ABSTRACT

Title of dissertation: INTEGRATED MANAGEMENT OF HIGHWAY MAINTENANCE AND TRAFFIC

Chun-Hung Chen, Doctor of Philosophy, 2003

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Highway maintenance, especially pavement rehabilitation or resurfacing, requires lane closures. This work develops an integrated model to help highway agencies in developing traffic control plans for maintenance activities and in efficiently managing traffic around highway work zones. Thus, the objective of this study is to develop methods for optimizing work zone characteristics in order to minimize the combined total costs for highway agencies and users. Work zone models are developed for three cases: (1) a single maintained road with steady traffic inflows, (2) a single maintained road with time-dependent inflows, and (3) a road network with multiple detour paths, as well as plans for maintenance activities and managing traffic around highway work zones.

For Case 1, with steady traffic inflows, four alternatives for two-lane highways and four alternatives for four-lane highways are proposed. Analytical solutions are found for optimized work zone lengths and diversion fractions based on minimizing the total cost. Guidelines for selecting the best alternative for different characteristics of traffic flows, road and maintenance processes are developed by deriving thresholds among alternatives. In Case 2, the models for two-lane highway and four-lane highway work

zones for time-dependent inflows are developed. Two optimization methods, Powell's and Simulated Annealing, are adapted for this problem and compared. In numerical tests, a Simulated Annealing algorithm yields better solutions using less computer time than Powell's Method. In testing the reliability of Simulated Annealing, the statistical analysis for 50 replications of the cost minimization indicates that Simulated Annealing is very likely to find solutions that are very close in value to the global optimum. The SAUASD (Simulated Annealing for Uniform Alternatives with a Single Detour) algorithm is developed to find the best single alternative within a maintenance project. The SAMASD (Simulated Annealing for Mixed Alternatives with a Single Detour) algorithm is developed to search through possible mixed alternatives and diverted fractions in order to further minimize total cost. Thus, traffic management plans with uniform alternatives or mixed alternatives within a maintenance project are developed.

For Case 3, work zone optimization models for a road network with multiple detour paths and the SAMAMD (Simulated Annealing for Mixed Alternatives with Multiple Detour paths) algorithm are developed. For analyzing traffic diversion through multiple detour paths in a road network, the SAMAMD algorithm is used to optimize work zone lengths and schedule the resurfacing work. Analyses based on the CORSIM simulation are used not only to estimate delay cost, but also to evaluate the effectiveness of optimization models. A comparison of the results from optimization and simulation models indicates that they are consistent. The optimization models do significantly reduce total cost, including user cost and maintenance cost, compared to the total cost of the current resurfacing policy in Maryland.

INTEGRATED MANAGEMENT OF HIGHWAY MAINTENANCE AND TRAFFIC

by

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DEDICATION

To my family

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Chapter I Introduction

1.1 Background

Highway maintenance, especially pavement rehabilitation or resurfacing, requires lane closures. Given the very substantial cost of the maintenance and the very substantial traffic disruption and safety hazards associated with highway maintenance work, it is desirable to plan and manage the work in ways that minimize the combined cost of maintenance, traffic disruptions and accidents. Work zone delays due to highway maintenance have been increasing in the U.S (Federal Highway Administration (FHWA), 2000). The aging highway system in the U.S. is undergoing extensive of reconstruction and maintenance in recent years. According to the FHWA (Wunderlich, 2003), 13 percent in 2001 and 20 percent in 2002 of the National Highway System (NHS) were under construction during the peak summer road work season and work zones on the NHS resulted in a loss of over 60 million vehicles of capacity per day. The number of persons killed in motor vehicle crashes in work zones has risen from 693 fatalities in 1997 to 1,181 fatalities in 2002 (an average of 936 fatalities a year) and more than 40,000 people are injured each year as a result of motor vehicle crashes in work zones (Traffic Safety Facts 2002, National Highway Traffic Safety Administration (NHTSA), 2003).

Highway maintenance and the management of traffic through or around work zones are important activities. Appropriate traffic management plans can increase the work efficiency and safety and decrease work zone delays. FHWA's statistics also show that 53 percent of work zones are designated for day work, 22 percent for night work, and 18 percent are active all day or nearly all day (Wunderlich, 2003). However, no comprehensive method has been developed to evaluate whether these work zones are dimensioned and scheduled appropriately, allowing motorists to travel safely and smoothly, and allowing work crews to accomplish their work safely. Therefore, it is worthwhile to develop appropriate work zone analysis methods that can be used to evaluate current work zone plans and to develop better traffic management plans for highway maintenance activities.

1.2 Problem Statement

The overall costs of road maintenance and traffic disruption may be very significantly reduced through properly integrated decisions about the conduct and schedule of maintenance activities and the development of appropriate traffic management plans. Several questions should be considered for comprehensive analysis:

- How long and wide should work zones be?
- How does the availability of alternate routes and their characteristics (e.g., length, design speed, excess capacity, traffic patterns) influence the above decisions?
- What fraction of traffic should be diverted to alternate routes?

When time-dependent inflows are considered, the analysis becomes more complex. Besides the above questions, the work scheduling, i.e., when the work should be done and how long closures should be last, must also be analyzed. The optimal work zone activities, including the optimized work zone lengths in different periods (day, night, peak period, off-peak period), the preferred starting time and ending time for each zone closure (e.g. terminating work during peak period to avoid to too serious traffic disruption), are also included among the problems considered. When considering time-dependent inflows, traffic management plans combining different alternatives, which have different work zone configurations or diversion, for different periods might be developed and applied to highway maintenance projects.

The above questions focus on a single maintained road. Furthermore, when a more complex road network is considered, not only should multiple detour paths be considered, but the scheduling of maintenance activities for roads in a road network must also be determined. Thus, the following two questions will be identified and solved:

1. How should roads and road networks be divided into work zones?
2. How does the effectiveness of various maintenance and traffic management solutions depend on the characteristics of particular road sections and the surrounding network, especially when considering multiple detour paths?

Various methods have been previously developed for analyzing some aspects of the above questions. However, no comprehensive method has been previously developed to jointly analyze these questions. This study aims to develop an integrated model as a decision support system to help highway agencies in developing traffic control plans for maintenance activities and in efficiently managing traffic around highway work zones. Work zone models will be developed for three cases: (1) a single maintained road with steady traffic inflows, (2) a single maintained road with time-dependent traffic demands, and (3) a road network with multiple detour paths, as well as plans for maintenance activities and managing traffic around highway work zones.

1.3 Research Objectives

The objective of this research is to develop an evaluation and decision support model for highway maintenance planning and traffic management. This research is intended to:

1. Identify feasible alternatives of work zone activities for various traffic control strategies and evaluate in detail their costs and other effectiveness measures for three different cases, namely, (1) steady traffic inflows, (2) time-dependent inflows, and (3) a road network with multiple detour paths.
2. Optimize the work zone characteristics to minimize the combined total costs for highway agencies and users.
3. Develop scheduling strategies and traffic management plans for the above three cases.

1.4 Research Scope and Tasks

Based on highway configuration, the scope of this study will cover (1) two-lane two-way highway work zones and (2) multiple-lane two way highway work zones. Based on traffic flow patterns, the scope will cover (1) steady traffic inflows and (2) time-dependent inflows. Based on detour type, the methods will cover (1) a single detour and (2) multiple detour paths.

The research tasks include the following:

- Classification of highway configuration and identification of possible work zone closure alternatives

- Development of work zone cost functions and an analytical optimization method for a single maintained road and a single detour with steady traffic inflows
- Development of work zone cost functions and optimization models (based on analytic method and Simulated Annealing algorithm) for a single maintained road and a single detour with time-dependent inflows
- Development of work zone cost functions and optimization models using analytic method, Simulated Annealing algorithm, and simulation model for a road network with multiple detour paths
- Development of appropriate traffic management plans combining different alternatives for all the cases analyzed

Figure 1.1 shows a flow chart for the tasks in this study.

1.5 Technical Approach

The objective of the work zone optimization problem is to minimize the total cost for work zone activities. The objective function for work zone activities can be expressed as follows:

$$Min\ C_T=C_M+C_U$$

where C_T is total cost, C_M is maintenance cost, or supplier cost, and C_U is user cost. The controllable variables affecting C_M include work zone length, fixed setup cost, and average maintenance cost per unit length; the controllable variables affecting C_U include work zone length, traffic volumes, speed, diverted fraction (if detour is available), etc. Both C_M and C_U are functions of work zone length since C_M and C_U are significantly influenced by work zone size.

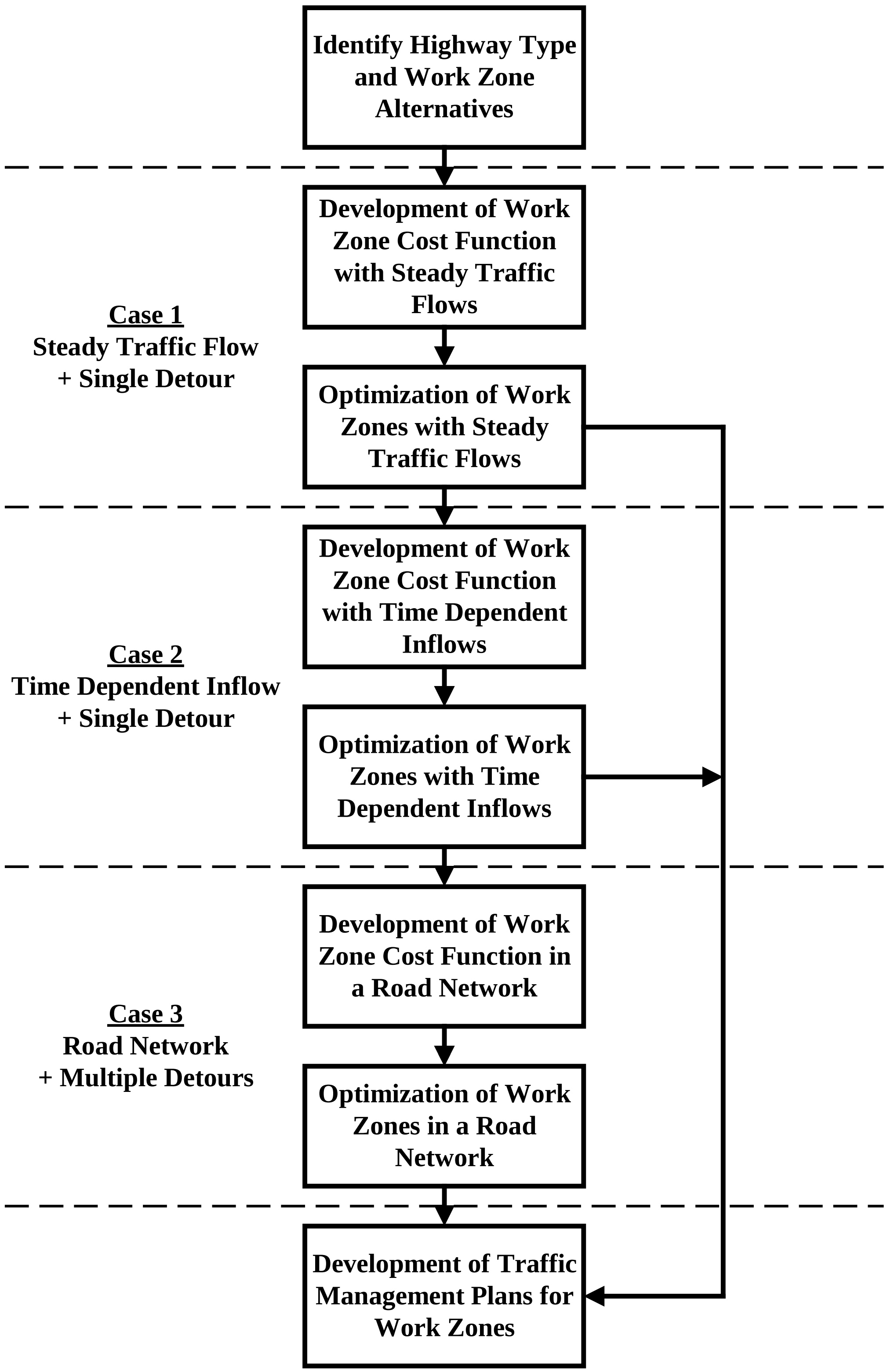
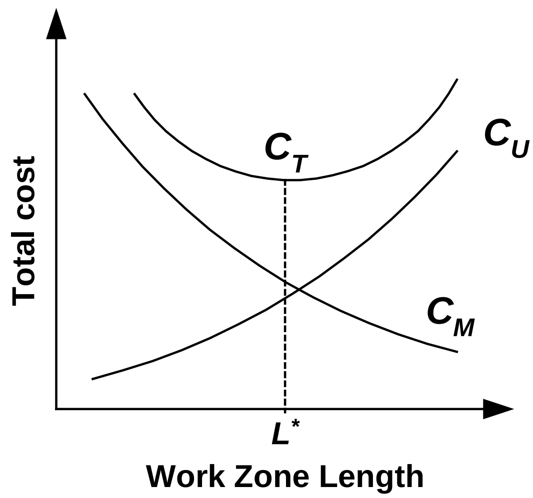


