal to 361s and keeping "yellow + all red" constant (3.5 s + 37 s) was performed, (see Figure 4.8). Total delay associated with signal split equals to 144 s and 217 s for WB and EB, respectively resulted in the minimum total delay along the segment of TCH between Home Rd and off ramp to Sarcee Tr SB. Considering this signal timing, Figure 4.9, the total delay on the study cordon was calculated, Table 4.23. In this scenario the following routes are the most affected routes by setting up the construction zone.

- TCH EB & WB between Home Rd and Sarcee Tr
- Home Rd NB & SB between TCH and Bowness Rd
- Bowness Rd EB & WB between 49 St NW and Home Rd
- 49 St NB & SB

There might be slight changes in travel time of other neighbouring routes which are not included in these calculations due to their minor effects. Additionally, during the construction period attractiveness of TCH at this segment might be reduced because of the congestion associated with the work zone activities. This might discourage some drivers from using this route. As mentioned in subsection 4.5.6, this is not in the scope of this study so this effect has not been included in user delay calculations. Based on the aforementioned information, the total user delay associated with this work zone composed of 288.1 (hr/voc) of cars delay and 16.8 (hr/voc) of trucks delay, (see Table 4.23).

Table 4.23 Total Delay of Vehicles at Closing One Bound Scenario (Model 3) Due to One Hour of Construction Work on Bow Bridge During Morning Peak

Description		# Route	Construction				Normal				User Delay (hr/ voc)	
			Travel Time (s/veh/voc)	Volume (veh)	% Trucks	Travel Time× Volume (hr/voc)	Travel Time (s/veh/voc)	Volume (veh)	% Trucks	Travel Time × Volume (hr/voc)	Cars	Trucks
TCH EB	[Home Rd - 49 St NW]	100	100.1	2,730	3	75.9	52.7	2,365	3	34.6	40.0	1.2
	[49 st NW- Bow Bridge]	100	78.9	2,730	2	59.8	80.6	2,730	2	61.1	-1.3	0.0
	[Bow Bridge]	103	26.9	2,730	5	20.4	19.1	2,730	5	14.5	5.6	0.3
	[Bow Bridge - End of acceleration lane]	101	217.7	2,730	5	165.1	38.2	2,730	5	29.0	129.3	6.8
	[End of acceleration lane - Sarcee Tr NB off ramp]	101	39.7	2,730	5	30.1	12.3	2,730	5	9.3	19.7	1.0
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	101	23.9	2,289	5	15.2	9.3	2,289	5	5.9	8.8	0.5
	[Sarcee Tr NB on ramp - Sarcee Tr]	101	9.3	2,302	5	5.9	6.5	2,302	5	4.2	1.7	0.1
	[Sarcee Tr- Sarcee Tr SB on ramp]	101	5.7	2,302	5	3.6	5.2	2,302	5	3.3	0.3	0.0
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	101	11.0	2,184	5	6.7	10.9	2,184	5	6.6	0.1	0.0
TCH WB	[Home Rd - 49 St NW]	100	27.3	880	10	6.7	21.6	795	10	4.8	1.7	0.2
	[49 st NW- Bow Bridge]	100	422.3	855	7	100.3	53.1	855	7	12.6	81.5	6.1
	[Bow Bridge]	103	24.3	855	5	5.8	18.1	855	5	4.3	1.4	0.1
	[Bow Bridge - End of acceleration lane]	101	41.2	855	5	9.8	38.9	855	5	9.2	0.5	0.0
	[End of acceleration lane - Sarcee Tr NB off ramp]	101	11.2	817	5	2.5	10.9	817	5	2.5	0.1	0.0
	[Sarcee Tr NB off ramp - Sarcee Tr NB on ramp]	101	9.6	569	5	1.5	9.4	569	5	1.5	0.0	0.0
	[Sarcee Tr NB on ramp - Sarcee Tr]	101	6.8	1,112	5	2.1	6.6	1,112	5	2.0	0.1	0.0
	[Sarcee Tr- Sarcee Tr SB on ramp]	101	5.6	1,112	5	1.7	5.6	1,112	5	1.7	0.0	0.0
	[Sarcee Tr SB on ramp- Sarcee Tr SB off ramp]	101	10.8	1,112	5	3.3	10.7	1,112	5	3.3	0.0	0.0
Home Rd	NB [TCH - Bowness Rd]	300	44.7	451	12	5.6	84.4	81	12	1.9	3.3	0.4
	SB [TCH - Bowness Rd]	300	40.8	297	7	3.4	33.4	212	7	2.0	1.3	0.1
Bowness Rd	EB [Home Rd - 49 St]	300	24.5	685	3	4.7	29.2	941	3	7.6	-2.9	-0.1
	WB [Home Rd - 49 St]	300	32.9	396	9	3.6	26.5	372	9	2.7	0.8	0.1
49 St NW	NB	20	59.2	68	2	1.1	39.2	433	2	4.7	-3.5	-0.1
	SB	20	0.0	0	2	0.0	22.1	85	2	0.5	-0.5	0.0
Total											**288.1**	**16.8**

4.7. Comparing User Delay Associated With Different Work Zone Configurations

In this case, three different work zone configurations were simulated using SimTraffic software and user delay associated with either configuration was calculated. The simulated work zone configurations are as the followings:

Model 1: Closing one lane in each direction (work zone in 2003)

Model 2: Complete detour of EB traffic via nearby major street (Bowness neighborhood)

Model 3: Closing one direction and diverting traffic to the opposing direction

(traffic light control, cycle length= 361 s, green WB= 103.5 s, yellow= 3.5 s, all red= 37 s)

Based on these calculations Model 1 (closing one lane in each direction) results in minimum user delay. The total user delay in this case is 186.6 (hr/ voc) of cars delay and 10.1 (hr/ voc) of trucks delay, (see Table 4.24). Model 3 (closing one direction and sharing the open direction with the opposing traffic) stands on the second place from minimum user delay point of view, 288.1 (hr/ voc) of passenger cars delay and 16.8 (hr/ voc) of trucks delay. Model 2 (detouring from Bowness Rd) causes the highest user delay, 736.8 (hr/ voc) and 25.2 (hr/ voc) of user delay for cars and trucks, respectively.

Table 4.24 User Delay Cost Associated With Different Work Zone Configurations

Model	Work Zone Configuration	User Delay (hr/ voc)	
		Cars	Trucks
1	Closing One Lane in Each Direction	186.6	10.1
2	Detour From Bowness Rd	736.8	25.2
3	Closing One Direction [Traffic Light: Cycle Length= 361s, Green WB= 103.5 s]	288.1	16.8

Using the findings of this chapter (user delay associated with different work zone configuration), the application of UDC in selecting optimum work zone configuration will be demonstrated in Chapter 5 of this thesis using a tool developed using Monte-Carlo simulation.

Chapter Five: Economic Analysis

5.1. Introduction

In this chapter an economic tool for monetizing user delay due to construction work on major urban arterials will be presented. Although the focus of the study is urban arterials, the developed tool is applicable to any sort of road way construction ranging from major urban arterial bridges to rural highways. The application of the tool will be demonstrated through two case studies: the case of the rehabilitation of Crowchild Bridge in 2002 will be used to calculate hourly user delay cost (UDC) and the case of the rehabilitation of Bow Bridge on TCH in 2003 will demonstrate how this information can be used to select the optimum work zone layout.

The following is the order of the contents of this chapter: Section 5.2 includes definition of UDC and its components, details discussion of FHWA's (Federal Highways Administration) deterministic approach for calculating UDC, and probabilistic approach suggested in this research for calculating UDC and its justification; Section 5.3 presents the input data and why that specific data has been selected for UDC quantification; Section 5.4 presents the monetary value of user delay associated with Crowchild Bridge calculated using the developed tool; in Section 5.5, the application of the tool for selecting optimum work zone layout is presented; and finally the chapter's findings are summarized in Section 5.6.

5.2. Monetizing User Delay

As with products and services, time spend travelling in a vehicle has economic value. The monetary value of travel time is based on the idea the time spent travelling could be used for either a paid activity or recreation (Mallela & Sadavisam, 2011). According to The United States

Department of Transportation (USDOT) Office of the Secretary of Transportation (OST), the value of travel time is composed of the following five components (USDOT, 2003):

- Monetary value of personal travel time with passenger cars (PPC)
- Monetary value of business travel time with passenger cars (BPC)
- Monetary value of truck drivers travel time with truck (TD)
- Cost of freight inventory delay carried by trucks (F)
- Cost of vehicle depreciation including all vehicles (DP)

FHWA in its 2011 report (FHWA-HOP-12-005) presents a detailed deterministic methodology to calculate the monetary value of user delay based on the above said OST breakdown of the value of travel time (Mallela & Sadavisam, 2011) as described in section 5.2.1. However, there is inherent variability with the majority of the input data used in FHWA methodology; ranging from work zone incurred delay time to the unit cost of the travel time components. As a result, this research will incorporate the probabilistic nature of these components into FHWA methodology (see Table 5.1) in order to provide decision makers with not only the expected value of UDC (FHWA Deterministic Approach) but also with the probable range of UDC values associated with the work zone as well as their corresponding level of certainty/risk.

5.2.1. FHWA deterministic approach versus probabilistic approach of the research.

5.2.1.1. Monetary value of personal travel time with passenger cars (PPC).

This component is composed of monetary value of personal local travel time with passenger car (PPCL) plus the monetary value of personal intercity travel time with passenger car (PPCI), equation 5.1 (Mallela & Sadavisam, 2011). According to this source PPCL and PPCI are calculated using equations 5.2 and 5.3.

Table 5.1 List of Input Data, Source, and the Difference with FHWA Recommendation

Input		Research Source & Input Type		FHWA Recommendation & Input Type	
Vehicle Occupancy	voc_{PPCL}	The City of Calgary (downtown mode split and vehicle occupancy methodology)	Fix value	NHTS & NPTS (national household transportation survey & nationwide personal transportation survey)	Fix value
	voc_{PPCL}				
	voc_{BPCL}				
	voc_{BPCI}				
	voc_{Truck}	The City of Calgary (2001 external truck survey)	Uniform distribution	FHWA (highway economic requirements system-state version, technical report)	Fix value
% of Income/ Compensation as Value of Travel Time	c_{PPCL}	FHWA (recommendation based on the USDOT (OST), valuation of travel time in economic analysis-revised departmental guidance, memorandum)	Uniform distribution	FHWA (Recommendation based on the USDOT (OST), valuation of travel time in economic analysis-revised departmental guidance, memorandum)	Fix value
	c_{PPCI}				
	c_{BPCL}				
	c_{BPCI}				
% Local Travel	Personal	Statistics Canada (Canadian travel survey, domestic travel, Alberta wide)	Fix value	NHTS & NPTS (national household transportation survey & nationwide personal transportation survey)	Fix value
	Business				
Income	Income	Statistic Canada (Calgary data)	Custom distribution	Median nationwide household income	Fix value
	Population	Statistics Canada (Calgary data)	Fix value	N/A	N/A
Wages & Benefits	Business Travel	Statistics Canada (Alberta data)	Custom distribution	Bureau of Labor Statistics (Nationwide hourly median)	Fix value
	Truck Drivers	Statistics Canada (Alberta data)	Custom distribution	Bureau of Labor Statistics (Nationwide hourly median)	Fix value
Freight	$V_{Commodity}$	USDOT (bureau of transportation, statistics transborder freight data, Alberta, Saskatchewan, British Columbia)	Uniform distribution	FHWA (Office of Freight Management and Operations)	Fix value
	$Pay\ load_i$	FHWA (FHWA-PL-00-029, Vol I)	Fix value		
	% truck type	The City of Calgary (2001 external truck survey)	Fix value		
	% loaded trucks				
Monetary	CPI	Statistics Canada	Fix value	U.S. Bureau of Labor Statistics	Fix value
	GDP Deflator	Statistics Canada	Fix value	Us Bureau of Economic Analysis	Fix value
	Prime Rate	Bank of Canada	Fix value	Federal Reserve	Fix value

$$PPC = PPCL + PPCI \tag{5.1}$$

$$PPCL = \frac{delay_{\ PC} \times c_{PPC} \times voc_{PPCL} \times \%\ local\ travel \times median\ household\ income\ \times c_{PPCL}}{2080} \tag{5.2}$$

$$PPCI = \frac{delay_{\ PC} \times c_{PPC} \times voc_{PPCI} \times \%\ intercity\ travel \times median\ household\ income \times c_{PPCI}}{2080} \tag{5.3}$$

Where:

- PPC	*Monetary value of personal travel time with passenger car*
- $PPCL$	*Monetary value of personal local travel time with passenger car*
- $PPCI$	*Monetary value of personal intercity travel time with passenger car*
- $Delay_{PC}$	*Total delay of passenger cars due to the work zone*
- c_{PPC}	*% of personal travels in passenger car*
- voc_{PPCL}	*Vehicle occupancy of passenger cars in local personal travel*
- voc_{PPCI}	*Vehicle occupancy of passenger cars in Intercity personal travel*
- $\%\ local\ travel$	*% of local personal travels with passenger car*
- $\%\ intercity\ travel$	*% of intercity personal travels with passenger car (1- % local travel)*
- c_{PPCL}	*% of median household income considered as the value of personal local travel time (35% to 60%) with 50% as a recommended value for deterministic calculations*
- c_{PPCI}	*% of median household income considered as the value of personal intercity travel time (60% to 90%) with 70% as a recommended value for deterministic calculations*
- *2080*	*Working hours per year (40 working hours per week and 52 weeks per year)*

In reality all components of PPC have some degree of variability particularly the household income and the percentage of a household income that the person treats as the value of her/ his personal travel time (C_{PPCL} & C_{PPCI}). In this research, effect of variability of these three parameters on the reliability of the UDC will be evaluated using Calgarians' income distribution instead of the national median household income and the value of C_{PPCL} & C_{PPCI} will be treated with uniform distribution. These parameters are presented in section 5.3.1. However, the tool has the capability of treating all parameters probabilistic in case their probability distribution is available.

5.2.1.2. Monetary value of business travel time with passenger cars (BPC).

This component is composed of monetary value of local business travel time with passenger car (BPCL) plus the monetary value of intercity business travel time with passenger car (BPCI), equation 5.4 (Mallela & Sadavisam, 2011). According to this source BPCL and BPCI are calculated using equations 5.5 and 5.6.

$$BPC = BPCL + BPCI \qquad 5.4$$

$$BPCL = delay_{\,PC} \times c_{BPC} \times voc_{BPCL} \times \%\, local\ travel \times total\ componsation\ cost/hr \times c_{BPCL} \qquad 5.5$$

$$BPCI = delay_{\,PC} \times c_{BPC} \times voc_{BPCI} \times \%\, intercity\ travel \times total\ componsation\ cost/hr \times c_{BPCI} \qquad 5.6$$

Where:

- BPC	*Monetary value of business travel time with passenger car*
- $BPCL$	*Monetary value of business local travel time with passenger car*
- $BPCI$	*Monetary value of business intercity travel time with passenger car*
- $Delay_{PC}$	*Total delay of passenger cars due to the work zone*
- c_{BPC}	*% of business travels in passenger cars (1- c_{PPC})*
- voc_{BPCL}	*Vehicle occupancy of passenger cars in local business travel*
- voc_{BPCI}	*Vehicle occupancy of passenger cars in Intercity business travel*
- $\%\ local\ travel$	*% of local business travels with passenger car*
- $\%\ intercity\ travel$	*% of intercity business travels with passenger car (1- % local travel)*
- c_{BPCL}	*% of total compensation cost per hour considered as the value of personal local travel time (80% to 120%) with 100% as a recommended value for deterministic calculations*
- c_{BPCI}	*% of compensation cost per hour considered as the value of personal intercity travel time (80% to 120%) with 100% as a recommended value for deterministic calculations*

Similar to the PPC, all components of BPC have some degree of variability particularly the compensation cost per hour which widely varies based on the employees' characteristics and the employer type. Percentage of employee's compensation cost to the employer which should be considered as the value of business travel time is another random variable (C_{BPCL} & C_{BPCI}), which in deterministic approach is estimated with a value equal to 100%. In this research effect of variability of these three parameters on the reliability of UDC will be evaluated. So, province wide compensation cost distribution (the most detailed available data) will be used instead of the nationwide median total compensation cost per hour (which is a fix number) and the value of C_{BPCL} & C_{BPCI} will be considered in the range of 80% to 120% with uniform distribution. These parameters have been detailed in section 5.3.2.

5.2.1.3. Monetary value of truck drivers travel time with truck (TD).

FHWA calculate the monetary value of truck drivers' travel delay equal to the median hourly wages and benefits of truck drivers multiplied by truck occupancy multiplied by trucks' total delay, equation 5.7 (Mallela & Sadavisam, 2011).

$$TD = Truck\ drivers'\ median\ hourly\ wages\ \&\ benefits \times\ trucks'\ occupancy \times trucks'\ total\ delay \quad 5.7$$

All elements of this equation are treated as a fix value; however, truck drivers' average hourly wages and benefits varies depending on the truck type (single unit versus semi-trucks) and the nature of the shipping company (couriers versus ordinary shippers). Thus, in this study probability distribution of truck drivers' average hourly wages and benefits will be used for quantifying their travel delay cost. Additionally, trucks' occupancy rate based on the truck type are available so it will be treated as a probabilistic parameter rather than a fix value in equation 5.7 (Section 5.3.3).

5.2.1.4. Cost of freight inventory delay (F).

When a valuable cargo is delayed while in transit, the owner is charged with interest carrying cost. In order words, if the owner of that cargo had a cash instead of the cargo, he / she would have been benefited a market return on the cash (AASHTO, 2010). Cost of the freight inventory delay carried by trucks is estimated equal to the market return on the cash equivalent of the delayed cargo during the delay period. This is calculated using equation 5.8 (Mallela & Sadavisam, 2011).

$$F = \sum V_{Commodity} \times Pay\ load_i \times \%\ of\ truck_{\ i} \times \%\ of\ loaded\ truck_i \times prime\ rate / 8760 \qquad 5.8$$

Where:

- F	*Cost of freight inventory delay*
- $V_{Commodity}$	*Average value of commodities shipped by truck* ($\frac{\$}{Kg}$)*, usually a nationwide average*
- i	*{Single unit truck, Semi- truck, etc.}*
- $Pay\ load_i$	*Pay load of truck type i*
- $\%\ of\ truck_{\ i}$	*% of truck type i in the truck fleet*
- $\%\ of\ loaded\ truck_i$	*% of truck type i fully or partially loaded*
- $prime\ rate$	*The average annual prime rate at the study year*
- 8760	*Number of hours in year (to change annual interest rate to hourly interest rate)*

Value of commodities carried by trucks is a probabilistic variable, which in the presence of reliable data, could be presented with a probability distribution.

5.2.1.5. Cost of vehicle depreciation including all vehicles (DP).

Vehicle depreciation is a function of aging and usage overtime, total depreciation composed of mileage depreciation (usage component) and time depreciation (aging). The time depreciation of vehicles due to the work zone delay is part of the depreciation cost that is included in calculating UDC incurred by a work zone setup.

The total depreciation is estimated from the average annual ownership cost of vehicles. Time depreciation is calculated by subtracting mileage depreciation from the total depreciation and mileage depreciation is estimated from the methods related to the vehicle operation cost (VOC) calculations (Mallela & Sadavisam, 2011) which is not in the scope of this research. As a result, any further discussion on depreciation component of UDC will be avoided.

5.2.1.6. Developed tool and Crystal Ball software.

The tool was developed using Crystal Ball an Excel base adds in. Crystal Ball is a graphically oriented forecasting and risk analysis program that takes the uncertainty out of decision-making using Monte-Carlo simulation, it forecasts the entire range of results possible for a given situation and calculates the confidence levels, so the user could know the likelihood of any specific event occurrence (EPM Information Development Team, 2016).

Using Crystal Ball, the developed tool draws random values for input data (Section 5.3) from user defined probability distributions, creates new scenarios and calculates UDC associated with each scenario. In this study 10,000 random scenarios are created for each of the models (Crowchild Bridge and three different work zone models of Bow Bridge). Using these 10,000 scenarios, probability distribution of the UDC is extracted which is a base for risk analysis and calculating expected UDC.

5.3. Input Data

One of the main challenges of this research was collecting reliable input data for each parameter. The challenges with the traffic data have been discussed in Chapter 3 and Chapter 4. The following sections provide detailed explanation of the input data used in monetizing UDC, the challenges faced in each case, and why that specific data has been used in the analysis. Extensive research was conducted using publically available data from internet sources (Statistic Canada website, Alberta transportation website, the City of Calgary website, USDOT website, FHWA website, website of freight companies, etc.). In addition, Statistics Canada and the City of Calgary were contacted to confirm the interpretation of the data.

5.3.1. Value of personal travel.

The hourly value of personal travel time with passenger car is estimated as percentage of the annual income divided by 2080 (Mallela, J. and Sadavisam, S., 2011, USDOT, 2003), assuming a full time employee will work 40 hours per week and 52 weeks a year. The recommended percentages for local and intercity travels are as Table 5.2.

Table 5.2 FHWA Recommended Percentages to Calculate Value of Personal Travel Time (Mallela, J. and Sadavisam, S., 2011, USDOT, 2003)

Travel Type	Per Person-Hour as a Percent of Income
Local	35% - 60%
Intercity	60% - 90%

In this study, Calgarians' income in 2002 and 2003 was derived from Table 111-0008 of Statistics Canada (Neighborhood income and demographics, taxfilers and dependents with income by total income, sex and age group, annual) (Government of Canada, Statistics Canada, 2016). This table cumulates the numbers upward; in other words, people with income X dollars and over, so the number of people in each income bracket was calculated by subtracting frequency of people in that bracket with that of the following bracket. This table excludes people with no income; however, they are part of the road users so their value of travel time needs to be included in the calculations. To include them, the total number of people with income (721,580 people in 2002 & 736,980 people in 2003) was subtracted from the population of Calgary in the corresponding year (1,007,510 people in 2002 & 1,029,552 people in 2003, according to CANSIM Table 051-0056 (Government of Canada, Statistics Canada, 2016)) in order to calculate the number of people with no income (285,930 people in 2002 & 292,572 people in 2003), as shown in Table 5.3.

Table 5.3 Income Distribution in Calgary, 2002 & 2003, Derived from CANSIM Table 111-0008 (Government of Canada, Statistics Canada, 2016)

Income Range ($)	No. of People in 2002	No. of People in 2003
With no income	285,930	285,930
With income less than $5000	70,840	70,840
$5000 to less than $10,000	66,270	66,270
$10,000 to less than $15,000	73,730	73,730
$15,000 to less than $20,000	67,720	67,720
$20,000 to less than $25,000	54,530	54,530
$25,000 to less than $35,000	102,080	102,080
$35,000 to less than $50,000	111,930	111,930
$50,000 to less than $75,000	93,940	93,940
$75,000 to less than $100,000	36,500	36,500
$100,000 to less than $150,000	24,840	24,840
$150,000 to less than $200,000	8,020	8,020
$200,000 to less than $250,000	3,530	3,530
$250,000 and more	7,650	7,650
Total	1,007,510	1,029,552

Since there is no data of the percentage of local and intercity personal travels with passenger cars, it is assumed that the percentage of income as a personal travel time value is varying uniformly between 35% and 90% for both type of the trips.

5.3.2. Value of business travel time.

The hourly value of business travel time with passenger car is estimated as 80% to 120% of the hourly employer cost for employee compensation (wages and benefits) both for local and intercity travels (Mallela, J. and Sadavisam, S., 2011, USDOT, 2003). The available data on employer cost for employee compensation are quarterly reports aggregated in Canada level not in city or provincial level (Available at CANSIM Table 380-0074, produced from National Gross Domestic Product by Income and by Expenditure Accounts - 1901 Survey (Government of

Canada, Statistics Canada, 2016)). This data source, lacks the number of employees by industry sector which was confirmed by personal communication with Statistics Canada. In its absence the wages and benefits data produced from the Labour Force Survey - 3701, (CANSIM Table 282-0151 & CANSIM Table 282-0141(Government of Canada, Statistics Canada, 2016)) was used as the closest estimate of the employer cost for employee compensation, which results in slightly underestimation of the UDC.

There are ten categories of National Occupations Classifications in Canada (NOC [0] to NOC [9]) and each category is comprised of more detailed subcategories, Table 5.4. CANSIM Table 282-0151 presents the average hourly wages of employees by type of work (NOC), unadjusted for seasonality, aggregated in monthly bases and provincial level. As well, CANSIM Table 282-0141 presents the monthly average number of employees at the corresponding NOC, unadjusted for seasonality aggregated at the provincial level. With combining data from both tables and using the equation 5.9 and 5.10, the average hourly wages and the average number of corresponding employees at each NOC subcategory in 2002 and 2003 have been calculated, Table 5.4. This table excludes NOC [7] (Trades, transport and equipment operators and related occupations) category, since occupations under this category relates to truck operation occupations which will be covered in truck travel delay cost. There is no data of the percentage of local and intercity business travels with passenger cars. In the lack of data in order to have a fair estimate, it is assumed there is uniform distribution of business travel destination.

$$average\ hourly\ wage(\$)_{NOC_{xy}} = \frac{\sum_{1}^{12}\left(monthly\ hourly\ wages_{NOC_{xy}} \times monthly\ employees'\ no._{NOC_{xy}}\right)}{\sum_{1}^{12}(\ monthly\ employees'\ no.)} \quad 5.9$$

$$average\ monthly\ employees'\ no._{NOC_{xy}} = \frac{\sum_{1}^{12}\left(monthly\ employees'\ no._{NOC_{xy}}\right)}{12} \quad 5.10$$

Table 5.4 Hourly Wages & Benefits in Alberta Versus Average Number of Employees, Excluding "Trades, Transport And Equipment Operators And Related Occupations" (NOC 7) in 2002 & 2003, Derived from CANSIM Table 282-0151 & Table 282-0141 (Government of Canada, Statistics Canada, 2016)