Figure 1.1 Research Flow Chart



Key: C_M = Maintenance Cost

C_U = User Cost

C_T = Total Cost

Figure 1.2 Conceptual Effect of Work Zone Length on Total Cost, Maintenance Cost, and User Cost

Chien et al (2001, 2002) proposed that longer zones tend to increase the user delays, but the maintenance activities can be performed more efficiently with fewer repeated setups in longer zones. Since work zones lengths and maintenance duration affect maintenance and user cost, it is important to determine the tradeoffs between maintenance cost and user cost in order to minimize total cost, as shown in Figure 1.2.

Maintenance cost usually includes labor cost, equipment cost, material cost and traffic management cost. The first step in estimating maintenance cost is to determine construction quantities/unit prices. Unit prices can be determined from highway agencies historical data on previously bid jobs of comparable scale (Wall, 1998). In this study, the cost of maintaining cost of length L is assumed to be a linear function of the form $C_M=z_1+z_2L$, in which z_1 represents the fixed cost for setting up a work zone and z_2 is the average additional maintenance cost per work zone unit length.

In this study user cost includes user delay cost and accident cost. The user delay can be classified into queuing delay and moving delay (Cassidy and Bertini, 1999, Schonfeld and Chien, 1999, Chien and Schonfeld, 2001). The user delay cost is determined by multiplying the user dealy by the value of user time (Wall, 1998). The accident cost is related to the historical accident rate, delay, work zone configuration, and average cost per accident. Chien and Schonfeld (2001) determined accident cost from the number of accidents per 100 million vehicle hours multiplied by the product of the user delay and average cost per accident and then divided by work zone length.

The proposed methodology includes the development and application of mathematical models for a single maintained road with steady traffic inflows, with time-dependent inflows, and finally, for a road network with multiple detour paths. The optimization approach is to formulate a total cost function, including agency cost (or maintenance cost) and user cost, and to find the work zone lengths and diversion fraction (if detour(s) is (are) available) which minimize that total cost function. Analytical solutions for optimized work zone lengths and diversion fraction are found. For cases where analytical solutions are impractical for time-dependent inflows and multiple detour paths, a heuristic algorithm is developed to find the optimized work zone lengths for each zone, zone start and end time, and the number of zones to minimize the total cost.

1.6 Organization of Dissertation

In this dissertation, previous studies are reviewed and summarized in Chapter 2. Work zone optimization models for steady traffic inflows are formulated and optimized analytically for two-lane and four-lane highway work zones in Chapter 3. Guidelines for selecting the best alternative for different characteristics of traffic flows, road and maintenance processes are developed by threshold analysis. In Chapter 4, the work zone optimization models for time-dependent inflows are developed. Two optimization methods, Powell's and Simulated Annealing, are adapted for this problem and compared. The reliability of the Simulated Annealing algorithm is also tested. In Chapter 5 are developed the work zone optimization models of four alternatives for two-lane highway and four alternatives for four-lane highway work zones with time-dependent inflows. The SAUASD (Simulated Annealing for Uniform Alternatives with a Single Detour) algorithm is developed to find the best single alternative within a maintenance project. The SAMASD (Simulated Annealing for Mixed Alternatives with a Single Detour) algorithm is developed to search through possible mixed alternatives and diverted fractions in order to further minimize total cost. Thus, traffic management plans with uniform alternatives or mixed alternatives within a maintenance project are developed.

In Chapter 6, work zone optimization models for a road network with multiple detour paths and SAMAMD (Simulated Annealing for Mixed Alternatives with Multiple Detour paths) algorithm are developed. For analyzing traffic diversion through multiple detour paths in a road network, the SAMAMD algorithm is used to optimize work zone lengths and schedule the resurfacing work. Analyses based on the CORSIM simulation, developed by the Federal Highway Administration, are used not only to estimate delay cost, but also to evaluate the effectiveness of optimization models. Finally, conclusions about this work and the opportunities for future research are discussed in Chapter 7. Table 1.1 shows which cases and models are developed in various sections of this dissertation.

Table 1.1 Organization of Dissertation

Traffic Pattern	Detour Type	Methodology	Traffic Management Plan	Chapter	Case
Steady Traffic Inflows	SD	Analytical Method	UA	Chapter 3	Case 1
Time-Dependent Inflows		SAUASD		Chapter 4, 5	Case 2
		SAMASD	MA	Chapter 5	
	MD	SAUAMD	UA	Chapter 6	Case 3
		SAMAMD	MA		
		Simulation			

SA: Simulated Annealing
UA: Uniform Alternatives
MA: Mixed Alternatives
SD: Single Detour
MD: Multiple Detour Paths

Chapter II Literature Review

The literature review consists of several sections. The first section identifies and summarizes the main issues for the analysis of work zones. The second section focuses on the work zone cost items that are important and sensitive to work zone configurations. Research trends for work zones and optimization algorithms are then discussed.

2.1 Work Zone Issues

Work zone studies have considered various aspects of work zone configurations. Work zone issues include (1) capacity estimation for work zones, (2) work zone travel speed estimation, (3) delay estimation, (4) maximum queue length estimation, (5) work zone safety models, (6) optimization of work zone lengths, (7) scheduling of work zone activities, (8) resurfacing procedures, and (9) work zone cost estimation. The main variables considered in these studies are traffic volumes, work zone capacity, availability of alternate roads, road types, work zone configurations, work zone length, work time, and work intensity.

These issues are directly related to the development of cost functions for analyzing work zones. Capacity estimation and work zone travel speed estimation are issues that many early work zone studies have focused on. Delay estimation and queue length estimation methods have been developed and used to analyze traffic disruptions and to determine the maximum feasible work intensity. Recently, work zone studies have sought to develop safety models that can predict the frequencies of accidents according to work zone configurations.

Optimizing work zone lengths is an important issue that has been relatively neglected. In general, longer zones tend to increase the user delays, but the maintenance activities can be performed more efficiently (i.e., with fewer repeated setups) in longer zones (Schonfeld and Chien, 1999). Such lengths have been usually designed to minimize costs for highway agencies and users.

Meanwhile, highway agencies have developed associated regulations to design work zone configurations to improve workers' and users' safety. Related regulations about scheduling maintenance work have also been developed to enhance public awareness and to decrease traffic disruption in peak periods.

Highway maintenance issues concern transportation engineers, structural engineers and construction management engineers, with different groups focusing on different aspects.

2.2 Work Zone Cost Items

Work zone costs may be classified into two categories: (1) agency costs and (2) user costs. Agency costs are those expenses required to finish the work zone activities based on the work types. Those normally include labor costs, equipment costs, material costs and traffic maintenance costs.

Meanwhile, user costs can be classified into (1) user delay costs and (2) safety (accident) costs. Since delays and accidents due to work zone activities are very important in optimizing work zone lengths and schedules, researchers have tried several methods to properly estimate the user delay and safety costs (McCoy and Peterson, 1987; Schonfeld and Chien, 1999; Venugopal and Tarko, 2000; Chien and Schonfeld, 2001;

and Chien et al., 2002). User costs have received such attention in work zone analysis because they tend to dominate other costs and because community concerns and reactions to work zone activities affect many aspects of work zone decisions.

2.3 Research Trends

1. Work Zone Capacity

Krammes and Lopez (1994) provided recommendations for estimating the capacity of the remaining lanes during short-term lane closures based on 45-hour capacity counts between 1987 and 1991 at 33 Texas freeway locations with work zones. Adjustments were suggested for the effects of the intensity of work zone activities, percentage of heavy vehicles in the traffic stream, and presence of entrance ramps near the beginning of a lane closure. Dudek and Richards (1982) presented more detailed information based on field data analysis for estimating road capacity during maintenance work. They considered lane closure strategies and obtained cumulative distribution of observed work zone capacities. In a later study (Dudek et al., 1986), they estimated capacities for work zones on four-lane highways.

Memmott and Dudek (1984) used a regression model to estimate the mean capacity for a work zone. The advantage of using the regression model was that most lane closure types were covered and the restricted capacity used for traffic management purposes could be estimated. However, they only used a capacity estimation risk factor as a variable instead of specifying other possible geometric variables. Kim and Lovell (2001) developed a multiple regression model to estimate capacity in work zones in order to establish a functional relationship between work zone capacity and several key independent factors, including the number of closed lanes, the proportion of heavy vehicles, grade and the intensity of work activities.

2. Speed and Delay

Since the travel delays of roadway users in a work zone are the primary determinant of user delay cost, studies related to speed and delay analysis for work zones have been reviewed. In a study of traffic characteristics on Illinois freeways with lane closures, Rouphail and Tiwari (1985) evaluated the effects of intensity and location of construction and maintenance activities on mean speeds through a work zone. The results showed that the mean speeds through a work zone decrease as the intensity of construction and maintenance activities increase. The mean speeds also decrease as the construction and maintenance activities move closer to the travel lanes.

Pain et al. (1981) provided a detailed study of speeds in work zones. The mean speeds were found to vary depending on such factors as traffic volumes (e.g., in peak and off-peak hours), lane closure configurations (e.g., right lane closure, left lane closure, and a two-lane bypass), traffic control devices (e.g., cones, tubular cones, barricades, and vertical panels) and locations within work zones. Rouphail et al. (1988) derived various mean values and coefficients of variation to describe the speed change in work zones. They found that the average speed does not vary considerably at light traffic volumes and that the speed recovery time is longer at high traffic volumes. Their results also indicated that speed control has a very important role in reducing accident frequency.

Memmott and Dudek (1984) developed a computer model, called Queue and User Cost Evaluation of Work Zone (QUEWZ), to estimate the average speed in work zones and calculate user costs, including user delays costs and vehicle operating costs. The effects of different lane-closure strategies and the number of hours available for lane closures are determined based on an assumed lane capacity and various traffic volumes. However, that model does not consider any alternate path and the effect of diverting traffic to it.

Jiang (1999) developed a traffic delay model to estimate work zone delay costs based on traffic data collected at work zones on Indiana's freeways. The delays related to work zones were classified into four categories: (1) deceleration delay by vehicle deceleration before entering a work zone, (2) moving delay by vehicles passing through work zones with lower speed, (3) acceleration delay by vehicle acceleration after exiting work zone, and (4) queuing delay caused by the ratio of vehicle arrival and discharge rates. In addition to the user delay generalized as queuing delay and moving delay considered by others (Cassidy and Bertini, 1999, Schonfeld and Chien, 1999, Chien and Schonfeld, 2001), Jiang also considered deceleration and acceleration delays to users.

3. Delay and Queue Length

Cassidy and Han (1994) used empirical data to estimate vehicle delays and queue lengths on two-lane highways operating under one-way traffic control. However, the work zone length was not optimized in that study.