National Occupational Classification (NOC)	Average Hourly Wage ($) in 2002	Average No. Of Employee in 2002	Average Hourly Wage ($) in 2003	Average No. Of Employee in 2003
Senior management occupations [00]	38.51	8,725	36.96	9,992
Specialized middle management occupations [01-05]	31.17	38,983	29.74	42,042
Middle management occupations in retail and wholesale trade and customer services [06]	20.85	68,842	19.19	62,892
Middle management occupations in trades, transportation, production and utilities [07-09]	31.56	75,758	30.33	74,258
Professional occupations in business and finance [11]	25.27	51,367	25.29	52,125
Administrative and financial supervisors and administrative occupations [12]	16.83	72,533	17.86	73,425
Finance, insurance and related business administrative occupations [13]	15.71	25,475	16.80	26,450
Office support occupations [14]	14.47	91,008	15.18	97,625
Distribution, tracking and scheduling co-ordination occupations [15]	13.67	34,800	15.29	38,358
Professional occupations in natural and applied sciences [21]	30.15	65,758	30.66	58,300
Technical occupations related to natural and applied sciences [22]	22.50	63,392	22.78	61,433
Professional occupations in nursing [30]	26.94	23,775	28.16	22,050
Professional occupations in health (except nursing) [31]	23.99	16,692	27.68	17,033
Technical occupations in health [32]	19.20	23,008	19.66	26,708
Assisting occupations in support of health services [34]	14.76	21,317	14.76	20,217
Professional occupations in education services [40]	25.06	63,258	25.95	56,833
Professional occupations in law and social, community and government services [41]	22.84	34,025	25.01	30,992
Paraprofessional occupations in legal, social, community and education services [42]	13.46	29,308	13.43	33,425
Occupations in front-line public protection services [43]	24.90	8,325	24.59	8,367
Care providers and educational, legal and public protection support occupations [44]	13.09	24,342	13.20	27,300
Professional occupations in art and culture [51]	19.66	12,667	19.61	15,467
Technical occupations in art, culture, recreation and sport [52]	14.39	19,067	14.41	25,200
Retail sales supervisors and specialized sales occupations [62]	18.76	49,233	19.26	51,800
Service supervisors and specialized service occupations [63]	11.13	57,183	11.47	63,642
Sales representatives and salespersons - wholesale and retail trade [64]	12.49	76,150	12.55	77,225
Service representatives and other customer and personal services occupations [65]	11.56	74,883	12.02	70,483
Sales support occupations [66]	8.43	48,642	8.64	57,175
Service support and other service occupations, n.e.c. [67]	9.69	82,392	9.76	83,033
Supervisors and technical occupations in natural resources, agriculture and related production [82]	23.12	25,992	23.00	29,050
Workers in natural resources, agriculture and related production [84]	14.88	22,608	15.84	24,367
Harvesting, landscaping and natural resources labourers [86]	13.79	8,808	14.75	12,358
Processing, manufacturing and utilities supervisors and central control operators [92]	24.31	17,842	24.83	22,258
Processing and manufacturing machine operators and related production workers [94]	15.08	28,767	15.64	29,017
Assemblers in manufacturing [95]	13.52	12,392	14.61	12,033
Labourers in processing, manufacturing and utilities [96]	11.76	12,225	12.95	13,400

5.3.3. Value of truck drivers' travel time.

The hourly value of truck drivers' travel time is considered equal to the cost of their compensation to employer, in other words, their wages and benefits (Mallela, J. and Sadavisam, S., 2011, USDOT, 2003). In Canada, occupations under NOC [7] category use truck as part of their businesses. So, travel time value of truck drivers is estimated base on the wages and benefits of this category of occupations. For the same reason mentioned in section 5.3.2, the hourly value of truck drivers' wages and benefits was derived from Labour Force Survey - 3701, (CANSIM Table 282-0151 & CANSIM Table 282-0141). These values are available in Table 5.5 for 2002 and 2003.

Table 5.5 Hourly Wages & Benefits in Alberta Versus Average Number of Employees, NOC 7 (Trades, Transport And Equipment Operators And Related Occupations) in 2002 & 2003 (Derived from CANSIM Table 282-0151 & Table 282-0141)

National Occupational Classification (NOC)	Average Hourly Wage ($) in 2002	Average No. of Employee in 2002	Average Hourly Wage ($) in 2003	Average No. of Employee in 2003
Industrial, electrical and construction trades [72]	20.11	113,583	20.19	117,092
Maintenance and equipment operation trades [73]	20.73	58,408	21.43	65,267
Other installers, repairers and servicers and material handlers [74]	13.75	21,967	14.26	21,908
Transport and heavy equipment operation and related maintenance occupations [75]	16.53	72,950	17.29	77,767
Trades helpers, construction labourers and related occupations [76]	13.65	17,292	14.26	20,608

5.3.4. Vehicle occupancy.

Vehicle occupancy is the number of people traveling in a vehicle and it is usually counted by observers stationed along the roadside using a count board. It includes cars and trucks but excludes transit and bicycles and is calculated by adding the number of passengers to the number of drivers and dividing by the number of drivers (The City of Calgary, 2012).

5.3.4.1. Vehicle occupancy (passenger cars).

According to the City of Calgary Mobility report, since 2002 inbound vehicle occupancy in morning peak to downtown Calgary has leveled off at 1.2 people per vehicle (The City of Calgary, 2012). Figure 5.1 which is taken from this reference shows the change in vehicle occupancy at downtown Calgary since 1977. However, according to Figure 25 of The City of Calgary report titled “Changing Travel Behaviour in the Calgary Region “, the average 24 hour weekday vehicle occupancy between 1971 and 2011 has increased from 1.28 to 1.39 persons per vehicle with 1.37 people per vehicle in 2001(The City of Calgary, 2014). In order to avoid over estimating UDC, in this study the reported value by the City of Calgary mobility report (1.2 people per vehicle) will be used as the vehicle occupancy of the passenger cars.

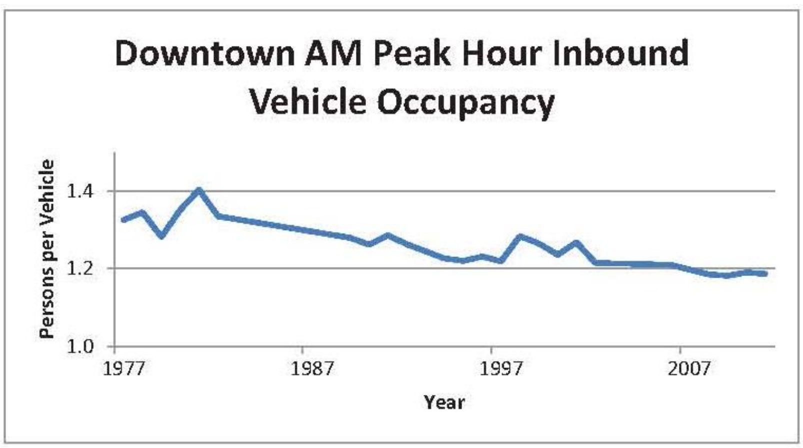

Figure 5.1 Downtown historical vehicle occupancy (The City of Calgary, 2012)

It is worth mentioning, there is no detail data on the vehicle occupancy of local versus intercity travels, as well as personal versus business travels. As a result, in this study vehicle occupancy for all types of trips with passenger cars will be considered 1.2 people per vehicle.

5.3.4.2. Trucks occupancy.

Based on Table 7 of "2001 External Truck Survey" Report, truck occupancy varies from 1.05 to 1.19 depending on the truck type, Table 5.6 (The City of Calgary, 2001). In this study, truck occupancy has been treated as a uniform distribution in the range of 1.05 to 1.19.

Table 5.6 Truck Occupancy

Truck Type	Occupants per Vehicle
Single Axle, Single Unit	1.18
Dual Axle, Single Unit	1.19
Semi, No Trailer	1.05
Semi, One Trailer	1.11

5.3.5. Cost of freight inventory delay.

The hourly cost of delay for freight is calculated by multiplying average payload of trucks with the average hourly interest value of commodities shipped by trucks. The following is the detail description of source of these data, their precision and obstacles faced in acquiring them. Statistics Canada staff provided comment and guidance given the dearth of actual data for this parameter. Additional resources were accessed at Alberta Transportation regarding the truck classification, percentage of loaded trucks and trucks' payload but no data about the value of cargo carried by trucks is available by neither Alberta Transportation nor Statistics Canada although it is part of the Statistic Canada's annual truck survey questionnaire. To fill the gap, internet research, email communications, phone conversations, and meeting with stakeholders were held and the following values were derived for the use in the model.

5.3.5.1. Truck classification and percentage of loaded trucks.

According to Alberta Transportation, FHWA class 9 and class 10 trucks are mainly used for transporting goods in Alberta. Additionally, on average 90% of these trucks are loaded in 2014 based on the data recorded at 6 WIM (Weight In Motion) sites around the province located at Edson, Fort Macleod, Leduc VIS, Red Dear, Leduc, Villeneuve (Government of Alberta Ministry of Transportation, 2014).

According to the City of Calgary "2001 External Truck Survey Report" 26.1% of the trucks in Calgary's highways are single unit trucks and 73.9% are semi-trucks and an estimated 95.3% of single unit trucks and 84.1% of semi-trucks are fully or partially loaded (The City of Calgary, 2001). However, this reference doesn't provide any data about the average tare weight and payload of the single unit and semi-trucks.

In comparison, information from "Calgary 2001 External Truck Survey Report " both geographically and time wise is better representative of Calgary's truck composition and percentage of loaded trucks in 2002 and 2003 than "2014 Weigh in Motion Report" of Alberta Transportation (Government of Alberta Ministry of Transportation, 2014). Thus, in this research, the data from "Calgary 2001 External Truck Survey Report" will be used in calculating cost of freight inventory delay, Table 5.7.

It is acknowledged that inner city truck composition and percentage of loaded trucks are different than the percentages in this report. However, with considering the lower participation of freight inventory delay cost in UDC in urban setting and the fact that FHWA allows to use nationwide percentages for these parameters (Mallela & Sadavisam, 2011), in the lack of inner city truck data “Calgary 2001 External Truck Survey Report” information are treated as an appropriate estimate of the Calgary’s truck composition and the percentage of loaded trucks in 2002 and 2003.

Table 5.7 Truck Composition and Percentage of Loaded Trucks

Truck Type	Truck Composition	% Loaded
Single Unit	26.1 %	95.3
Semi-truck	73.9 %	84.1

5.3.5.2. Truck payload.

Pay load of a truck is that part of its load that generates revenue. As discussed there are no clear statistics of average payload of trucks in Calgary; however, average ESAL (Equivalent Single Axle Loading) data for all lanes at all WIM sites in Alberta from the years 2010 to 2014 is available (Government of Alberta Ministry of Transportation, 2014). Although these data are representative of Alberta, they do not give clear information about payload of the truck fleet in Alberta. As a result, the data from FHWA regarding the average gross weight, tare (empty) weight, and payload of the trucks are used for the purpose of this study, Table 5.8 (FHWA, 2000).

Table 5.8 Characteristics of Typical Vehicles and Their Common Weight (FHWA, 2000)

Vehicle Configuration	Gross Weight (Kg)	Empty Weight (Kg)	Payload Weight (Kg)
Three-Axle Single Unit Truck	**24,494**	**10,251**	**14,243**
Four-Axle Single Unit Truck	29,030	11,975	17,055
	32,205	11,975	20,230
Five-Axle Semitrailer	**36,287**	**13,835**	**22,453**
Six-Axle Semitrailer	40,823	14,288	26,535
	43,998	14,288	29,710

As it is mentioned in section 5.3.5.1 five axle and six axle semitrailer trucks are mainly used for carrying cargos in the province (Government of Alberta Ministry of Transportation, 2014). Thus, in this study in order to demonstrate the methodology for calculating freight delay cost without overestimation, the payload of three-axle single unit truck (14,243 Kg) as the representative of the pay load of single unit trucks and the payload of five-axle semitrailer truck (22,453 Kg) as the representative of the payload of semi-trucks are considered. This independent selection agrees with the recommended representative truck types (three axles single unit and five axles semitrailer truck) by FHWA, report No. FHWA-HOP-12-005 in the lack of precise data (Mallela & Sadavisam, 2011). The recommended average payload by this reference for three axle single unit trucks and five axle semitrailer trucks are 11,340 Kg and 19,051 Kg, respectively. There is around 3,000 Kg discrepancy between the payloads of the same type of trucks based on two publications from the same organization (FHWA). However, with considering the fact that average allowed gross truck weight in Canada is higher than US (FHWA, 2000) and interstate highways allowable payload in US could be different than the allowable payload of inner state roads which could affect the recommended average payload by FHWA report No. FHWA-HOP-12-005, this study will use 14,243Kg and 22,453 Kg as the representative of single unit and semi-trucks payload, respectively.

5.3.5.3. Average value of commodities shipped by trucks.

FHWA Office of Freight Management Operations provides updated average dollar value of commodities shipped by trucks in the US. FHWA recommends the average value of commodities shipped nationwide in US equal to 1.43 USD/ lb in 1993 dollars which could be transferred to the current dollars using implicit price deflator (GDP deflator) (Mallela &

Sadavisam, 2011). As it is mentioned at section 5.3.5, Statistics Canada does not provide such information and 2001 Truck survey report does not have information on this regard as well.

According to 2002-2012 Transportation & Trade Report, Alberta is a province which heavily relays on international export, for instance in 2002 value of international export was $52.21 versus $11.18 international import (Government of Alberta Ministry of Transportation, 2014). Based on this reference from 2002 to 2012, US was the top destination for Alberta's exports with 6.5% annual increase and Mexico has continuously been ranked as the fourth or fifth largest destination for Alberta's exports during this period (2002-2012). By value, the United States was the top origin for Alberta's imports from 2002 to 2012 and Mexico was ranked as the second largest import origin for Alberta, from 2002 to 2004. Additionally, part of British Columbia's and Saskatchewan's road export to US is transited through Alberta's boarder ports and vice versa. However, the amount of Alberta's export to the US which was transited through North Portal, Saskatchewan in 2002 and 2003 was way above its road export through Kingsgate, British Columbia (Government of Alberta Ministry of Transportation, 2014).

So, in the absence of precise data, in order to have a close estimate of the average value of commodities carried by trucks around the province, this value is estimated by the "average dollar value per Kg" of commodities exported from Alberta, British Columbia, and Saskatchewan to the US by trucks in 2002 and 2003, as shown in Table 5.9. These data were extracted from U.S. Department of Transportation, Bureau of Transportation Statistics, TransBorder Freight Data website (Transborder.bts.gov, 2016). These data consider the origin of product as the province of export not the province of port of entry to the US. For instance, if the product has been produced in Alberta but has been exported to the US through a border port in, British Columbia, it will be categorized as export from Alberta to the US. The values provided by this website are in US dollars, using the historical exchange rate which were extracted from CanadaianForex website (Canadianforex.ca, 2016), the average value of exported commodities were converted to Canadian dollars, Table 5.9.

Table 5.9 Ratio of “Export Value” to “Export Weight by Truck” from Canada to US

Year	Exchange Rate (USD/ CAD)	Origin	Average Export Value (USD/Kg)	Average Export Value (CAD/Kg)
2002	0.636723	Canada	1.78	2.80
		AB	1.16	**1.82**
		BC	0.93	**1.46**
		SK	0.71	**1.12**
2003	0.718459	Canada	1.81	2.52
		AB	1.21	**1.68**
		BC	0.95	**1.32**
		SK	0.67	**0.93**

As it is seen the value per Kg of exports from Alberta is higher than British Columbia and that is higher than Saskatchewan. It is assumed the value per Kg of commodities carried in province roads follows a uniform distribution with maximum value equal to Alberta’s value per Kg of exports and the minimum value equal to the average value per Kg of exports of British Columbia and Saskatchewan. Which yields the following ranges (1.29 – 1.82 CAD/ Kg) and (1.13 – 1.68 CAD/ Kg) in 2002 and 2003, respectively. It is acknowledged that the value per Kg of commodities in the study cordons to some extent might be different than this range. However, in the absence of the precise Calgary specific data this is the most reliable available estimate. Adding the fact that FHWA allows using nationwide aggregated average value per Kg for this parameter.

5.3.6. Implicit price deflator (GDP deflator).

GDP (Gross Domestic Product) is the total market value of all final goods and service produced inside national boarders in a specific time period (usually one year; however, it could be calculated in quarterly bases as well). GDP is a broad measure of a country's overall economic activity.

Implicit price deflator (GDP deflator) is a measure of inflation and is calculated by dividing nominal GDP (GDP calculated in current dollars) to real GDP (GDP calculated in base year dollars) (Investopedia, 2003). In this way, it will be clarified what percentage of increase in GDP is due to the inflation and what percentage is associated to real growth at the economy. In Canada GDP deflator is calculated by Statistics Canada in quarterly reports and the annual index could be calculated by averaging the quarterly values. The base year for this research is 2007, so the GDP deflator at this year equals 100. The GDP deflators used in this research are available in Table 5.10 (Government of Canada, Statistics Canada, 2016).

Table 5.10 Annual Implicit Price Deflator, Derived from CANSIM Table 380-0066 (Government of Canada, Statistics Canada, 2016)

Year	2002	2003	2007	2016
Annual Implicit Price Deflator	85.7	88.5	100	112.2

5.3.7. Consumer price index (CPI).

Statistics Canada defines CPI as: "An indicator of the changes in consumer prices experienced by Canadians. It is obtained by comparing, through time, the cost of a fixed basket of commodities purchased by Canadian consumers in a particular year." This basket composed of eight categories of items including: food, shelter, household operations and furnishings, clothing and footwear, transportation, health and personal care, recreation, education and reading, alcoholic beverages and tobacco products. (Government of Canada, Statistics Canada, 1996). The price of the CPI basket in the base year is assigned a value of 100.0 and the prices in other years are expressed as percentages of the price in the base year (Australian Bureau of Statistics, 2016). For example, if the price of the basket in 2016 had increased by 28.03% since the base year, then the index would read 128.03. According to CANSIM Table 326-0022, the current base period for the CPI in Canada is 2002 (Government of Canada, Statistics Canada, 2016). CPI is released in monthly bases by Statistics Canada. The annual CPI could be calculated by averaging monthly CPI in the year of interest. Bank of Canada also releases CPI with excluding 8 of the most volatile components of CPI (Government of Canada, Statistics Canada, 2016). The difference in CPI from both sources in target years of this study (2002, 2003, and 2016) are negligible. Consequently, in order to keep the source of input data consistent, CPI calculated by Statistics Canada will be used in this study, Table 5.11. At the time this research is undergone CPI data in 2016 are available until July 2016, so annual CPI in 2016 has been estimated by averaging monthly CPI from January 2016 to July 2016.

Table 5.11 Annual and Monthly Consumer Price Index (CPI), Derived from CANSIM Table 326-0022 (Government of Canada, Statistics Canada, 2016)

Year	Jan	Feb	March	April	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
2002	98.1	98.4	98.8	99.5	99.4	99.6	100.3	100.7	100.8	101.3	101.6	101.5	100.00
2003	102.4	102.9	103	102.3	102.1	102.3	102.5	102.8	103	102.9	103.1	103.6	102.74
2016	127.6	127.4	127.7	128	128.3	128.6	128.6	----	----	----	----	----	128.03

5.3.8. Average prime rate.

By definition, prime rate is the rate that is suggested by the Bank of Canada on which most commercial banks charge their credit worthy customers (The Mortgage Group, 2000 & Investopedia, 2003). Banks set their prime rate based on the cost of short-term funds rate, and on competitive pressures among them. The Bank of Canada influences the cost of short-term funds by setting the target for the overnight rate, the interest rate at which major financial institutions borrow or lend one day (overnight) funds among themselves (Bank of Canada, 2016). The historical prime rates from Bank of Canada archive in 2002 and 2003 are as table 5.12 (Bank of Canada, 2016). The average of these monthly rates has been treated as the annual prime rate on effect in 2002 and 2003 for this study.

Table 5.12 Prime Rate (%) According to The Bank of Canada Website (Bank of Canada, 2015)

Year	Jan	Feb	March	April	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
2002	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.75	4.25	4.25	4.67
2003	4.25	4.25	4.25	4.25	4.25	4.25	4.00	4.00	4.00	4.00	4.00	4.00	4.125

5.3.9. Summary of input data.

Input data used in monetizing UDC associated with Crowchild Bridge and Bow Bridge work zones as well as their URL have been summarized in Table 5.13.

Table 5.13 Numeric Value of Input Data Used In Monetizing UDC Associated With Crowchild and Bow Bridge Work Zones as Well as Their URL

Input			*CrowChild Bridge*	*Bow Bridge*	URL
Vehicle Occupancy	voc_{PPCL}		*1.2*	*1.2*	https://www.calgary.ca/transportation/tp/documents/data/2012/mobility_monitor_march_2012.pdf?noredirect=1
	voc_{PPCL}		*1.2*	*1.2*	
	voc_{BPCL}		*1.2*	*1.2*	
	voc_{BPCI}		*1.2*	*1.2*	
	voc_{Truck}		*min:1.05 ave:1.12 max:1.19 (Uniform distribution)*	*min:1.05 ave:1.12 max:1.19 (Uniform distribution)*	https://www.calgary.ca/Transportation/TP/Documents/forecasting/truck_survey.pdf?noredirect=1
% of Income/ Compensation as Value of Travel Time	c_{PPCL}		*min:35% ave:62.5% max:90% (Uniform distribution)*	*min:35% ave:62.5% max:90% (Uniform distribution)*	http://www.ops.fhwa.dot.gov/wz/resources/publications/fhwahop12005/fhwahop12005.pdf
	c_{PPCI}				
	c_{BPCL}		*min:80% ave:100% max:120% (Uniform distribution)*	*min:80% ave:100% max:120% (Uniform distribution)*	
	c_{BPCI}				
	Truck drivers		*100*	*100*	
% Travel type (Passenger Car)	*Personal*		*81.2*	*81.2*	http://www.statcan.gc.ca/pub/87-212-x/87-212-x2002000-eng.pdf
	Business		*18.8*	*18.8*	
% Local Travel	*Personal*		*100*	*100*	*N/A*
	Business		*100*	*100*	
Income	*Income ($/ annum)*		*min:0 ave:28,847 max:500,000 (Custom distribution)*	*min:0 ave: 29,407 max:500,000 (Custom distribution)*	http://www5.statcan.gc.ca/cansim/a26?lang=eng&id=1110008
	Population		*1,007,510*	*1,029,552*	http://www5.statcan.gc.ca/cansim/a26?lang=eng&id=510056
Wages & Benefits	*Business travel ($/hr)*		*min:8.43 ave: 18.66 max:38.51 (Custom distribution)*	*min: 8.64 ave: 18.72 max: 36.96 (Custom distribution)*	http://www5.statcan.gc.ca/cansim/pick-choisir?lang=eng&p2=33&id=2820151
	Truck drivers ($/hr)		*min: 13.65 ave: 18.43 max: 20.73 (Custom distribution)*	*min: 14.26 ave:18.88 max: 21.43 (Custom distribution*	http://www5.statcan.gc.ca/cansim/pick-choisir?lang=eng&p2=33&id=2820141
Freight	$V_{Commodity}$ ($/Kg)		*min: 1.29 ave: 1.56 max: 1.82 (Uniform distribution)*	*min: 1.13 ave:1.41 max: 1.68 (Uniform distribution)*	https://transborder.bts.gov/programs/international/transborder/TBDR_VWR.html
	Payload (Kg)	*Single unit*	*14,243*	*14,243*	https://www.fhwa.dot.gov/policy/otps/truck/finalreport.cfm
		Semi-truck	*22,453*	*22,453*	https://www.fhwa.dot.gov/policy/otps/truck/finalreport.cfm
	Truck composition	*Single unit*	*26.1%*	*26.1%*	https://www.calgary.ca/Transportation/TP/Documents/forecasting/truck_survey.pdf?noredirect=1
		Semi-truck	*73.9%*	*73.9%*	
	Loaded trucks	*Single unit*	*95.3%*	*95.3%*	
		Semi-truck	*84.1%*	*84.1%*	
Monetary	*CPI*		*100*	*102.74*	http://www5.statcan.gc.ca/cansim/a26?lang=eng&id=3260022
	GDP deflator		*85.73*	*88.5*	http://www5.statcan.gc.ca/cansim/a26?lang=eng&id=3800066
	Prime rate		*4.67%*	*4.13%*	http://www.bankofcanada.ca/wp-content/uploads/2010/09/selected_historical_v122148.pdf

5.4. Monetary Value of User Delay Cost Due to Rehabilitation of Crowchild Bridge

Using the developed tool and input data detailed in section 5.3, distribution of the monetary value of UDC due to one hour of construction work in the morning peak at Crowchild Bridge in 2002 has been calculated and presented in Figure 5.2. Additionally, the distribution of the other four components of this cost including passenger cars personal travel delay cost, passenger cars business travel delay cost, truck delay cost, and freight delay cost has been presented in Figure 5.3, Figure 5.4, Figure 5.5, and Figure 5.6, respectively. These values are in 2002 dollars. Of interest to the City of Calgary is UDC in 2016 dollars. In order to do this conversion, following the recommendations of FHWA passenger cars and trucks delay cost were transferred to 2016 dollars using CPI and freight delay cost was converted to the current dollars using GDP deflator, equation 5.11 (Mallela, J. and Sadavisam, S., 2011). The result has been presented in Figure 5.7.

Additionally, the expected value, median, standard deviation, minimum, maximum, and the range associated with Figures 5.3 to Figure 5.7 have been presented in Table 5.14. Moreover, UDC percentiles have been shown in Table 5.15 and Figure 5.8.

$$UDC = (PPC + BPC + TD) \times \frac{CPI_{\ 2016}}{CPI_{\ 2002}} + \ (F) \times \frac{GDP\ deflator_{\ 2016}}{GDP\ deflator_{\ 2002}} \qquad 5.11$$

Where:

- UDC — *User delay cost*
- PPC — *Monetary value of personal travels time with passenger car*
- BPC — *Monetary value of business travels time with passenger car*
- TD — *Monetary value of truck drivers' travels time with trucks*
- F — *Cost of freight inventory delay*
- CPI_i — *Consumer price index in year i*
- $GDP\ defelator_{\ i}$ — *Implicit price deflator (GDP deflator) in year i*