Jiang (2001) developed a queue estimation method to calculate traffic delay using queue-discharge rates instead of work zone capacity because author noted that queue-discharge rates are lower than work zone capacity (Jiang, 1999).

4. Models for Optimizing Work Zone Length and Safety

McCoy et al. (1980) developed a method to optimize the work zone length by minimizing the road user and traffic control costs in construction and maintenance zones of rural four-lane divided highways. This method provided a framework for optimizing the lengths of work zones by minimizing the total costs, including construction costs. The user delay costs were modeled based on average daily traffic (ADT) volumes, while the accident costs were computed by assuming that the accident rate per vehicle mile was constant in a work zone area. The optimal work zone length was derived based on 1979 data. Because the unit cost factors had changed considerably since 1981, McCoy and Peterson (1987) found the optimum work zone lengths to be about 64% longer that those used previously. They (1987) also conducted a safety study for various lengths of work zones on four-lane divided highways. No relation was found between the lengths of work zones and accident rates or any of the speed distribution parameters, such as the standard deviation of vehicle speeds and the range of vehicle speeds. They also found the average accident rate was 30.8 accidents per 100 million vehicle miles (acc/100 mvm) on I-80 in Nebraska between 1978 and 1984.

Considering traffic safety in construction and maintenance work zones, Pigman and Agent (1990) conducted a statewide work zone analysis. The accident data were collected from the Kentucky Accident Reporting System (KARS) for the 1983-1986 periods. They found that the work zone accident rate varied from 36 to 1,603acc/100 mvm on different highways.

Some efforts to mitigate the impacts of work zones have been made by Janson et al (1987). One of such efforts optimized work zone traffic control design and practice considering such aspects as optimal design of control devices, optimal lane closure configuration and optimal work zone length. Martinelli and Xu (1996) added the vehicle queue delay costs into McCoy's (1980) model. The work zone length was optimized by minimizing the total user cost, excluding the maintenance and accident costs. To estimate the roadway maintenance costs, Underwood (1994) analyzed the work duration and the maintenance cost per 10,000 m^2 for five different roadway maintenance activities (i.e., surface dressing, asphalt surface, porous asphalt, 10% patching, and milling out). The average maintenance costs were calculated based on prices quoted to highway authorities in the summer of 1993.

Chien and Schonfeld (2001) developed a mathematical model to optimize the work zone lengths on four-lane highways using a single-lane closure approach. The objective of the study was to minimize the total cost including agency cost, accident cost and user delay cost based on two steady traffic inflows. They did not consider alternate paths and assumed uniform traffic flow. Viera-Colon (1999) developed a similar model of four-lane highways which considered the effect of different traffic conditions and an alternate path. However, that study did not develop alternative selection guidelines for different traffic flows or road characteristics.

Schonfeld and Chien (1999) also developed a mathematical model to optimize the work zone lengths plus associated traffic control for two-lane, two-way highways where one lane at a time is closed. They found the optimal work zone length and cycle time for traffic control and minimized the total cost, including agency cost and user delay cost, but no alternative routes were considered in that study.

5. Scheduling Work Zone Activities

Fwa, Cheu, and Muntasir (1998) developed a traffic delay model and used genetic algorithms to minimize traffic delays subject to constraints of maintenance operational requirements. Pavement sections, work teams, and start time and end for each section were scheduled. However, many conditions in that study were given, e.g. work zone configuration and available work duration for each team, and road section length. These variables were not optimized in that study. Chang, Sawaya, and Ziliaskopoulos (2001) used traffic assignment approaches to evaluate the traffic delay caused by work zones and a Tabu Search methodology was employed to select the schedule with the least total traffic delays, which include the impact of work zone combinations on an urban street network. Chang considered impact of network delay for urban areas while Fwa's research neglected the impact of network delay due to detours.

Chien, Tang, and Schonfeld (2002) developed a model to optimize the scheduling of work zone activities associated with traffic control for two-lane two-way highways where one lane at a time is closed. However: (1) the traffic pattern used in that research was simplified into four traffic volumes during four period in a day: morning peak, daytime, evening peak, and nighttime periods, which could not fully reflect the real traffic situation, (2) the search approach to determine each zone length is a greedy method, whose results may be sub-optimal, and (3) the effects of highway networks on work zone characteristics were not considered. Jiang and Adeli (2003) used neural networks and simulated annealing to optimize only one work zone length and starting time for a four-lane freeway, considering factors such as darkness and numbers of lanes closed. More complete scheduling plans for multiple-zone maintenance projects were not attempted in that work.

6. Construction Congestion Cost

Carr (2000) developed a construction congestion cost (CO^3) system to estimate the impact of traffic maintenance contract provisions on congestion, road user cost, and construction cost. CO^3 was implemented in a Microsoft Excel spread sheet and consists of three sheets: (1) route sheet computing equivalent average vehicle routes for complex diversion routes, (2) input sheet providing for documentation of vehicle and route inputs and computing user cost for single trips through the work zone, diversions, and cancellations, and (3) traffic sheet computing daily traffic impacts and user costs for each construction method. Although CO^3 provides practical information with which engineers select construction methods, it does not optimize work zone configurations.

7. QuickZone Software for Work Zones

The 1998 FHWA report “Meeting the Customer’s Needs for Mobility and Safety During Construction and Maintenance Operations” recommends the development of an analytical tool to estimate and quantify work zone delays. This scope of work lays out a plan for the development of an easy-to-master analytic tool (currently under the working title "QuickZone") for quick and flexible estimation of work zone delay. The primary functions of QuickZone include quantification of corridor delay resulting from capacity decreases in work zones, identification of delay impacts of alternative project phasing plans, supporting tradeoff analyses between construction costs and delay costs, examination of impacts of construction staging, by location along mainline, time of day (peak vs. off-peak) or season, and assessment of travel demand measures and other delay mitigation strategies. The costs can be estimated for both an average day of work and for the whole life cycle of construction. However, there is no optimization function in Quickzone.

The Maryland State Highway Administration and the University of Maryland (Kim and Lovell, 2001) used QuickZone's open source code to customize the program to meet the State's needs. The University has added its own capacity estimation model to the program and has used a 24-hour traffic count, instead of the average daily traffic count found in original version. FHWA and Maryland’s Quickzone versions provide a useful to estimate work zone delay; however, there was still no optimization model in these programs.

8. Simulation Modeling for Work Zones

CORSIM (Corridor Simulator) is a microscopic simulation model developed by the Federal Highway Administration (FHWA) and can simulate coordinate traffic operations on surface streets and freeways. Generally, work zone delays occurring in a single road section or simple road network can be derived from deterministic queuing theory; however, with a simulation method such as CORSIM, it is much easier to estimate work zone delays in a more complex road network. Nemeth and Rathi (1985), Cohen and Clark (1996), and Chien and Chowdhury (1998) used CORSIM to study velocity and analyze capacity for freeway operations. CORSIM can be adapted to simulate traffic operations around a work zone by assuming one more lane closure for a work zone as the lane closure caused by an incident. Schrock and Maze (2000) developed a work zone simulation model and used CORSIM to evaluate four alternatives for work zones along Interstate 80 in Iowa. The simulation model was developed as a planning tool to determine the potential benefits of alternative traffic management plans at a long-term work zone.

Maze and Kamyab (1999) used ARENA, a simulation model with an advanced animation module, to develop a work zone simulation model, including car-following and lane-changing algorithms, to estimate work zone delays. That study only applied ARENA for a work zone in a single road. No detours or road networks were considered.

2.4 Optimization Algorithms

When work zone optimization is based on steady traffic inflows, the optimization result can often be obtained directly with an analytic method. When time-dependent inflows or multiple detour networks are considered, the cost functions will become more complex and thus more complex algorithms are needed for large optimization problems.

Optimization techniques such as genetic algorithms (GA), simulated annealing (SA), and tabu search (TS) are widely used in combinatorial optimization problems (COP), where the objective is to choose a best solution out of a large number of possible solution, and obtain very good results in NP-hard combinatorial optimization problems. These three probabilistic heuristic methods share two main characteristics. One is that these three algorithms are inspired by real phenomena in physics, biology, and social science. The other is that they use a certain amount of repeated trials to find the optimal or near optimal solution (Colorni et al., 1996). Pham and Karaboga (2000) found that GA performs better than TS and SA for the traveling salesman problem. Sadek et al. (1999)

used SA and GA to solve a dynamic traffic routing problem and found that SA tends to perform better than GA. Nalamottu et al. (2002) compared GA to SA in solving transportation location-allocation problems and found SA to be better than GA in its convergence to exact solutions and its computation time. Zolfaghari and Liang (2002) compared GA, SA, and TS in terms of solution quality, search convergence behavior and presearch effort for solving binary comprehensive machine-grouping problems. Their results indicated that SA outperforms both GA and TS, particularly for large problems.

Recently, hybrid methods combining these three algorithms were developed for combinatorial optimization problems (Liu et al., 2000, Adamopoulos et al, 1998). A hybrid method combines the advantages of each algorithm. For example, Liu et al. combined the advantages of GA, SA, and TS to solve the reactive power optimization problem. They adopt the acceptance probability of SA to improve the convergence of the GA, and apply TS to find more accurate solutions.

Generally, it is recognized that GA's are not well suited for finely tuned local search. However, after promising regions of the source space are identified by the GA, it may be useful to invoke a local search routine to optimize the members of the final population (Grefenstette, 1987). SA has been proven effective for the optimal or near-optimal solution for a local regional search (Pham and Karaboga, 2000). Li et al. (2002) used GA to generate a group of initial solutions and then used SA to search the local optimum for solving machine operation process plans. Colorni et al. (1996) concluded that SA has a "well-defined" advantage with likely lower future developments, and TS and GA have a "dynamic" advantage with large possibilities of novel research for theories and results.

In view of the above literature review, there are two main reasons why SA is applied in this study for work zone optimization problems. First, SA is more completely developed and provides more finely tuned results than other two methods for combinatorial optimization problems. Second, the methodology in Case 1 will be applied to generate the initial solutions for Case 2 and Case 3. From the research flow of this study, the results of Case 1 for steady traffic inflows are the fundamentals of Case 2 for time-dependent inflows and of Case 3 for multiple detour networks. Then SA can be used to seek a global or near global optimum by using the initial solution obtained by the methodology in Case 1. Due to these characteristics, SA will be applied to solve work zone optimization problems in this study.

The SA approach was derived from statistical mechanics for finding near optimal solutions to large optimization problems. Simulated annealing was developed by Metropolis (1953) when it was used to simulate the annealing process of crystals on a computer. Kirkpatrick et al. (1983) generalized an approach by introducing a multi-temperature approach in which the temperature is lowered slowly in stages. Kirkpatrick et al. applied this methodology to solve the problems of combinatorial optimization, especially the problems of wire routing and the component placement in VLSI (Very Large Scale Integration) design.

SA is sensitive to a number of control parameters and stopping rules (Wilhelm and Ward, 1987). The algorithm has potential to find high-quality solutions but at the cost of substantial computational efforts (Aarts and Korst, 1989). For example, if the initial temperature is too high and the cooling schedule is very slow, the cooling will takes long computational time to approach final temperature. However, it is inefficient even the solution has high quality. If the initial temperature is too low and the cooling schedule is too fast, the solution may not be close to the optimum. Therefore, the cooling schedule should be chosen carefully.

SA is widely used in transportation related research. Hadi and Wallace (1994), Oda et al. (1997), and Lee and Machemehl (1997) used SA to solve signal phasing and timing optimization problems. Taniguchi et al. (1999) and Kokubugata et al. (1997) applied SA to find optimal assignment for vehicle routing and scheduling problems. Chang (1994) used SA to solve flight sequencing and gate assignment problems.

For the work zone optimization problem, Jiang and Adeli (2003) used neural networks and simulated annealing to optimize work zone length and starting time for a four-lane freeway. Only one zone length and starting time are optimized in that study. More complete scheduling plans for multiple-zone maintenance projects are needed in practice.