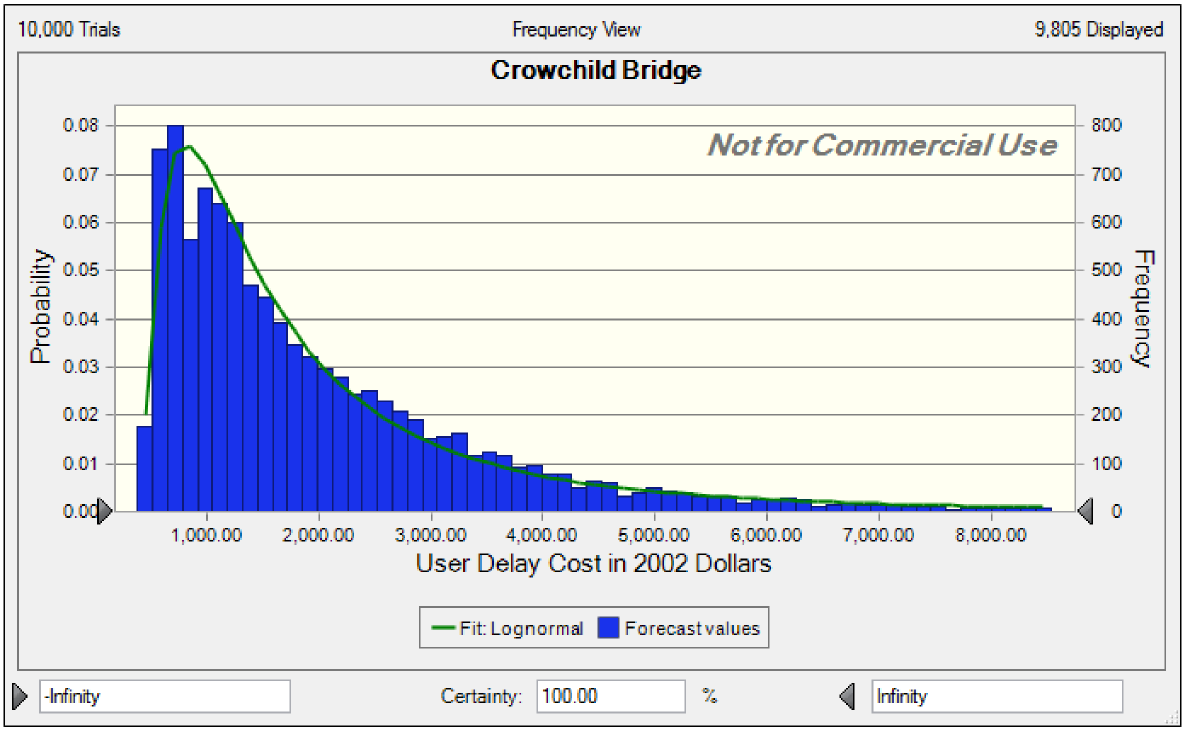

Figure 5.2 User delay cost (UDC) probability distribution in 2002 dollars

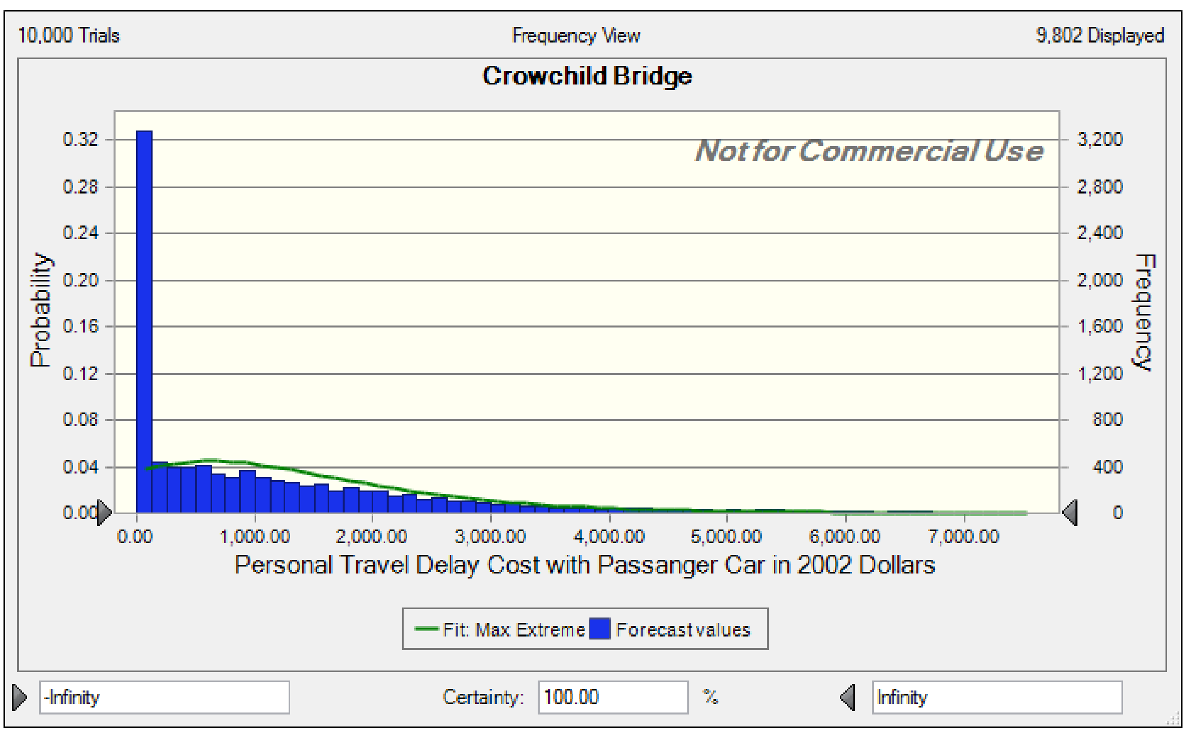

Figure 5.3 Personal travel delay cost in passenger cars (PPC) probability distribution in 2002 dollars

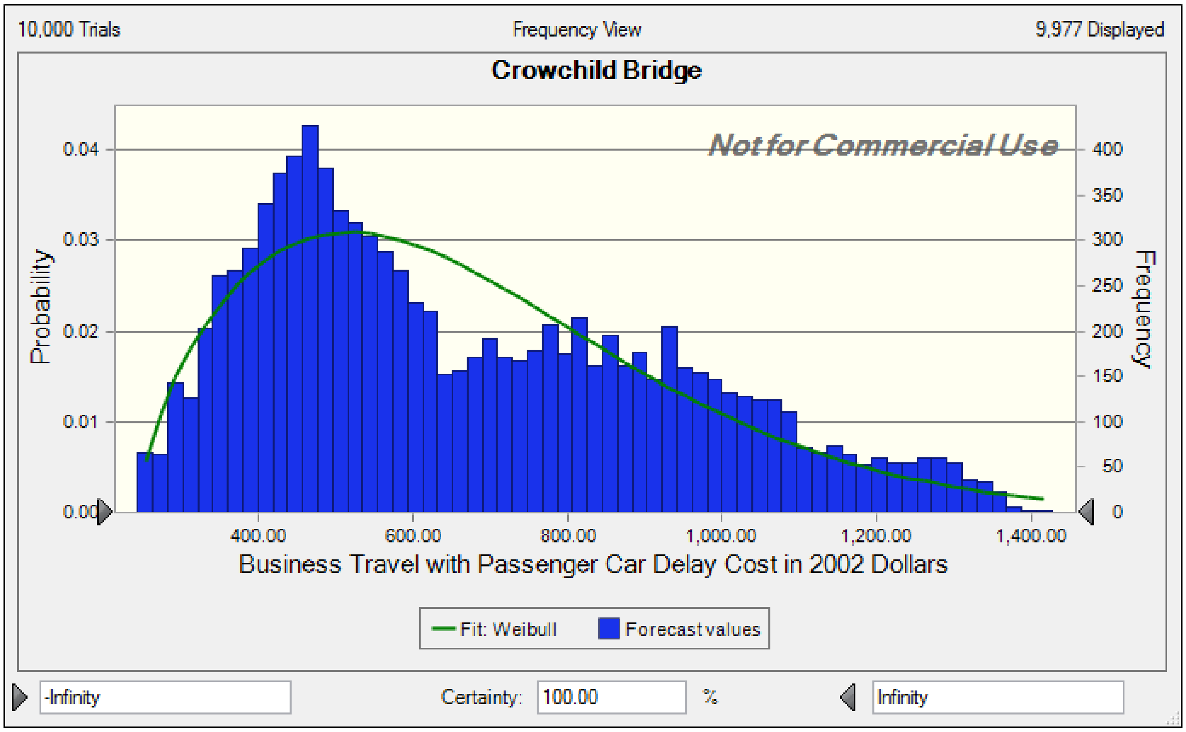

Figure 5.4 Business travel delay cost in passenger cars (BPC) probability distribution in 2002 dollars

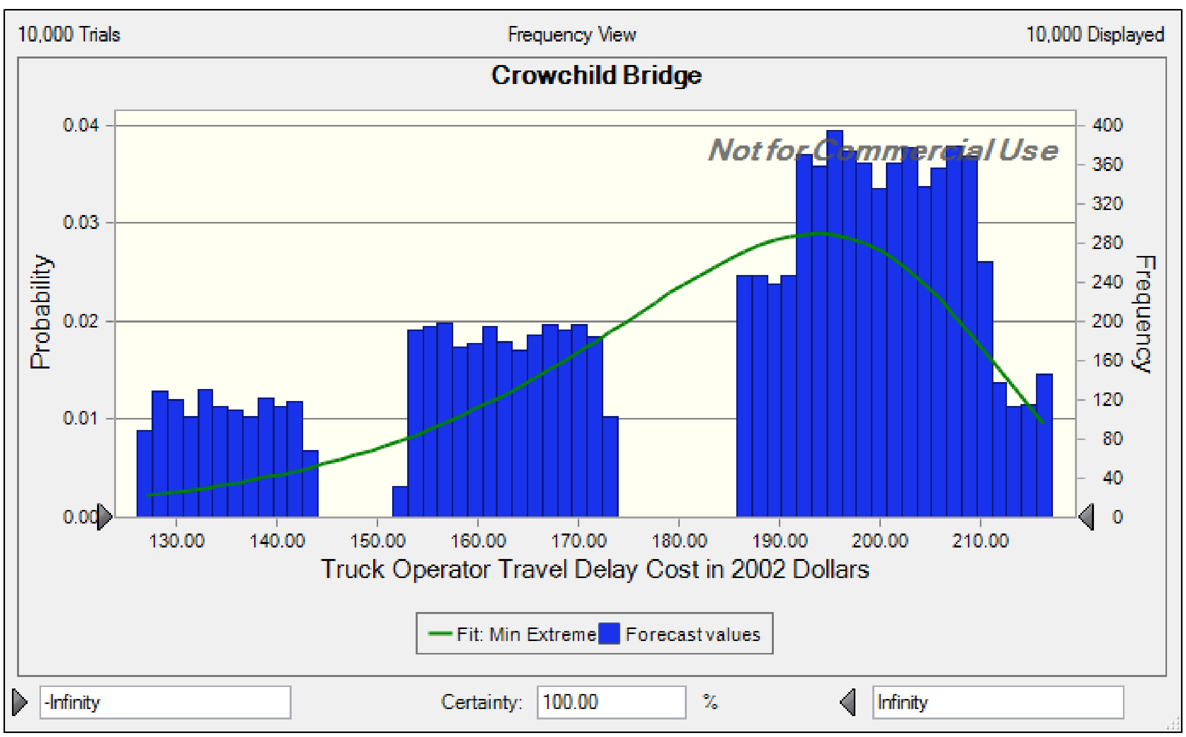

Figure 5.5 Truck operators travel delay cost (TD) probability distribution in 2002 dollars

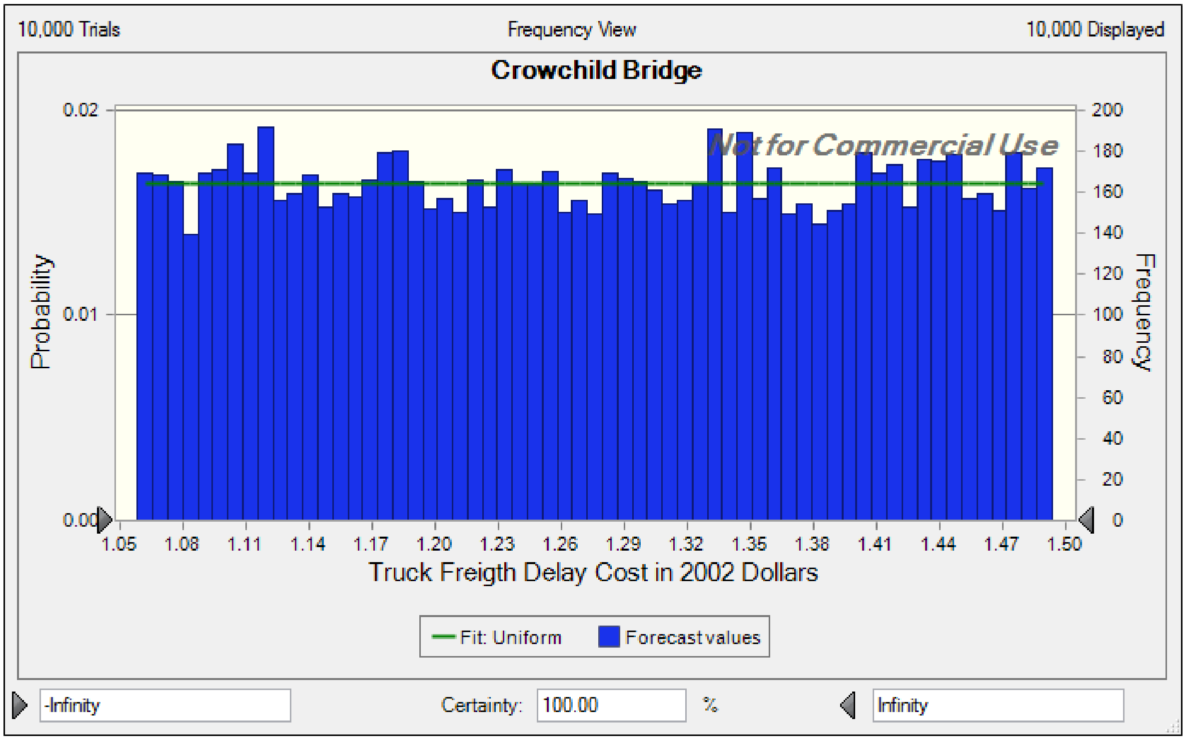

Figure 5.6 Truck freight delay cost (F) probability distribution in 2002 dollars

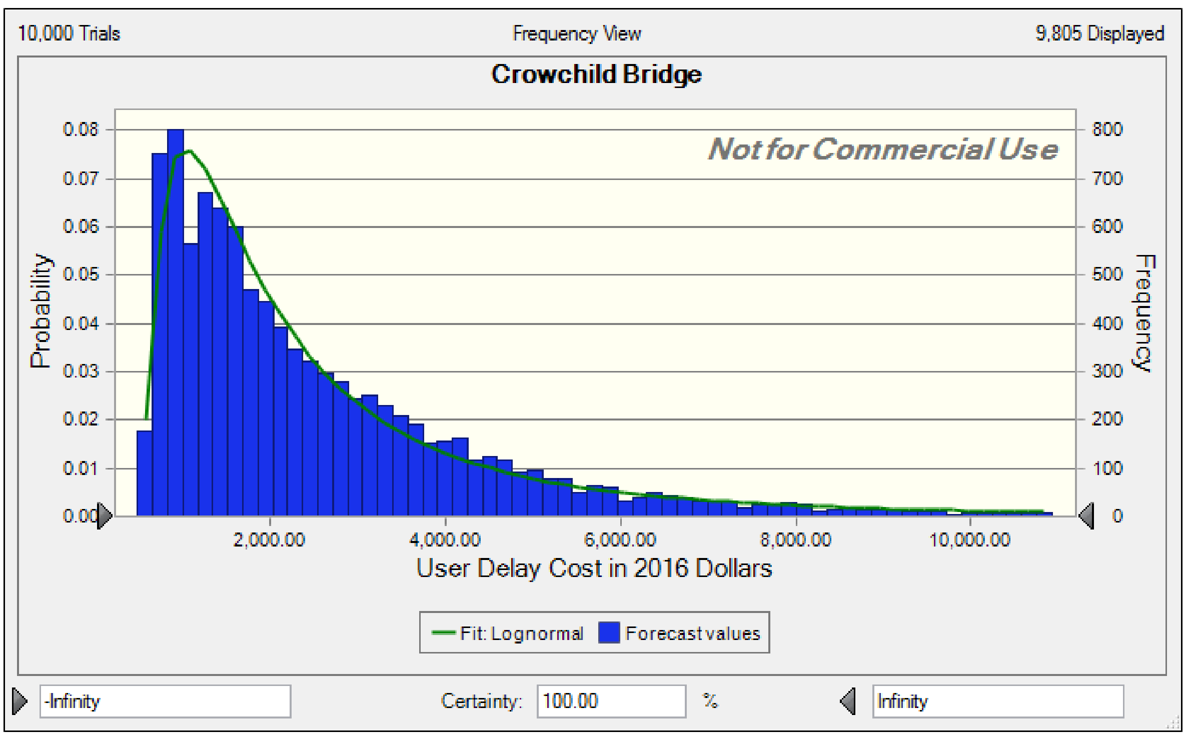

Figure 5.7 User delay cost (UDC) probability distribution in 2016 dollars

Table 5.14 Mean, Median, and Other Statistics of UDC and Components (10,000 Trials)

Statistics	Mean	Median	Standard Deviation	Minimum	Maximum	Range Width
UDC (2002 $)	2,199.4	1,544.5	2,259.6	387.2	32,168.1	31,780.9
UDC (2016 $)	2,816.0	1,97.5	2,893.0	495.8	41,184.9	40,689.1
PPC (2002 $)	1,341.7	659.7	2,237.6	0.00	31,181.1	31,181.1
BPC (2002 $)	674.3	606.5	268.6	244.1	1,661.1	1,417.0
TD (2002 $)	182.1	192.3	25.3	126.1	217.1	90.9
F (2002 $)	1.3	1.3	0.1	1.1	1.5	0.4

Table 5.15 Percentile of UDC in 2002 and 2016 Dollars (10,000 Trials)

Percentiles	0%	10%	20%	25%	30%	40%	50%	60%	70%	75%	80%	90%	95%	100%
UD in 2002 dollars	387	664	850	967	1,061	1,274	1,544	1,910	2,384	2,665	3,026	4,170	5,653	32,168
UDC in 2016 dollars	496	850	1,088	1,238	1,359	1,631	1,977	2,446	3,052	3,412	3,874	5,338	7,237	41,185

According to the results there is 50% certainty that UDC due to one hour of construction work in morning peak in Crowchild NB is greater than $1,544 (2002 dollars since here onward) and with 35% certainty it is greater than $ 2,199.4. There is 95% certainty that it is not above $ 5,653, Table 5.15 and Figure 5.8. Depending on the level of risk that a client (in this case the City of Calgary) is willing to accept, the value of UDC due to one hour of construction work in morning peak could be chosen from Table 5.15, or Figure 5.8. Summation of this value with agency cost and other quantified social costs could be a defendable base for calculating cost of construction delays and allocation of relevant contractual incentive and disincentives. In section

5.5 application of this tool in selecting optimum work zone configuration from UDC point of view is demonstrated.

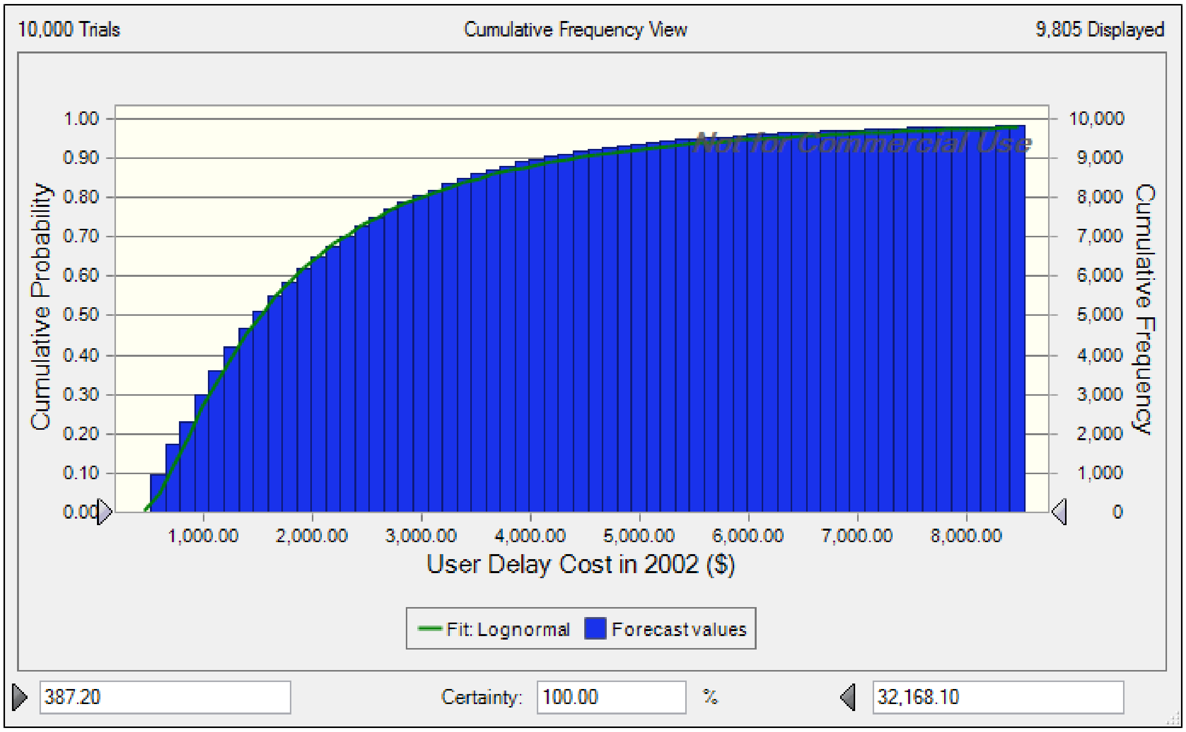

Figure 5.8 Cumulative user delay cost (UDC) distribution in 2002 dollars

5.5. Optimum Work Zone Layout During Rehabilitation of Bow Bridge From User Delay Cost Point of View

Total cost of a work zone is the sum of agency cost (direct costs), UDC, and other social costs and therefore an optimum work zone is the one that has minimum total cost. As it is remembered from Chapter 4, three work zone configurations for rehabilitation of Bow Bridge in 2003 were micro-simulated (Section 4.1, Figure 4.1) and passenger cars delay and trucks delay associated with these work zones were calculated (Section 4.7). Table 5.16 which is restatement of Table 4.24 is a summary of the assessed work zone configurations and their associated delays.

In this section application of the developed tool in selecting optimum work zone configuration from UDC point of view will be demonstrated. Due to the scope limitation the cost calculation for each work zone configuration is limited to their UDC. In other words, it is hypothetically assumed that monetary value of agency cost and other social costs of studied work zone configurations (Model 1, Model 2, and Model 3) are equal and could be deleted out of the total cost equation in scenario comparisons. However, in a real world project these cost components need to be independently calculated for each work zone configuration and added to the associated UDC in order to calculate the total cost of each work zone configuration, then using the following demonstrated methodology the optimum work zone configuration would be selected.

Table 5.16 Assessed Work Zone Layouts in Bow Bridge Study Cordon and Associated Delays

Model	Work Zone Configuration	User Delay (hr/ voc)	
		Cars	Trucks
1	Closing one lane in each direction	186.6	10.1
2	Complete detour of WB traffic from Bowness Rd	736.8	25.2
3	Closing one bound and diverting traffic to the opposing bound	288.1	16.8

5.5.1. Optimum work zone layout and ANOVA test.

Using the developed tool UDC distribution associated with each work zone configuration was calculated, Figure 5.9 to Figure 5.11 and the related statistics for cost comparison extracted (mean, standard deviation, no of trials, etc.) in 2003 dollars, see Table 5.17. ANOVA (Analysis of Variances) was used to investigate any significant difference between the mean UDC of the three scenarios. As seen in Table 5.18 the p-value is less than 0.001 which means the mean UDC of the three work zone configurations are not statistically equal, so two by two comparisons using one-sided t tests were conducted to figure out the scenario with the minimum UDC. It is worth mentioning, these three cost distributions are representative of the population; as a result, z statistics will be more appropriate testing method; however, considering the sample size (10,000 trials), t statistics will approach the same value of z statistics. In other words, in this specific case these two statistical tests could be used interchangeably.

Table 5.17 Statistics of User Delay Cost ($) Associated with Bow Bridge Work Zone Layouts

Scenario	Mean ($)	Standard Deviation ($)	Standard Error ($)	No of Trials
Model 1	2,574.57	2,644.27	26.44	10,000
Model 2	9,998.94	10,180.99	101.81	10,000
Model 3	4,036.81	4,013.01	40.13	10,000

Table 5.18 ANOVA Test Result

Parameter	SS	df	MS	F	p
Between Groups	309,349,652,466.67	2	154,674,826,233.33	3,660.964	0.000
Within Group	1,267,365,775,647.51	29,997	42,249,750.83		
Total	1,576,715,428,114.18	29,999			

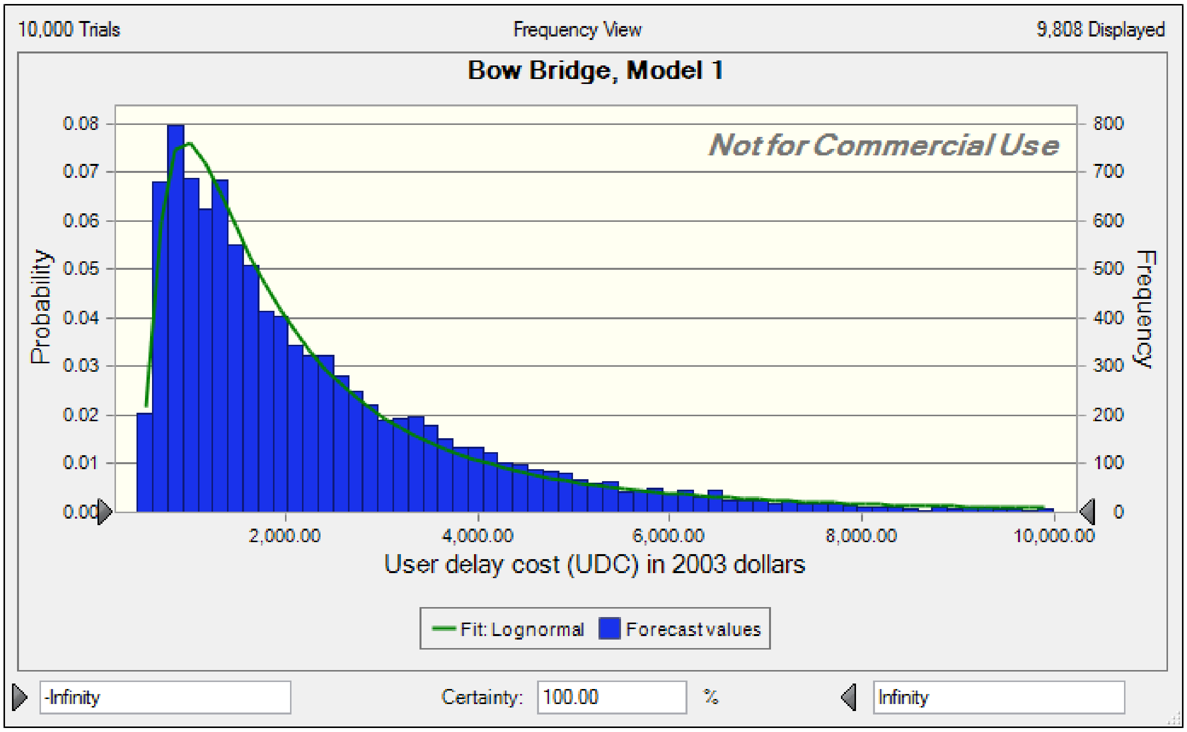

Figure 5.9 Bow Bridge Model 1, user delay cost (UDC) distribution in 2003 dollars