2.5 Summary

After a review the above studies, it appears that work zone capacity, delays, work zone length, and costs have already been developed for steady traffic inflows and partially for time-dependent inflows. However, further research on work zone optimization with detours, including a single detour and multiple detour paths, for both steady and time-dependent inflows is quite necessary and important for the development of practical work zone project scheduling and traffic management plans.

Some analytical and heuristic methods were proposed for solving work zone optimization problems in the above studies; however, those studies did not present complete results for steady and time-dependent inflows, with and without detour(s). No comprehensive method has been previously developed to jointly analyze the work zone optimization problem. Therefore, this study will focus on the work zone optimization methods for steady and time-dependent inflows with a single detour and with multiple detour paths.

Chapter III Work Zone Optimization for Steady Traffic Inflows

In this chapter, work zone optimization models for steady traffic inflows are developed for two-lane highway and four-lane highway work zones. The highway system and various work alternatives are defined in Section 3.1. Analytical optimization models are developed for two-lane highway and four-lane highway work zones in Sections 3.2 and 3.3. Sections 3.4 and 3.5 show the speeds along work zones and detours are determined and how the threshold analysis is conducted. Finally, numerical results for two-lane and four-lane highways are shown in Sections 3.6 and 3.7.

3.1 Highway System Definition

In this study highway types are classified into two-lane two-way highways and multiple-lane two-way highways. Two-lane two-way highways often require closing one lane for a work zone. In such circumstances, vehicles travel in the remaining lane along the work zone, alternating direction within each control cycle. Such a two-lane work zone can be considered as a one-way traffic control system in which queuing and delay processes are analogous to those at a two-phase signalized intersection.

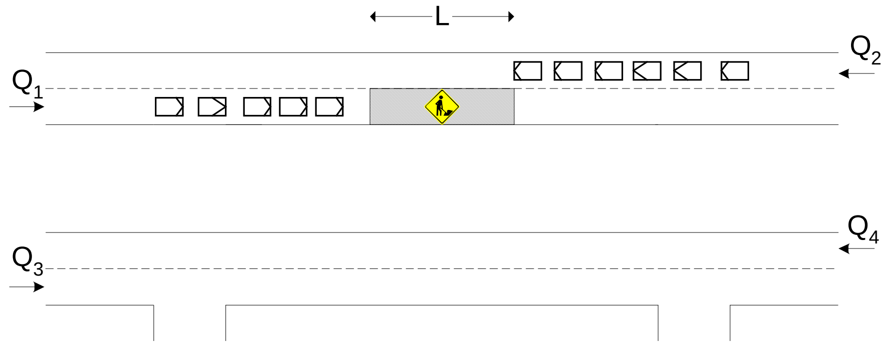
Pavement maintenance on multiple-lane two-way highways often requires closing one or two lanes to set up a work zone. This does not require alternating one-way control as in a two-lane highway work zone because at least one lane is usually still available in the direction of closure. Because work zones in two-lane highways and multiple-lane highways have different delay and queuing patterns, the work zone cost functions are separately developed.

Several work zone alternatives of two-lane highways and multiple-lane highways are demonstrated as follows:

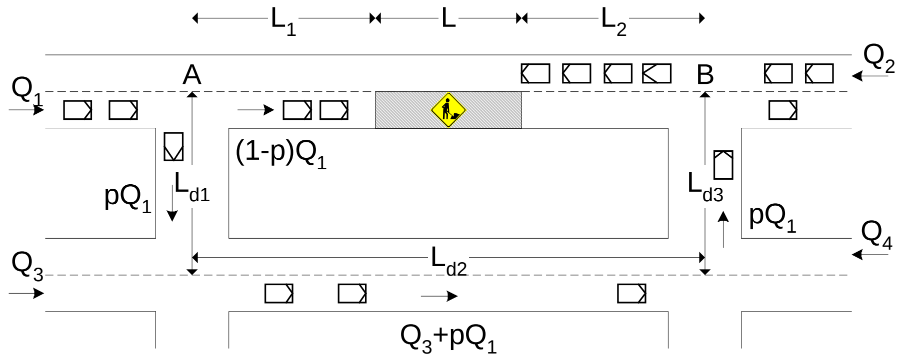
1. Two-Lane Two-Way Highway Work Zone

Schonfeld and Chien (1999) analyzed the effect of longer work zones and cycle times in increasing the user delay and decreasing the total maintenance time and costs due to fewer setups for fewer zones. Note that this case in which traffic flows from both directions are alternated on one lane, without any detour, is considered the first alternative for two-lane roads, labeled Alternative 2.1. The geometries of all alternatives are shown in Figure 3.1.

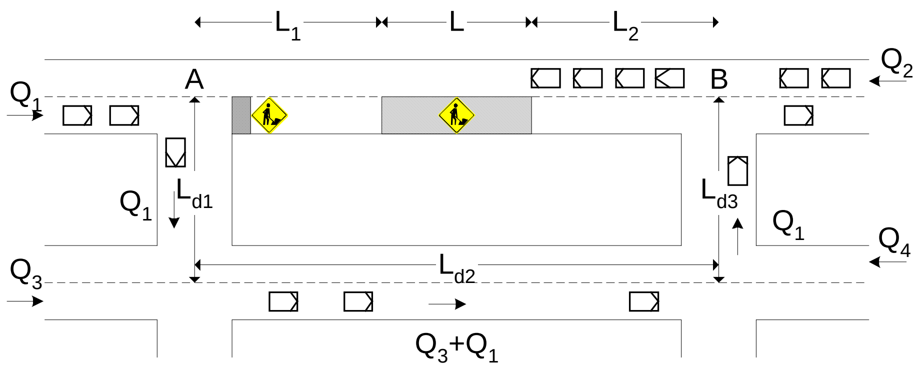
In the second alternative, we consider the best available alternate route that bypasses the work zone area, so that the original traffic flow on the road is divided between the flow passing along the work zone and the flow through the detour. Thus, in the second alternative considered, the remaining lane is still used for alternating two-way traffic, but traffic from the maintained road also can use the alternate route. In the third alternative all traffic in one direction is diverted to the alternate route, while the remaining lane is only used for traffic in the other direction. Thus, the diverted traffic percentage from one direction of the main road is 0% in Alternative 2.1, 100% in Alternative 2.3 and somewhere between those extremes in Alternative 2.2. In Alternative 2.4, all traffic in both directions is diverted to the alternate route and both lanes are closed for work. The preferred alternative can be determined after evaluating all four alternatives.



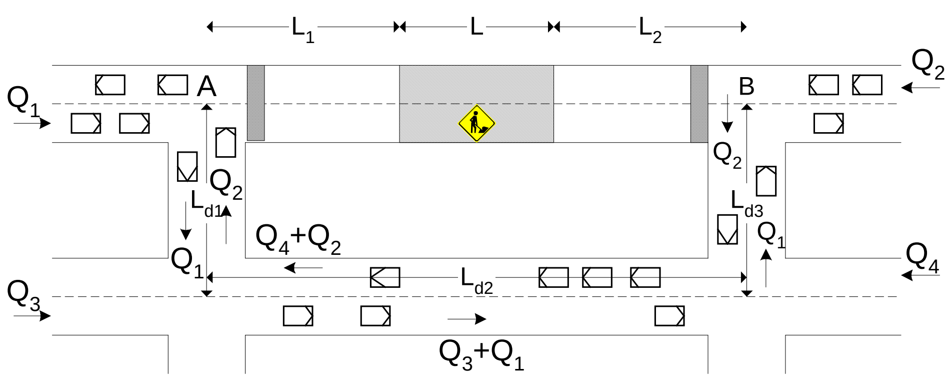
(a) Alternative 2.1: without Detour



(b) Alternative 2.2: with Detour; pQ_1 on Detour, $(1\text{-}p)Q_1$ along Work Zone



(c) Alternative 2.3: Detour for Only One Direction



(d) Alternative 2.4: Two Directions Detoured

Figure 3.1 Geometries of Analyzed Work Zones for Two-Lane Two-Way Highways

2. Multiple-Lane Two-Way Highway Work Zone

Pavement maintenance on multiple-lane, two-way highways usually requires closing one or two lanes to set up a work zone. Chien and Schonfeld (2001) developed a work zone cost function (accounting for user delays, accidents, and agency costs) for four-lane two-way highways without considering detours. That case in which one of the two lanes in one direction is closed, without any detour, is considered Alternative 4.1, as shown in Figure 3.2(a). Here, four-lane highways are classified as “multiple-lane” highways.

Here we consider the best available alternate route that bypasses the work zone area, so that the original flow, Q_1, in Direction 1 on the road is divided between the flow passing along the work zone and the flow through the detour, as shown in Figure 3.2(b). Thus, in Alternative 4.2 one lane in Direction 1 is closed, while the remaining lane in Direction 1 is still usable, but traffic in Direction 1 can also use the alternate route. In Alternative 4.3 all traffic in Direction 1 is diverted to the alternate route since both lanes are closed, as shown in Figure 3.2(c). Thus, the diverted traffic percentage from Direction 1 is 0% in Alternative 4.1, 100% in Alternative 4.3 and somewhere between those extremes in Alternative 4.2. In Alternative 4.4, both lanes in Direction 1 are closed for a work zone and all traffic in Direction 1 crosses over to one lane in the opposite direction, as shown in Figure 3.2(d). The preferred alternative can be again determined here after evaluating all four alternatives.

In this chapter a methodology is proposed for minimizing the total cost, including agency cost, user delay cost, and accident cost, and to optimize the work zone length for each alternative, while considering the best available alternate route that bypasses the work zone. Guidelines for determining the best alternative for different conditions of traffic flow, road characteristics (i.e. detour length, the distance of main road between the beginning and end of detour) and maintenance characteristics (i.e. maintenance setup cost, average maintenance time per kilometer) are developed in the following sections by deriving the minimum cost thresholds between pairs of alternatives with respect to key variables.

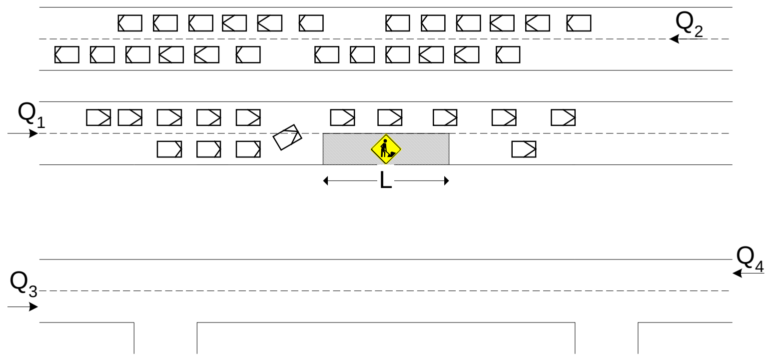
3.2 Work Zone Optimization - Two-Lane Two-Way Highway

The basic method followed here for tow-lane two-way highway and four-lane two-way highway is to formulate a total cost objective function and use it to optimize work zone lengths at work zones for four alternatives. The queuing delays to users are formulated with deterministic queuing models. Then thresholds among alternatives are derived with respect to key variables, to determine the best alternative for different conditions of traffic flow, road characteristics and maintenance characteristics.

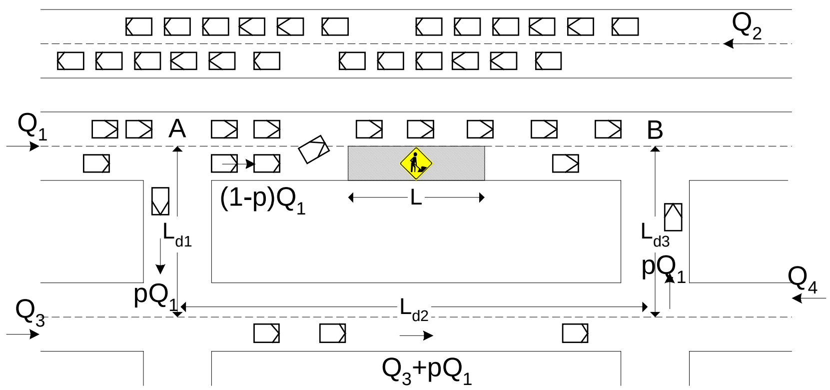
3.2.1 Alternatives and Assumptions

The following four alternatives are considered for two-lane two-way highways in this study:

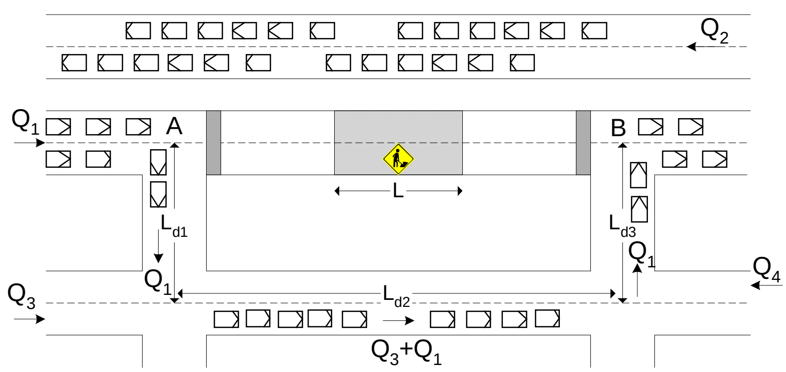
1. Alternating flow on one lane, without any detour
2. Alternating flow on one lane, with a detour
3. One-directional flow on one lane along work zone; other direction on detour
4. Both directions detoured and both lanes closed for work



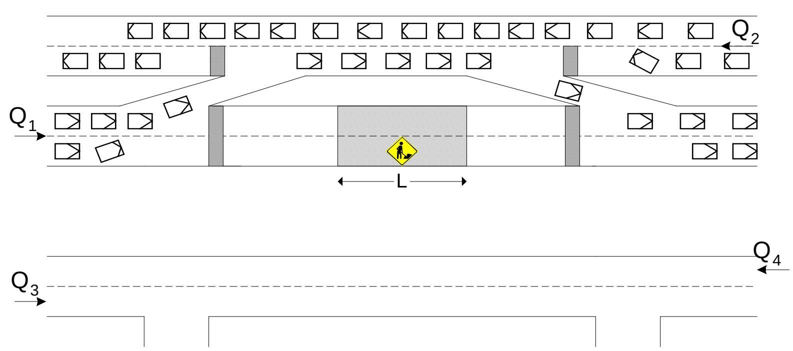
(a) Alternative 4.1: No Detour, One of the Two Lanes closed for Q_1 Traffic



(b) Alternative 4.2: A Fraction of Q_1 Traffic through Detour



(c) Alternative 4.3: All Q_1 through Detour, Allowing Work Zone on Both Lanes in Direction 1



(d) Alternative 4.4: Crossover of All Q_1 into One Lane in Opposite Direction, Allowing Work Zone on Both Lanes in Direction 1

Figure 3.2 Geometries of Analyzed Work Zones for Four-Lane Two-Way Highways

The geometries of these four cases are shown in Figure 3.1.

Several simplifying assumptions made in formulating this problem are listed below.

1. Traffic moves at a uniform speed through a work zone and at a different uniform speed elsewhere.
2. The effects on speeds of the original detour flows on the relatively short L_{d1} and L_{d3} in Figures 3.1 are negligible.
3. Queues in both directions will be cleared within each cycle for two-lane two-way highways. Thus, the one-lane work zone capacity exceeds the combined flows of both directions.
4. Possible signal or stop sign delays on the detour in Alternatives 2.2, 2.3, and 2.4 may be neglected.
5. Queue backups to the maintained road along the first detour L_{d1} may be neglected.
6. The detour capacity always exceeds the original detour flow plus diverted flow, so queue delay on the detour may be neglected.
7. The value of user time used in numerical analysis is the weighted average cost of driver and passenger's user time for cars and trucks. In this study vehicle operation costs are not considered separately but may be accounted for in the value of user time.

3.2.2 Model Formulation

Work zone cost functions of four alternatives for two-lane highways are formulated in this section. Alternative 2.1 is based on the study by Schonfeld and Chien

(1999) but the model is modified by adding moving delay cost along work zone and accident cost. Other alternatives, Alternatives 2.2, 2.3, and 2.4, are developed here as extensions of Alternative 2.1 by considering an alternate route.

Alternative 2.1: Flow on one lane without detour

Schonfeld and Chien (1999) developed a work zone cost function which includes user delay cost and maintenance cost:

$$C_T = C_M + C_U \tag{3.1}$$

where C_T = total cost per lane-kilometer; C_M = maintenance cost per lane-kilometer; C_U = user delay cost per lane-kilometer.

The user delay cost consists of the queuing delay costs due to a one-way traffic control and the moving delay costs through work zones. The queuing delay cost C_q per maintained lane-kilometer is the total delay per cycle Y in both directions multiplied by the number of cycles N per maintained lane-kilometer and the users' value of time v (in $/veh-hr):

$$C_q = Ynv \tag{3.2}$$

where Y = summation of the delays (e.g., Y_1 and Y_2) incurred by the traffic flows from directions 1 and 2 per cycle. Y_1 and Y_2 can be derived by using deterministic queuing analysis. Schonfeld and Chien (1999) formulated the zone delay cost without any alternate route around the work zone and obtained the following relation:

$$C_q^{21} = \frac{(z_3 + z_4 L)[Q_1(\frac{3600}{H} - Q_1) + Q_2(\frac{3600}{H} - Q_2)]v}{V(\frac{3600}{H} - Q_1 - Q_2)} \tag{3.3}$$

where C_q^{21} = queuing delay cost per lane-kilometer for Alternative 2.1; z_3 = setup time; z_4 = average maintenance time per lane-kilometer; L = work zone length; Q_1 = hourly flow rate in Direction 1; Q_2 = hourly flow rate in Direction 2; H = average headway; V = average work zone speed; v = value of user time; and z_3+z_4L represents the maintenance duration per zone.

Eq.(3.3) represents the queuing delay cost due to one-way traffic control, as proposed by Schonfeld and Chien (1999). Here we consider moving delay cost through work zone. The moving delay cost of the traffic flows Q_1 and Q_2, denoted as C_v^{21}, is the cost increment due to the work zone. It is equal to the flow ($Q_1 + Q_2$) multiplied by: (1) the average maintenance duration per kilometer, $\frac{z_3}{L} + z_4$, (2) the travel time difference over zone length with the work zone, $\frac{L}{V}$, and without the work zone, $\frac{L}{V_0}$, and (3) the value of time, v. Thus:

$$C_v^{21} = (Q_1 + Q_2)(\frac{z_3}{L} + z_4)(\frac{L}{V} - \frac{L}{V_0})v \tag{3.4}$$

where V_0 represents the speed on the original road without any work zone.

The user delay cost for Alternative 1 C_U^{21} is equal to the sum of queue delay cost C_q^{21} and moving delay cost C_v^{21}.

The accident cost incurred by the traffic passing the work zone can be determined from the number of accidents per 100 million vehicle hours n_a multiplied by the product of the increasing delay ($C_q^{21}/v + C_v^{21}/v$) and the average cost per accident v_a (Chien and Schonfeld, 2001). The average accident cost per lane-kilometer C_a^{21} is formulated as:

$$C_a^{21} = [\frac{(z_3 + z_4 L)[Q_1(\frac{3600}{H} - Q_1) + Q_2(\frac{3600}{H} - Q_2)]}{V(\frac{3600}{H} - Q_1 - Q_2)} + (Q_1 + Q_2)(\frac{z_3}{L} + z_4)(\frac{L}{V} - \frac{L}{V_0})]\frac{n_a v_a}{10^8} \quad (3.5)$$

The maintenance cost per zone is assumed to be z_1+z_2L, where z_1 = fixed setup cost; and z_2 = average maintenance cost per additional lane-kilometer. The average maintenance cost per lane-kilometer, C_M, is the total maintenance cost per zone divided by the zone length L:

$$C_M = (z_1 + z_2 L)/L = \frac{z_1}{L} + z_2 \quad (3.6)$$

Then the total cost for Alternative 2.1, C_T^{21}, is $C_M + C_U^{21} + C_a^{21}$. Its optimized work zone length of Alternative 1, L^{*21}, obtained by setting the partial derivative of the total cost function C_T^{21} with respect to L equal to zero and solving for L, is:

$$L^{*21} = \sqrt{\frac{\frac{z_1}{v + \frac{n_a v_a}{10^8}}}{\frac{z_4[Q_1(\frac{3600}{H} - Q_1) + Q_2(\frac{3600}{H} - Q_2)]}{V(\frac{3600}{H} - Q_1 - Q_2)} + (Q_1 + Q_2) z_4 (\frac{1}{V} - \frac{1}{V_0})}} \quad (3.7)$$

The second derivative of C_T^{21} with respect to L is positive in this case and the following ones, indicating that function is convex and has a unique global minimum for L.

Alternative 2.2: Flow on one lane as well as a detour

It is assumed in Alternative 2.2 (Figure 3.1(b)) that the fraction p of the flow Q_1 in Direction 1 is diverted to the alternate route. Then the user queuing delay cost of the remaining flow in Direction 1, $(1-p)Q_1$, and Q_2, denoted as C_q^{22}, has the same formulation as Eq.(3.3) but with $(1-p)Q_1$ substituted for Q_1.

$$C_q^{22} = \frac{(z_3 + z_4 L)[(1-p)Q_1(\frac{3600}{H} - (1-p)Q_1) + Q_2(\frac{3600}{H} - Q_2)]v}{V(\frac{3600}{H} - (1-p)Q_1 - Q_2)} \tag{3.8}$$

The user moving delay cost of the remaining traffic flow in Direction 1, $(1-p)Q_1$, and Q_2, denoted as $C_{v(1-p)2}^{22}$, is the cost increment due to the work zone. It has the same formulation as Eq.(3.4) but with $(1-p)Q_1+Q_2$ substituted for Q_1+Q_2.

$$C_{v(1-p)2}^{22} = ((1-p)Q_1 + Q_2)(\frac{z_3}{L} + z_4)(\frac{L}{V} - \frac{L}{V_0})v \tag{3.9}$$

The user moving delay cost of the diverted flow pQ_1 from Direction 1, denoted as C_{vp}^{22}, is equal to the flow pQ_1 multiplied by: (1) the average maintenance duration per kilometer, $\frac{z_3}{L} + z_4$, which is the maintenance duration per zone, z_3+z_4L, divided by work zone L, (2) the time difference between the time vehicles through the detour, $\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}}$, and the time vehicles through the maintained road AB without work zone, $\frac{L_t}{V_0}$, and (3) the value of time, v. Thus:

$$C_{vp}^{22} = pQ_1(\frac{z_3}{L} + z_4)[\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0}]v \tag{3.10}$$

where L_{d1}, L_{d2}, L_{d3} are the lengths of the first, second and third segments of the detour shown in Figure 3.1. V_0 represents the speed on the maintained road without any work zone and V_d^{*3} is the detour speed affected by diverted traffic in Direction 3 in Alternative 2.2. Both speeds are computed with Eq.(3.81), derived below in Section 3.5.

In addition to delay costs of flows remaining on the maintained road, the moving delay cost to the original flow on the detour, Q_3, as affected by the pQ_1, is also considered. Denoted as C_{v3}^{22}, it equals the flow Q_3 multiplied by: (1) the average maintenance duration per kilometer, $\frac{z_3}{L}+z_4$, (2) the travel time difference over L_{d2} with the diverted flow pQ_1, $\frac{L_{d2}}{V_d^{*3}}$, and without it, $\frac{L_{d2}}{V_{d0}}$, and (3) the value of time, v. Thus:

$$C_{v3}^{22} = Q_3(\frac{z_3}{L}+z_4)(\frac{L_{d2}}{V_d^{*3}}-\frac{L_{d2}}{V_{d0}})v \tag{3.11}$$

where V_{d0} represents the original speed on L_{d2} unaffected by pQ_1.