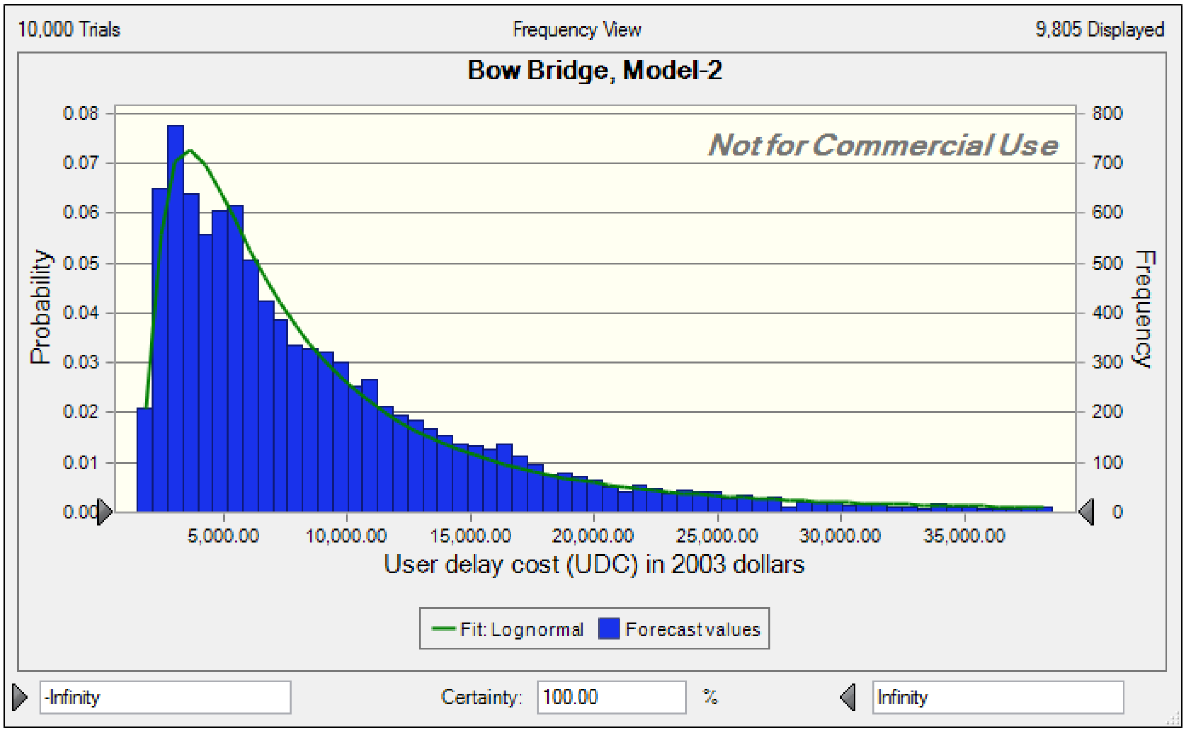

Figure 5.10 Bow Bridge Model 2, user delay cost (UDC) distribution in 2003 dollars

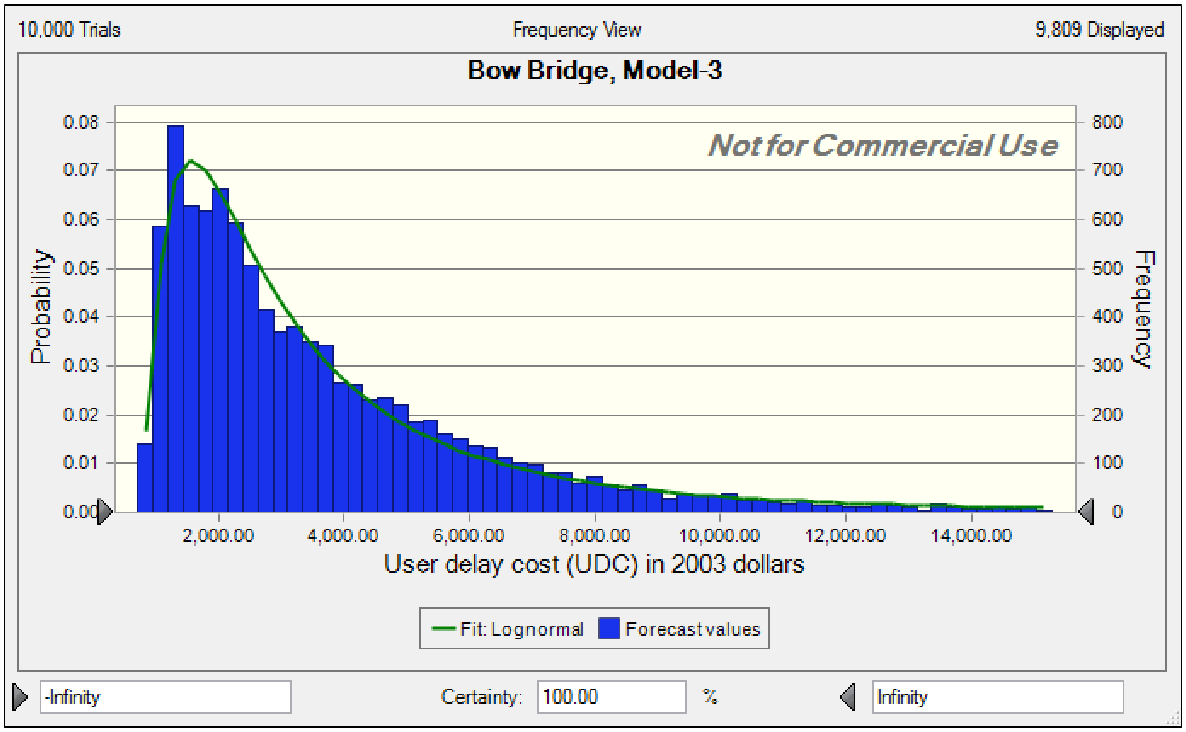

Figure 5.11 Bow Bridge Model 3, user delay cost (UDC) distribution in 2003 dollars

In order to compare the UDC of the work zone scenarios two by two, the following null and alternative hypothesis is tested, (equation 5.12 to equation 5.15). In these equations μ_1, μ_2, μ_3 are representatives of mean UDC associated with Model 1, Model 2, and Model 3, respectively which were summarized in Table 5.17.

H_0: $\mu_1 = \mu_3$	*Mean UDC of Model 1 is equal to Mean UDC of Model 3*	*5.12*
H_1: $\mu_1 < \mu_3$	*Mean UDC of Model 1 is less than Mean UDC of Model 3*	*5.13*
H_0: $\mu_3 = \mu_2$	*Mean UDC of Model 3 is equal to Mean UDC of Model 2*	*5.14*
H_1: $\mu_3 < \mu_2$	*Mean UDC of Model 3 is less than Mean UDC of Model 2*	*5.15*

The purpose of equation 5.9 and 5.10 is to test if the mean UDC associated with Model 1 is less than that of Model 3. According to the one-sided t test result, see Table 5.19, p-value is less than 0.0001 and at 95% confidence level, mean UDC associated with Model 1 is less than that of Model

3. Similarly, the goal in equation 5.11 and equation 5.12 is to investigate if mean UDC associated with Model 3 is higher than that of Model 2. According to the one-sided t test result, see Table 5.19, p-value is less than 0.0001. In other words, at 95% confidence level mean UDC associated with Model 3 is less than that of Model 2.

Table 5.19 One-Sided t Test Result

Case		T statistic	P
H_0: $\mu_1 = \mu_3$	H_1: $\mu_1 < \mu_3$	30.43	<0.0001
H_0: $\mu_3 = \mu_2$	H_1: $\mu_3 < \mu_2$	54.48	<0.0001

It can be concluded that the mean UDC associated with Model 1 is less than that of Model 3 which is less than that of Model 2; in other words, "$\mu_1 < \mu_3 < \mu_2$". Consequently, according to these analyses during the rehabilitation of Bow Bridge in 2003, Model 1 or conventional method of closing one lane in each direction is the most cost effective work zone configuration from UDC point of view.

5.6. Summary & Conclusion

In this chapter the concept of road user delay cost (UDC) and its components were explained. According to USDOT, the value of travel time is composed of the following components:

- Monetary value of personal travel time with passenger cars (PPC)
- Monetary value of business travel time with passenger cars (BPC)
- Monetary value of truck drivers travel time with truck (TD)
- Cost of freight inventory delay carried by trucks (F)
- Cost of vehicle depreciation including all vehicles (DP)

Then FHWA's deterministic approach for calculating monetary value of user delay based on the USDOT's definition of the travel time value was detailed (Mallela & Sadavisam, 2011). Considering the probabilistic nature of the input data required for calculating UDC, the probabilistic approach developed in this research as an advanced form of the FHWA's deterministic method was introduced. This approach incorporates the probabilistic nature of UDC components with FHWA methodology in order to provide decision makers with not only the expected value of UDC but also with the probability distribution of UDC associated with the work zone. In other word, decision makers are provided with the certainty associated with each potential UDC value they consider for the work zone. Using this approach, a tool has been developed in Crystal Ball software environment which using Monte-Carlo simulation method, calculates probability distribution associated with UDC and its components.

Application of the tool for monetizing user delay has been elaborated through the case of Crowchild Bridge rehabilitation in 2002. As it is remembered from Chapter 3, user delay due to the rehabilitation of this bridge were calculated using traffic micro-simulation. One of the major challenges in monetizing UDC was collecting Canada specific economic input data. Since UDC calculation is not a common practice in Canada's roadway construction industry, there is no tailor made free data for this purpose and in some cases, like freight value, no data is available. So, data which were the best representative of the situation were used. Source of the input data, associated challenges, and justifications for using each of the input data has been elaborated in Section 5.3. According to the findings every hour of morning peak rehabilitation in NB Crowchild Bridge resulted in UDC probability density function which follows log normal distribution with mean and standard deviation equal to \$ 2,199.4 and \$2,259.6, respectively in 2002 dollars or (mean and standard deviation equal to \$ 2,816.0 and \$2,893.0, respectively in 2016 dollars), see Table 5.14.

There is 95% certainty that UDC is equal or less than \$5,652.54 in 2002 dollars (\$ 7,236.98 in 2016 dollars, see Table 5.15).

The application of the tool for selecting optimum work zone layout from UDC point of view has been demonstrated by the case of Bow Bridge rehabilitation in 2003. As it is detailed in Chapter 4, three work zone configurations were studied and travel delays associated with each work zone configurations were calculated using traffic microsimulation. These work zone configurations are as the following:

- Model 1: Closing One Lane in Each Direction
- Model 2: Complete Detour of WB traffic from Bowness Rd
- Model 3: Closing One Bound and Diverting Traffic to The Opposing Bound

Using the developed tool probability distribution of UDC associated with each of the work zone configurations were calculated, see Table 5.17, then using ANOVA test it is examined if there is any significant difference in the UDC of these three work zone layouts. The results showed there is significant difference in their UDC ($p < 0.001$). Then using two series of one-sided t tests, the work zone layout with least UDC was evaluated. According to the test results, UDC associated with Model 1 is less than that of Model 3 which is less than that of Model 2; so, in this case traditional method of closing one lane in each direction and limiting traffic to one open lane was the most cost effective alternative from UDC point of view.

In this research two application of the developed probabilistic tool was explained; however, it could be used for other purposes as well, for instance: selecting ABC versus conventional construction method, developing base charts for calculating UDC, and calculating contractual incentive and disincentives (I/D).

Chapter Six: Conclusion, Contributions, And Future Works

This chapter focuses on the research summary, its results and contribution, and suggestions for future works. Section 6.1 presents a brief description of the research and summary of the findings. Section 6.2 explains the contributions of the research and in section 6.3 suggestion for future works are presented.

6.1 Research Summary and Results

The total cost of roadway construction includes direct and indirect costs. Direct cost is a visible cost comprised of engineering, construction, construction supervision, and administration costs, while indirect cost is the invisible cost paid by the road users, known as user cost. User cost (UC) has three main components:

- User delay cost (UDC)

  Monetary value of increased travel time due to the either reduced posted speed through the work zone or congestion ahead of the work zone

- Added vehicle operation cost (AVOC)

  Increased cost of vehicle operation due to the increased travel time through the work zone and nature of driving through the work zone accompanied with acceleration and deceleration

- Added accident cost (AAC)

  The work zone area is more susceptible to accident than normal condition of a roadway.

Work zone instigated UC in densely populated areas could outweigh the direct costs of a project and therefore need to be considered in project alternative selection. This is the case in bridge rehabilitation projects in major urban arterials where there is high traffic demand with limited or no alternative river crossing as well as limited work zone space and layout options.

In chapter 2 of this thesis, research works on UC in Canada and other parts of the world with focus on the USA was presented. USA is the leading country in research on UC with the FHWA's 2011 report being the most comprehensive work on this regard (Mallela, J. and Sadavisam, S., 2011). The current research trend focuses on application of UC in justifying accelerated construction (AC) methods and designing contractual incentives and disincentives (I/D). I/D are contractual measures to encourage the contractor to finish construction works ahead of time (I) or on time (D). However, both FHWA, 2011 report and other research works, use a deterministic approach for calculating UC while UC is probabilistic in nature. Additionally, most of the research work on UC focuses on freeways or rural highways rather than urban arterials where considerable delay could occur to the traffic in the network as a result of construction work zone.

To address these gaps this research applied traffic microscopic simulation to include the effect of nearby controls (traffic signals) in work zone instigated delay calculations in urban setting (the subject of Chapter 3 and Chapter 4). Moreover, the probabilistic nature of UC was addressed by incorporating Monte-Carlo simulation technique in Monetizing UC (subject of Chapter 5). Due to the time frame of master program only UDC component of UC was focused although the developed method is applicable to other components (AVOC, AAC) as well. The developed tool was demonstrated using the recorded travel data during the rehabilitation of Crowchild Tr Bridge in Calgary, Canada. Additionally, application of the tool in selecting optimum work zone layout from UDC point of view was demonstrated using the data from the rehabilitation of Bow Bridge in Calgary, Canada. The following is the summary of the findings.