The combined user delay cost for the maintained road AB and the detour can be derived as:

$$C_U^{22} = C_q^{22} + C_{v(1-p)2}^{22} + C_{vp}^{22} + C_{v3}^{22} \tag{3.12}$$

The accident cost per maintained kilometer for, C_a^{22}, is:

$$C_a^{22} = \frac{(C_q^{22} + C_{v(1-p)2}^{22} + C_{vp}^{22} + C_{v3}^{22})}{v}\frac{n_a v_a}{10^8} \tag{3.13}$$

Then the total cost for Alternative 2.2, C_T^{22}, is $C_M + C_U^{22} + C_a^{22}$. Its optimized work zone length L^{*22} is obtained by setting the partial derivative of C_T^{22} with respect to L equal to zero and then solving for L. This yields:

$$L^{*22} = \sqrt{\frac{\dfrac{z_1}{v+\dfrac{n_a v_a}{10^8}} + pQ_1 z_3(\dfrac{L_{d1}+L_{d3}-L_t}{V_0}+\dfrac{L_{d2}}{V_d^{*3}}) + Q_3 z_3(\dfrac{L_{d2}}{V_d^{*3}}-\dfrac{L_{d2}}{V_{d0}})}{\dfrac{z_4[(1-p)Q_1(\dfrac{3600}{H}-(1-p)Q_1)+Q_2(\dfrac{3600}{H}-Q_2)]}{V(\dfrac{3600}{H}-(1-p)Q_1-Q_2)} + [(1-p)Q_1+Q_2]z_4(\dfrac{1}{V}-\dfrac{1}{V_0})}} \tag{3.14}$$

The second derivative of C_T^{22} with respect to L is also positive in this case and the following ones, indicating that function is convex and has a unique global minimum for L.

Alternative 2.3: One direction along the work zone and the other detoured

Here it is assumed that the entire flow Q_1 in Alternative 2.1 is diverted to the alternate route. Then the user moving delay cost in Direction 1, denoted as C_{v1}^{23}, has the same formulation as Eq. (3.10) but with Q_1 substituted for pQ_1.

$$C_{v1}^{23} = Q_1(\frac{z_3}{L} + z_4)[\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0}]v \tag{3.15}$$

The user moving delay cost of the traffic flow Q_2, denoted as C_{v2}^{23}, is the cost increment due to the work zone. It is equal to the flow Q_2 multiplied by: (1) the average maintenance duration per kilometer, $\frac{z_3}{L} + z_4$, (2) the time difference over section AB (in Figure 3.1(c)) with the work zone, $\frac{L_1 + L_2}{V_0} + \frac{L}{V}$, and without the work zone, $\frac{L_t}{V_0}$, and (3) the value of time, v. Thus:

$$\begin{aligned} C_{v2}^{23} &= Q_2(\frac{z_3}{L} + z_4)(\frac{L_1 + L_2}{V_0} + \frac{L}{V} - \frac{L_t}{V_0})v \\ &= Q_2(\frac{z_3}{L} + z_4)(\frac{L}{V} - \frac{L}{V_0})v \end{aligned} \tag{3.16}$$

The moving delay cost C_{v3}^{23} of the original flow Q_3 in Direction 3, as affected by the Q_1, is also considered. It has the same formulation as Eq. (3.11) but V_d^{*3} is affected by pQ_1 in Alternative 2.2 and by Q_1 in Alternative 2.3.

$$C_{v3}^{23} = Q_3(\frac{z_3}{L} + z_4)(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v \tag{3.17}$$

The total user delay cost including original road and detour can be determined as follows:

$$C_U^{23} = C_{v1}^{23} + C_{v2}^{23} + C_{v3}^{23} \tag{3.18}$$

where C_U^{23} = user delay cost per kilometer per lane for Alternative 2.3.

The accident cost per maintained kilometer for, C_a^{23}, is:

$$C_a^{23} = \frac{(C_{v1}^{23} + C_{v2}^{23} + C_{v3}^{23})}{v} \frac{n_a v_a}{10^8} \tag{3.19}$$

Then the total cost for Alternative 2.3, C_T^{23}, is $C_M + C_U^{23} + C_a^{23}$. Its optimized work zone length L^{*23} is then found to be:

$$L^{*23} = \sqrt{\frac{z_1 + [Q_1 z_3(\frac{L_{d1} + L_{d3} - L_t}{V_0} + \frac{L_{d2}}{V_d^{*3}}) + Q_3 z_3(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})](v + \frac{n_a v_a}{10^8})}{Q_2 z_4(\frac{1}{V} - \frac{1}{V_0})(v + \frac{n_a v_a}{10^8})}} \tag{3.20}$$

Because the second derivatives $\partial C_T^{21}/\partial L^2$, $\partial C_T^{22}/\partial L^2$, $\partial C_T^{23}/\partial L^2$ of all three objective functions C_T^{21}, C_T^{22} and C_T^{23} are positive, those functions are convex and L^{*21}, L^{*22} and L^{*23} are global optima.

Alternative 2.4: Both directions detoured and both lanes closed for work

Here it is assumed that the entire flows Q_1 and Q_2 are diverted to the alternate route as both lanes between A and B are entirely closed for maintenance. Then the user moving delay cost in Direction 1, denoted as C_{v1}^{24}, has the same formulation as Eq.(3.9) but with Q_1 substituted for pQ_1.

$$C_{v1}^{24} = Q_1(\frac{z_3}{L} + z_4)[\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0}]v \tag{3.21}$$

The user moving delay cost of the flow Q_2, denoted as C_{v2}^{24}, has the same formulation as Eq.(3.21) but with Q_2 substituted for pQ_1 and with V_d^{*4} substituted for V_d^{*3}.

$$C_{v2}^{24} = Q_2(\frac{z_3}{L} + z_4)[\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*4}} - \frac{L_t}{V_0}]v \tag{3.22}$$

where V_d^{*4} is the detour speed in Direction 4 affected by Q_2.

The moving delay cost C_{v3}^{24} of the original flow Q_3 in Direction 3, as affected by the Q_1, is also considered. It has the same formulation as Eq.(3.17):

$$C_{v3}^{24} = Q_3\left(\frac{z_3}{L} + z_4\right)\left(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}}\right)v \tag{3.23}$$

Similarly, the delay cost C_{v4}^{24} of the original flow Q_4 in Direction 4, as affected by the Q_2, is considered as well. It has the same formulation as Eq.(3.23) but with Q_4 substituted for Q_3 and V_d^{*4} substituted for V_d^{*3}.

$$C_{v4}^{24} = Q_4\left(\frac{z_3}{L} + z_4\right)\left(\frac{L_{d2}}{V_d^{*4}} - \frac{L_{d2}}{V_{d0}}\right)v \tag{3.24}$$

It is assumed here that Q_3 and Q_4 are equal so that the original detour speeds for Direction 3 and 4 are equal, V_{do}, Those speeds, V_{do}, will be derived in Eq.(3.81).

The accident cost per maintained kilometer for, C_a^{24}, is:

$$C_a^{24} = \frac{(C_{v1}^{24} + C_{v2}^{24} + C_{v3}^{24} + C_{v4}^{24})}{v}\frac{n_a v_a}{10^8} \tag{3.25}$$

The total user delay cost C_U^{24} can be determined as follows:

$$C_U^{24} = C_{v1}^{24} + C_{v2}^{24} + C_{v3}^{24} + C_{v4}^{24} + C_a^{24} \tag{3.26}$$

Because Alternative 2.4 is a two-lane maintenance work zone, the maintenace cost for Alternative 2.4 differs from that of other one-lane alternatives. Here we define the parameter α to be a reduction factor that is equal to the maintenance cost for two lanes divided by the maintenance cost for one lane. It allows for the possibility that resurfacing cost per lane-kilometer may decrease when two adjacent lanes are resurfaced together. The maintenace cost per lane-kilometer is equal to the maintenance cost per zone z_1+z_2L multiplied by α (for two-lane maintenance cost), and divided by (1) zone length L, (2) number of lanes, 2. The maintenance cost C_M is:

$$C_M = \frac{1}{2}\alpha\left(\frac{z_1}{L} + z_2\right) \tag{3.27}$$

In the numerical examples of this study, α is assumed to be equal to 2. Then the total cost for Alternative 2.4, C_T^{24}, is $C_M + C_U^{24}$. The first and second partial derivatives of C_T^4 are then found to be:

$$\frac{\partial C_T^{24}}{\partial L} = -[\frac{z_1}{L^2} + \frac{Q_1 z_3}{L^2}(\frac{L_{d1}+L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0})v + \frac{Q_2 z_3}{L^2}(\frac{L_{d1}+L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*4}} - \frac{L_t}{V_0})v + \frac{Q_3 z_3}{L^2}(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v + \frac{Q_4 z_3}{L^2}(\frac{L_{d2}}{V_d^{*4}} - \frac{L_{d2}}{V_{d0}})v] < 0 \tag{3.28a}$$

$$\frac{\partial^2 C_T^{24}}{\partial L^2} = 2\frac{z_1}{L^3} + 2\frac{Q_1 z_3}{L^3}(\frac{L_{d1}+L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0})v + 2\frac{Q_2 z_3}{L^3}(\frac{L_{d1}+L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*4}} - \frac{L_t}{V_0})v + 2\frac{Q_3 z_3}{L^3}(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v + 2\frac{Q_4 z_3}{L^3}(\frac{L_{d2}}{V_d^{*4}} - \frac{L_{d2}}{V_{d0}})v > 0 \tag{3.28b}$$

The first partial derivative of C_T^{24} is negative and the second partial derivative is positive. Therefore the function C_T^{24} is convex and has a unique global optimum for zone length L_t.

3.3 Work Zone Optimization - Four-Lane Two-Way Highway

3.3.1 Alternatives and Assumptions

The following four alternatives are considered for four-lane two-way highways in this study:

1. There is no detour and one of the two lanes is closed for Q_1 traffic.
2. A fraction of Q_1 traffic is diverted through detour.

3. All of Q_1 is diverted through detour, allowing work zone on both lanes in Direction 1.
4. All of Q_1 crosses over into one lane in the opposite direction, allowing work on both lanes in Direction 1.

The geometries of these four cases are shown in Figure 3.2.

Several simplifying assumptions made in formulating this problem are listed below.

1. Traffic moves at a uniform speed through a work zone and at a different uniform speed elsewhere.
2. The effects on speeds of the original detour flows on the relatively short L_{d1} and L_{d3} in Figures 3.2 are negligible.
3. Possible signal or stop sign delays on the detour in Alternatives 4.2, 4.3 may be neglected.
4. Queue backups to the maintained road along the first detour L_{d1} may be neglected.
5. The detour capacity always exceeds the original detour flow plus diverted flow, so queue delay on the detour may be neglected.

3.3.2 Model Formulation

Work zone cost functions of four alternatives for four-lane highways are formulated in this section. Alternative 4.1 is based on the study by Chien and Schonfeld (2001). Other alternatives, Alternatives 4.2, 4.3, and 4.4, are developed here as extensions of Alternative 4.1 by considering an alternate route or crossover flow to the opposite direction.

Alternative 4.1: No Detour and One of the Two Lanes closed for Q_1 Traffic

Chien and Schonfeld (2001) developed a work zone cost function, which includes the user delay, the accident, and the agency costs, for four-lane two-way highway without considering a detour (Figure 3.2(a)). The user delay cost consists of the queuing delay costs upstream of work zones and the moving delay costs through work zones. The following variables are defined:

Q_1 = approaching traffic flow in Direction 1 of work zone maintained (veh/hr)

c_w = work zone capacity (veh/hr)

D = maintenance duration per zone

If Q_1 exceeds the work zone capacity c_w, a queue forms, which then dissipates when the closed lane is open again, shown in Figure 3.3. The area of A, queue length during D, is equal to the area of B, the number of dissipated vehicles. The queue dissipation time t_d is:

$$t_d = \frac{(Q_1 - c_w)D}{(c_0 - Q_1)} \tag{3.29}$$

where c_0 represents the road capacity in normal (two lanes) conditions in Direction 1 without work zone.

The queuing delay cost per maintained kilometer for Alternative 4.1, C_q^{41}, is queue delay t_q^{41} multiplied by the average delay cost v and divided by L:

$$C_q^{41} = \frac{t_q^{41} v}{L} \tag{3.30}$$

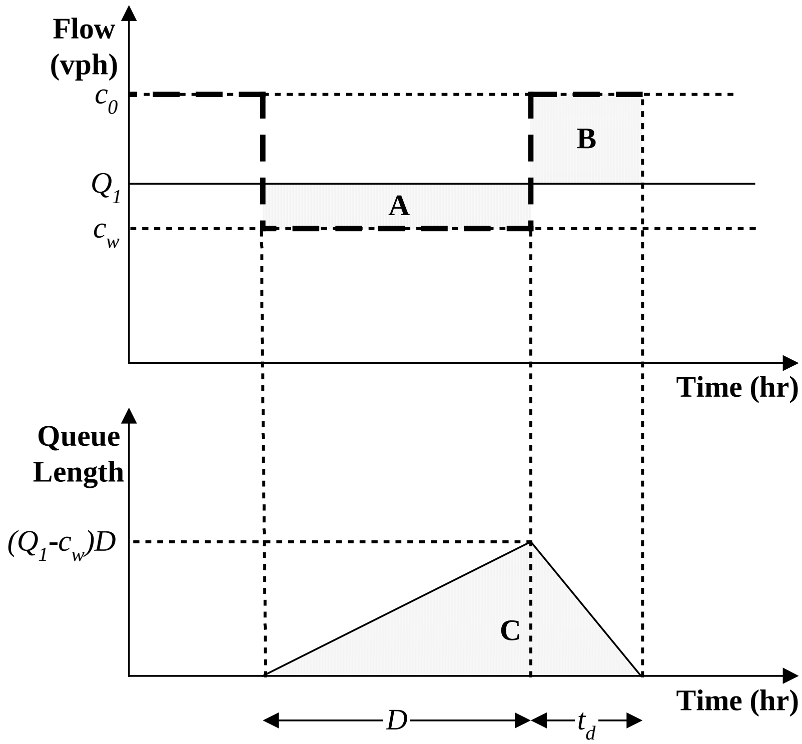


Figure 3.3 Queue Length for Four-lane Highway Work Zone
(Chien and Schonfeld, 2001)

where t_q^{41} = queue delay incurred by the approaching traffic flow Q_1 for Alternative 4.1while work on one zone is completed and the queue is dissipated, which is equal to the area C in Figure 3.3. If Q_1 is less than the maximum discharge rate of work zone, c_w, the queue delay t_q^{41} is neglected. If Q_1 is greater than c_w, the queue delay t_q^{41} is:

$$\begin{aligned} t_q^{41} &= \frac{1}{2}(D + t_d)[(Q_1 - c_w)D)] \\ &= \frac{1}{2}(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q - c_w)(z_3 + z_4 L)^2 \end{aligned} \tag{3.31}$$

Then:

$$C_q^{41} = 0 \qquad when\ \ Q_1 \le c_w \tag{3.32a}$$

$$C_q^{41} = \frac{v}{2L}(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w)(z_3 + z_4 L)^2 \qquad when\ \ Q_1 > c_w \tag{3.32b}$$

The moving delay cost per maintained kilometer C_v^{41} is the moving delay t_m^{41} multiplied by the average delay cost v and divided by L:

$$C_v^{41} = \frac{t_m^{41} v}{L} \tag{3.33}$$

where t_m^{41} = moving delay incurred by the approaching traffic flow Q_1. t_m^{41} is a function of the difference between the travel time on a road with and without a work zone:

$$t_m^{41} = (\frac{L}{V_w} - \frac{L}{V_a})Q_1 D \qquad when\ Q_1 \le c_w \tag{3.34a}$$

$$t_m^{41} = (\frac{L}{V_w} - \frac{L}{V_a})c_w D \qquad when\ Q_1 > c_w \tag{3.34b}$$

where V_a = average approaching speed; V_w = average work zone speed. If Q_1 is greater than c_w, the variable Q_1 is reduced by c_w, because the maximum flow allowed to pass through the work zone is c_w. Then:

$$C_v^{41} = (\frac{1}{V_w} - \frac{1}{V_a})Q_1(z_3 + z_4 L)v \qquad when\ Q_1 \le c_w \tag{3.35a}$$

$$C_v^{41} = (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)v \qquad when\ Q_1 > c_w \tag{3.35b}$$

Total user delay cost per maintained lane kilometer for Alternative 4.1 C_U^{41} is:

$$C_U^{41} = C_q^{41} + C_v^{41} \tag{3.36}$$

The accident cost incurred by the traffic passing the work zone can be determined from the number of accidents per 100 million vehicle hour n_a multiplied by the product of the increasing delay ($t_q^{41} + t_m^{41}$) and the average cost per accident v_a and then divided by work zone length L (Chien and Schonfeld, 2001). Average accident cost per maintained kilometer C_a^{41} is formulated as:

$$C_a^{41} = \frac{(t_q^{41} + t_m^{41})}{L}\frac{n_a v_a}{10^8} \tag{3.37}$$

Then:

$$C_a^{41} = (\frac{1}{V_w} - \frac{1}{V_a})Q_1(z_3 + z_4 L)\frac{n_a v_a}{10^8} \qquad when \;\; Q_1 \le c_w \tag{3.38a}$$

$$\begin{aligned} C_a^{41} = [& \frac{1}{2L}(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w)(z_3 + z_4 L)^2 \\ & + (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)]\frac{n_a v_a}{10^8} \qquad when \;\; Q_1 > c_w \end{aligned} \tag{3.38b}$$

Total cost is:

$$C_T^{41} = C_M + C_U^{41} + C_a^{41} \tag{3.39}$$

Then:

$$C_T^{41} = (\frac{z_1}{L} + z_2) + (\frac{1}{V_w} - \frac{1}{V_a})Q_1(z_3 + z_4 L)(v + \frac{n_a v_a}{10^8}) \qquad when \;\; Q_1 \le c_w \tag{3.40a}$$

$$\begin{aligned} C_T^{41} = (\frac{z_1}{L} + z_2) + [& \frac{1}{2L}(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w)(z_3 + z_4 L)^2 \\ & + (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)](v + \frac{n_a v_a}{10^8}) \qquad when \;\; Q_1 > c_w \end{aligned} \tag{3.40b}$$

The resulting optimized work zone length L^{*41} is then found to be:

$$L^{*41} = \sqrt{\frac{z_1}{z_4 Q_1 P_3 P_4}} \qquad when \, Q_1 \le c_w \tag{3.41a}$$

$$L^{*41} = \sqrt{\frac{2z_1 + P_1 P_2 P_3 z_3^2}{P_1 P_2 P_3 z_4^2 + 2P_3 P_4 c_w z_4}} \qquad when \, Q_1 > c_w \tag{3.41b}$$

where

$$P_1 = Q_1 - c_w \tag{3.42}$$

$$P_2 = 1 + \frac{Q_1 - c_w}{c_0 - Q_1} \tag{3.43}$$

$$P_3 = v + \frac{n_a v_a}{10^8} \tag{3.44}$$

$$P_4 = \frac{1}{V_w} - \frac{1}{V_a} \tag{3.45}$$

The second derivative of C_T^{41} with respect to L is positive in this case and the following ones, indicating that function is convex and has a unique global minimum for L.

Alternative 4.2: A Fraction of Q_1 Traffic through Detour

It is assumed in Alternative 4.2 (Figure 3.2(b)) that the fraction p of the flow Q_1 in Direction 1 is diverted to the alternate route. In this section pQ_1 and $(1\text{-}p)Q_1$ are considered separately. The user delay costs include queuing delay and moving delay cost.

Total user delay cost per maintained lane kilometer for $(1\text{-}p)Q_1$, $C^{42}_{U(1-p)}$, is:

$$C^{42}_{U(1-p)} = C^{42}_{q(1-p)} + C^{42}_{v(1-p)} \tag{3.46}$$

The user queuing delay cost of the remaining flow in Direction 1, $(1\text{-}p)Q_1$, denoted as $C^{42}_{q(1-p)}$, is the queue delay $t^{42}_{q(1-p)}$ for $(1\text{-}p)Q_1$ multiplied by the average delay cost v and divided by L. $t^{42}_{q(1-p)}$ has the same formulation as Eq.(3.31) but with $(1\text{-}p)Q_1$ substituted for Q_1:

$$t^{42}_{q(1-p)} = 0 \qquad when\ (1-p)Q_1 \le c_w \tag{3.47a}$$

$$t^{42}_{q(1-p)} = \frac{1}{2}\left(1 + \frac{(1-p)Q_1 - c_w}{c_0 - (1-p)Q_1}\right)((1-p)Q - c_w)(z_3 + z_4 L)^2 \qquad when\ (1-p)Q_1 > c_w \tag{3.47b}$$

Then $C^{42}_{q(1-p)}$ has the same formulation as Eq. (3.32) but with $(1\text{-}p)Q_1$ substituted for Q_1:

$$C^{42}_{q(1-p)} = 0 \qquad when\ (1-p)Q_1 \le c_w \tag{3.48a}$$

$$C^{42}_{q(1-p)} = \frac{v}{2L}(1 + \frac{(1-p)Q_1 - c_w}{c_0 - (1-p)Q_1})((1-p)Q_1 - c_w)(z_3 + z_4 L)^2 \quad when\ (1-p)Q_1 > c_w \tag{3.48b}$$

The moving delay cost per maintained kilometer $C^{42}_{v(1-p)}$ for *(1-p)Q1* is the moving delay $t^{42}_{m(1-p)}$ for *(1-p)Q1* multiplied by the average delay cost v_d and divided by *L*. $t^{42}_{m(1-p)}$ has the same formulation as Eq.(3.34) but with $(1\text{-}p)Q_1$ substituted for Q_1:

$$t^{42}_{m(1-p)} = (\frac{L}{V_w} - \frac{L}{V_a})(1-p)Q_1 D \qquad when\ (1-p)Q_1 \le c_w \tag{3.49a}$$

$$t^{42}_{m(1-p)} = (\frac{L}{V_w} - \frac{L}{V_a})c_w D \qquad when\ (1-p)Q_1 > c_w \tag{3.49b}$$

Then, $C^{42}_{v(1-p)}$ has the same formulation as Eq.(3.35) but with $(1\text{-}p)Q_1$ substituted for Q_1:

$$C^{42}_{v(1-p)} = (\frac{1}{V_w} - \frac{1}{V_a})(1-p)Q_1(z_3 + z_4 L)v \qquad when\ (1-p)Q_1 \le c_w \tag{3.50a}$$

$$C^{42}_{v(1-p)} = (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)v \qquad when\ (1-p)Q_1 > c_w \tag{3.50b}$$

The user delay cost per maintained lane kilometer for the detoured flow in Direction 1, pQ_1, denoted as C^p_u, is equal to:

$$C^{42}_{Up} = C^{42}_{qp} + C^{42}_{vp} \tag{3.51}$$

where C^{42}_{qp} represents the queuing delay for pQ_1 and C^{42}_{vp} represents the moving delay for pQ_1. We assume the detour capacity c_d always exceeds pQ_1 plus Q_3, so the queuing delay of pQ_1 is zero.