In Chapter 3 application of traffic microsimulation for calculating user delay due to one hour of construction work on a bridge during morning rush hour was introduced. Data from rehabilitation of Crowchild Tr Bridge in 2002 was used as the case study. In this case the work zone composed of closing all but one Northbound (NB) lanes of Crowchild Bridge. Traffic study cordon were built around the bridge, then using SimTraffic software (Version 6) two traffic microsimulation models were developed. One is simulating traffic on Crowchild Tr and other streets of the study cordon during normal condition (no work zone) and the other model is simulating traffic on the same study cordon under work zone condition.

In order to calculate total delay due to the construction work, the travel time per vehicle on the targeted streets during the normal condition and work zone condition were simulated then in those targeted streets, the cumulative travel time (average travel time per vehicle × number of vehicles) under normal and work zone condition were calculated. The difference of these two cumulative travel times is the total delay per vehicle occupancy rate (voc) which is incurred due to the one hour of construction work in morning peak. Based on the results of this study the total delay incurred to vehicles due to the one hour of construction work in morning peak is 169.2 (hr/voc). It is composed of 8.8 (hr/voc) of trucks delay and 160.4 (hr/voc) of cars delay.

In Chapter 4 using the methodology applied in Chapter 3 and data from the rehabilitation of Bow Bridge in 2003, the effect of different work zone layouts on user delay was studied. The ultimate goal was demonstrating the application of UDC in selecting optimum work zone layout (subject of Chapter 5). Construction work on Bow Bridge interrupted access from "Trans Canada Highway (TCH) & Home Rd" intersection to "TCH & Sarcee Tr" interchange via TCH. In this study three different work zone layouts were micro-simulated using SimTraffic software and user delay associated with either layout was calculated. The examined work zone layouts are as the followings:

- Model 1: Closing one lane in each direction (work zone in 2003)
- Model 2: Complete detour of EB traffic via nearby major Street (Bowness neighborhood)
- Model 3: Closing one bound and diverting traffic to the opposing bound

Based on these calculations Model 1 resulted in minimum user delay. The total user delay in this case was 186.6 (hr/ voc) of cars delay and 10.1 (hr/ voc) of trucks delay, Table 6.1. Model 3 stood on the second place from minimum user delay point of view, 288.1 (hr/ voc) of passenger cars delay and 16.8 (hr/ voc) of trucks delay. Model 2 created the highest user delay, 736.8 (hr/ voc) of cars delay and 25.2 (hr/ voc) of trucks delay.

Table 6.1 User Delay Cost Associated With Different Work Zone Configurations (Bow Bridge)

Model	Work Zone Configuration	User Delay (hr/ voc)	
		Cars	Trucks
1	Closing One Lane in Each Direction	186.6	10.1
2	Detour From Bowness Rd	736.8	25.2
3	Closing One Bound [Traffic Light: Cycle Length= 361s, Green WB= 103.5 s]	288.1	16.8

According to USDOT, the value of travel time is composed of the following components (Mallela & Sadavisam, 2011):

- Monetary value of personal travel time with passenger cars (PPC)
- Monetary value of business travel time with passenger cars (BPC)
- Monetary value of truck drivers travel time with truck (TD)
- Cost of freight inventory delay carried by trucks (F)

- Cost of vehicle depreciation including all vehicles (DP)

In Chapter 5 after elaborating the concept of UDC and its components, FHWA's deterministic approach for calculating monetary value of user delay based on the USDOT's definition of the travel time value was detailed (Mallela & Sadavisam, 2011). Considering the probabilistic nature of the input data required for calculating UDC, in this research a probabilistic approach was developed as an advanced form of the FHWA's deterministic method. The developed approach incorporates the probabilistic nature of UDC components with FHWA methodology in order to provide decision makers with not only the expected value of UDC but also with the probability distribution of UDC associated with the work zone. Based on this approach, an Excel tool was developed which using Monte-Carlo simulation, calculates statistics and probability distribution associated with UDC and its components.

According to the findings every hour of morning peak rehabilitation in NB Crowchild Bridge resulted in UDC probability density function which follows log normal distribution with mean and standard deviation equal to \$ 2,199.4 and \$2,259.6, respectively in 2002 dollars or (mean and standard deviation equal to \$ 2,816.0 and \$2,893.0, respectively in 2016 dollars. There is 95% certainty that UDC is equal or less than \$5,652.54 in 2002 dollars (\$ 7,236.98 in 2016 dollars).

The application of the tool for selecting optimum work zone layout from UDC point of view was demonstrated by the case of Bow Bridge rehabilitation in 2003. Using the developed tool probability distribution of UDC associated with each of the work zone configurations of Bow Bridge were calculated, see Table 6.2, then using ANOVA test it is examined if there is any significant difference in the UDC of these three work zone layouts. The results showed there is significant difference in their mean UDC ($p < 0.001$). Then using two series of one-sided t tests, the work zone layout with least UDC was evaluated. According to the test results, UDC associated with Model 1 is less than that of Model 3 which is less than that of Model 2; so, in this case traditional method of closing one lane in each direction and limiting traffic to one open lane was the most cost effective alternative from UDC point of view.

Table 6.2 Mean User Delay Cost ($) Associated with Bow Bridge Work Zone Layouts

Scenario	Model 1	Model 2	Model 3
Mean in 2003 dollars	2,574.57	9,998.94	4,036.81

In this thesis two application of the developed probabilistic tool was explained; however, it could be used for other purposes as well, for instance: selecting accelerated bridge construction (ABC) methods versus conventional construction method, developing base charts for calculating UDC, and calculating contractual incentive and disincentives (I/D).

6.2 Research Contributions

At the moment there is no tool for quantifying road occupancy cost in urban setting in Calgary. The result of this research will provide the City of Calgary with a decision support tool to quantify road occupancy cost associated with its road work particularly bridge rehabilitation activities and thereby the City could incentivize its contractors to finish projects in a timely manner and consequently, reduce congestion and economic costs to the City. Additionally, the developed tool could assist the City with selecting the most optimum work zone layout and justifying accelerated bridge construction method versus conventional construction methods.

6.3 Future Research Scope and Recommendations

Recommendations for future research are categorized in to two areas of technological development and policy developments.

Technological:

- Inclusion of work zone instigated added vehicle operation cost (AVOC), added accident cost (AAC), added emission cost (AEC) in the developed tool.
- Research on application of the probabilistic user cost approach in justifying accelerated construction (AC) methods particularly case of arterial bridges with high traffic volumes in Canada.
- Research on the percentage of user cost to be considered as contractual incentives and disincentives tailor-made to cold weather condition.
- Real time monitoring of UDC and incorporation into ITS.

Policy Development

- Research on potential barriers to the introduction and implementation of user delay concept in Canadian roadway industry.
- Developing guideline for collecting required input data for calculating UC with the help of relevant agencies including Statistics Canada, Ministries of Transportation, and municipalities.
- The importance of quantifying UC (particularly VOC and AEC components) on Canada's greenhouse gas emissions reduction target.

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Appendix I: Some Features of SimTraffic Software

A.1. Car Following Model

The popularity of traffic microsimulation models is continuously increased by increase in the traffic volume at urban networks and reduction in the available funds for existing tools (Trueblood, 2013). Traffic microsimulation models simulate behavior of every individual driver and describe the interaction of every individual vehicle with other vehicles and the road network. These models are generally a composition of sub models which control specific task in the simulation process among them behavioral models. One of the most important behavioral models is car following model which controls the driver's behavior with respect to the preceding vehicle in the same lane. In other word acceleration, deceleration and distance keeping of the driver (Olstam & Tapani, 2004). Car flowing models could be classified into main three categories based on the logic utilized:

- Gazis-Herman-Rothery models (GHR): Based on these models the trailing vehicle's acceleration is proportional to the speed of the trailing vehicle, the speed difference between trailing and leader vehicle and the space gap between them. These are known as the Basic models as well (Olstam & Tapani, 2004).
- Psycho-physical car-following models (General Motors models): These models use thresholds for, e.g., the minimum speed difference between trailing and leading vehicle perceived by the follower (Olstam & Tapani, 2004).
- Safety distance models (Advance models): Safety distance models are based on the assumption that the trailing vehicle always keeps a safe distance to the leading vehicle in order to smoothly reach the desired speed or safely proceed behind it (Olstam & Tapani, 2004).

SimTraffic simulation model apply a sets of formula for car following, which has been presented in the following paragraphs. The software manual does not clarify in which of the above car following categories its car following model is classified; however, based on the utilized formulation it is at the category of Safety distance models (advance models). There are two car following algorithms in the software depending on the speed of the leading vehicle:

- Fat following will be used if the leading vehicle travels with speed over 0.6 m/s
- Slow following is used to follow a slow or stopped vehicle or to stop at a fixed point such as the stop-bar or mandatory lane change start point (Husch & Albeck, 2003).

Fast Following

In this case a safe distance between trailing and leading vehicle is defined based on their differential speed and distance between them, deceleration rate, speed, and the desired headway of the trailing vehicle equation A.1. Then a safety factor is defined based on the DSafe and speed of the leading vehicle and desired headway, equation A.2, which is a base for the software to change the distance between leading and the trailing vehicle by changing the acceleration or deceleration rate of the trailing vehicle known as dv in the software, see Table A.1. The fast car following model uses 0.1 s time slices for car following purposes. The following formulas are exact transfer from the SimTraffic manual (Husch & Albeck, 2003).

$$DSafe = DBv + min\,(spdU^2 - spdV^2, 0)/(2 \times DecelNormal) - spdV \times HW \qquad I.1$$

$$Sf = Dsafe/\,(spdV \times HW) \qquad I.2$$

Where:

- DSafe= distance between vehicles, adjusting for speed differential and reduced by trailing vehicle's HW
- HW: desired headway (dependent on driver parameters and link headway factor

- *spdU: speed of the leading vehicle*

- *spdV= speed of the trailing vehicle*

- *DBV= distance between vehicles*

- *Sf: safety factor*

- *dv: recommended acceleration or deceleration*

Table 0.1 Vehicle deceleration or acceleration rate based on the safety factor (sf)

Sf	Rule
Sf = 0	Vehicle is at correct distance
Sf = -1	Vehicle is 1 headway too close, unsafe following and maximum deceleration is applied
Sf = 1	Vehicle is 1 headway too far , accelerate
Sf >= -0.3	$dV = decelHard \times \frac{Sf}{1.5}$
-0.3 > Sf >= -1.0	$dV = decelHard \times [-0.2 + (SF + 0.3) \times 8/7]$
Sf >-1.0	$dV = -decelHard$ *

*decelhard is the maximum possible deceleration rate (3.6 m/s^2), this is normally reserved for crisis situations

The reader is referenced to the SimTraffic manual for further detail.

<u>Slow Following</u>

The following methods is used for the slow following:

$$DB2 = DBv - 2 \times spdU / 10 - 1 \qquad I.3$$

$$dv2 = \frac{(spdV + 2 \times accelMin / 10)^2}{(2 \times DB2)} \qquad I.4$$

$$dv4 = \frac{(spdV + 4 \times accelMin / 10)^2}{(2 \times DB2)} \qquad I.5$$

$$dv6 = \frac{(spdV + 6 \times accelMin / 10)^2}{(2 \times DB2)} \qquad I.6$$

Where:

- DB2: new distance between leading and trailing vehicles after 0.1 second
- DBv: *distance between vehicles*
- dv2: deceleration required after accelerating at 2×(accelMin= 0.6m/s^2)
- dv4: deceleration required after accelerating at 4×(accelMin= 0.6m/s^2)
- dv6: deceleration required after accelerating at 6×(accelMin= 0.6m/s^2)

According to the software manual if dv2, dv4, or dv6 are greater than (-decelNormal= -1.2 m/s^2) then the vehicle will accelerate by (2×accelMin), (4×accelMin), or (6×accelMin), respectively subject to the vehicle's maximum acceleration capabilities (Husch & Albeck, 2003).

If $(dv2 < -decelNormal)$ then *I.7*

$$dV = -spdV^2 / (2 \times DB2)$$

If $(DB2 < 0)$ then *I.8*

$$dV = -decelMax$$

Where:

- DB2: new distance between leading and trailing vehicles after 0.1 second
- dv: deceleration rate of the trailing vehicle
- decelNormal:1.2 m/s^2
- decelMax:3.6 m/s^2

The acceleration must be greater or equal to -decelMax and less than or equal to the vehicle's maximum acceleration capabilities (Husch & Albeck, 2003).

A.2. Route Assignment

Route assignment in SimTraffic is based on the traffic volumes. These volumes could be adjusted for growth factor, PHF, or percentile adjustment. When a vehicle is created it is assigned a turn at the end of its link and the next eight links. The turns are random based on the turning counts for each direction (Husch & Albeck, 2003). In other words, this software uses static routing, meaning that drivers select the route at departure without further update along the journey during simulation (Chiu et al., 2011). So, in this research static routing has been used. The following is the route assignment algorithm of the software based on the software manual (Husch & Albeck, 2003).

If $(Rv < vLeft)$ then $(Vehicle\ turns\ left)$ *I.9*

Else If $(Rv < (vLeft + vThru))$ then $(Vehicle\ proceeds\ straight)$

Else $(Vehicle\ turns\ right)$

Where:

- *vT: sum of approach traffic*
- *Rv = random number between 0 and vT -1*
- $vLeft$: $nun = number\ of\ left\ turn\ at\ the\ intersection$
- $vThru$: $number\ of\ through\ movements\ at\ the\ intersection$

A vehicle may also be assigned to a mid-block sink using similar logic.

$$(vSink = vUp - vT + vMidBlock) \qquad I.10$$

Where:

- *vSink: sink volume*
- *vUp = volume from upstream intersection*
- $vMidblock$: $volume\ entering\ mid - block$ (Husch & Albeck, 2003).