The user moving delay cost of the diverted flow pQ_1 from Direction 1, C^{42}_{vp}, is equal to the flow pQ_1 multiplied by: (1) the average maintenance duration per kilometer,

$\frac{z_3}{L}+z_4$, which is the maintenance duration per zone, z_3+z_4L, divided by work zone L, (2) the time difference between the time vehicles through the detour, $\frac{L_{d1}+L_{d3}}{V_a}+\frac{L_{d2}}{V_d^{*2}}$, and the time vehicles through the maintained road AB without work zone, $\frac{L_t}{V_a}$, and (3) the value of time, v. Thus:

$$C_{vp}^{42} = pQ_1(\frac{z_3}{L}+z_4)[\frac{L_{d1}+L_{d3}}{V_a}+\frac{L_{d2}}{V_d^{*3}}-\frac{L_t}{V_a}]v \tag{3.52}$$

Therefore, the user delay cost for pQ_1 is:

$$C_{Up}^{42} = C_{qp}^{42}+C_{vp}^{42} = C_{vp}^{42} = pQ_1(\frac{z_3}{L}+z_4)[\frac{L_{d1}+L_{d3}}{V_a}+\frac{L_{d2}}{V_d^{*3}}-\frac{L_t}{V_a}]v \tag{3.53}$$

where V_d^{*3} is the detour speed affected by diverted flow pQ_1 in Direction 3 in Alternative 4.2

The additional moving delay cost of the original flow Q_3 in Direction 3, as affected by the detoured flow Q_1, is denoted C_{v3}^{42}. It has the same formulation as Eq.(3.11).

$$C_{v3}^{42} = Q_3(\frac{z_3}{L}+z_4)(\frac{L_{d2}}{V_d^{*3}}-\frac{L_{d2}}{V_{d0}})v \tag{3.54}$$

The total user delay cost C_U^{42} can be determined as follows:

$$C_U^{42} = C_{U(1-p)}^{42}+C_{Up}^{42}+C_{v3}^{42} \tag{3.55}$$

The average accident cost per maintained kilometer for $(1\text{-}p)Q_1$, $C_{a(1-p)}^{42}$, is:

$$C_{a(1-p)}^{42} = \frac{(t_{q(1-p)}^{42}+t_{m(1-p)}^{42})}{L}\frac{n_a v_a}{10^8} \tag{3.56}$$

Then:

$$C^{42}_{a(1-p)} = (\frac{1}{V_w} - \frac{1}{V_a})(1-p)Q_1(z_3 + z_4 L)\frac{n_a v_a}{10^8} \qquad when\ (1-p)Q_1 \le c_w \qquad (3.57a)$$

$$C^{42}_{a(1-p)} = [\frac{1}{2L}(1 + \frac{(1-p)Q_1 - c_w}{c_0 - (1-p)Q_1})((1-p)Q_1 - c_w)(z_3 + z_4 L)^2 + (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)]\frac{n_a v_a}{10^8} \qquad when\ (1-p)Q_1 > c_w \qquad (3.57b)$$

The average accident cost per maintained kilometer for pQ_1, C^{42}_{ap}, is:

$$C^{42}_{ap} = \frac{(t^{42}_{qp} + t^{42}_{mp})}{L}\frac{n_a v_a}{10^8} \qquad (3.58)$$

where

$$t^{42}_{mp} = \frac{C^{42}_{vp} L}{v_d} = (\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_a})pQ_1(z_3 + z_4 L) \qquad when\ pQ_1 \le c_d \qquad (3.59)$$

and t^{42}_{qp} =0. Then:

$$C^{42}_{ap} = (\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_a})pQ_1(\frac{z_3}{L} + z_4)\frac{n_a v_a}{10^8} \qquad when\ pQ_1 \le c_d \qquad (3.60)$$

The average accident cost per maintained kilometer for Q_3, C^{42}_{a3}, is

$$C^{42}_{a3} = \frac{t^{42}_{m3}}{L}\frac{n_a v_a}{10^8} \qquad (3.61)$$

where

$$t^{42}_{m3} = \frac{C^{42}_{v3} L}{v} = Q_3(z_3 + z_4 L)(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}}) \qquad (3.62)$$

Then:

$$C^{42}_{a3} = Q_3(\frac{z_3}{L} + z_4)(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})\frac{n_a v_a}{10^8} \qquad (3.63)$$

The total accident cost C^{42}_a can be determined as follows:

$$C^{42}_a = C^{42}_{a(1-p)} + C^{42}_{ap} + C^{42}_{a3} \qquad (3.64)$$

Then, the total cost is:

$$C_T^{42} = C_M + C_U^{42} + C_a^{42} = C_M + (C_{U(1-p)}^{42} + C_{Up}^{42} + C_{v3}^{42}) + (C_{a(1-p)}^{42} + C_{ap}^{42} + C_{a3}^{42}) \qquad (3.65)$$

The resulting optimized work zone length is:

$$L^{*42} = \sqrt{\frac{z_1 + pQ_1 z_3(\frac{L_{d1}+L_{d3}}{V_a} + \frac{L_{d2}}{V^{*2}} - \frac{L_t}{V_a})(v + \frac{n_a v_a}{10^8}) + Q_3 z_3(\frac{L_{d2}}{V_d^{*2}} - \frac{L_{d2}}{V_{d0}})(v + \frac{n_a v_a}{10^8})}{(\frac{1}{V_w} - \frac{1}{V_a})(1-p)Q_1 z_4(v + \frac{n_a v_a}{10^8})}}$$

$$when\ (1-p)Q_1 \le c_w \qquad (3.66a)$$

$$L^{*42} = \sqrt{\frac{z_1 + [(1 + \frac{(1-p)Q_1 - c_w}{c_0 - (1-p)Q_1})((1-p)Q_1 - c_w)\frac{z_3^2}{2} + pQ_1 z_3(\frac{L_{d1}+L_{d3}}{V_a} + \frac{L_{d2}}{V^{*2}} - \frac{L_t}{V_a})](v + \frac{n_a v_a}{10^8}) + Q_3 z_3(\frac{L_{d2}}{V_d^{*2}} - \frac{L_{d2}}{V_{d0}})(v + \frac{n_a v_a}{10^8})}{(\frac{1}{V_w} - \frac{1}{V_a})c_w z_4(v + \frac{n_a v_a}{10^8}) + (1 + \frac{(1-p)Q_1 - c_w}{c_0 - (1-p)Q_1})((1-p)Q_1 - c_w)\frac{z_4^2}{2}(v + \frac{n_a v_a}{10^8})}}$$

$$when\ (1-p)Q_1 > c_w \qquad (3.66b)$$

The second derivative of C_T^{42} with respect to L is also positive in this case and the following ones, indicating that function is convex and has a unique global minimum for L.

Alternative 4.3: All Q_1 Traffic through Detour, Allowing a Work Zone on Both Lanes in Direction 1

Here it is assumed that the entire flow Q_1 in Alternative 4.2 is diverted to the alternate route (Alternative 4.3, Figure 3.2(c)). Then the total cost in Direction 1 has the same formulation as Eq.(3.65) but with Q_1 substituted for pQ_1 and p is replaced by 1. Here Q_1 may be greater than c_w because Q_1 would not pass through work zone. The total cost for Alternative 4.3 is:

$$C_T^{43} = (\frac{z_1}{L} + z_2)$$
$$+ Q_1(\frac{z_3}{L} + z_4)[\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*2}} - \frac{L_t}{V_a}]v$$
$$+ Q_3(\frac{z_3}{L} + z_4)(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v \tag{3.67}$$
$$+ Q_1(\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*2}} - \frac{L_t}{V_a})(\frac{z_3}{L} + z_4)\frac{n_a v_a}{10^8}$$
$$+ Q_3(\frac{z_3}{L} + z_4)(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})\frac{n_a v_a}{10^8}$$

where V_d^{*3} is the detour speed affected by Q_1 in Direction 3 in Alternative 4.3.

The first and second partial derivatives of C_T^{43} are then found to be:

$$\frac{\partial C_T^{43}}{\partial L} = -[\frac{z_1}{L^2} + \frac{Q_1 z_3}{L^2}(\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*2}} - \frac{L_t}{V_a})v + \frac{Q_3 z_3}{L^2}(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v$$
$$+ \frac{Q_1 z_3}{L^2}(\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*2}} - \frac{L_t}{V_a})\frac{n_a v_a}{10^8} + \frac{Q_3 z_3}{L^2}(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})\frac{n_a v_a}{10^8}] < 0 \tag{3.68}$$

$$\frac{\partial^2 C_T^{43}}{\partial L^2} = \frac{2z_1}{L^3} + \frac{2Q_1 z_3}{L^3}(\frac{L_{d1} + L_{d3}}{V_a} + \frac{L_{d2}}{V_d^{*2}} - \frac{L_t}{V_a})v + 2\frac{Q_3 z_3}{L^3}(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v$$
$$+ \frac{2Q_1 z_3}{L^3}(\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*4}} - \frac{L_t}{V_0})\frac{n_a v_a}{10^8} + 2\frac{Q_3 z_3}{L^3}(\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})\frac{n_a v_a}{10^8} > 0 \tag{3.69}$$

The first partial derivative of C_T^{43} is negative and the second partial derivative is positive. Therefore the function C_T^{43} is convex and there is no local or global minimum for zone length is between 0 and L_t. The minimal cost occurs when the zone length is L_t.

Alternative 4.4: Crossover of All Q_1 Traffic into One Opposite Lane, Allowing a Work Zone on Both Lanes in Direction 1

Here it is assumed that the entire flow Q_1 in Alternative 4.1 crosses over to one lane in the opposite direction (Figure 3.2(d)). Both lanes in Direction 1 are closed for work zone. The flow Q_2 in Direction 2 only uses the remaining lane. In Alternative 4.4, we assume (1) the vehicles in Q_1 and Q_2 along work zone have the same speed, V_w, (2) the capacity of each lane in Direction 2 between the start and end of work zone for Q_1 and Q_2 is equal to work zone capacity, c_w, (3) the distance between the start and end of work zone in Direction 1 is equal to the distance of crossover route through alternate lane in Direction 2.

In Alternative 4.4, the queuing delay and moving delay may occur for either Q_1 or Q_2. Below are all possible combinations for user queuing delay costs, moving delay costs, and accident costs.

$$C_{qj}^{44} = 0 \qquad when\ Q_j \le c_w \qquad j = 1,2 \tag{3.70a}$$

$$C_{qj}^{44} = \frac{v}{2L}(1 + \frac{Q_i - c_w}{c_0 - Q_1})(Q_i - c_w)(z_3 + z_4 L)^2 \qquad when\ Q_j > c_w \qquad j = 1,2 \tag{3.70b}$$

where C_{qj}^{44} is user queuing delay cost for Q_j.

$$C_{vj}^{44} = (\frac{1}{V_w} - \frac{1}{V_a})Q_j(z_3 + z_4 L)v \qquad when\ Q_j \le c_w \qquad j = 1,2 \tag{3.71a}$$

$$C_{vj}^{44} = (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)v \qquad when\ Q_j > c_w \qquad j = 1,2 \tag{3.71b}$$

where C_{vj}^{44} is user moving delay cost for Q_j.

$$C_{aj}^{44} = (\frac{1}{V_w} - \frac{1}{V_a})Q_j(z_3 + z_4 L)\frac{n_a v_a}{10^8} \qquad when\ Q_j \le c_w \qquad j = 1,2 \tag{3.72a}$$

$$\begin{aligned} C_{aj}^{44} = [& \frac{1}{2L}(1 + \frac{Q_j - c_w}{c_0 - Q_i})(Q_j - c_w)(z_3 + z_4 L)^2 \\ & + (\frac{1}{V_w} - \frac{1}{V_a})c_w(z_3 + z_4 L)] \frac{n_a v_a}{10^8} \qquad when\ Q_j > c_w \qquad j = 1,2 \end{aligned} \tag{3.72b}$$

where C_{aj}^{44} is accident cost for Q_j.

The total cost is then:

$$
\begin{aligned}
C_T^{44} &= C_M + C_U^{44} + C_a^{44} \\
&= C_M + (C_{U1}^{44} + C_{U2}^{44}) + (C_{a1}^{44} + C_{a2}^{44}) \\
&= C_M + C_{q1}^{44} + C_{v1}^{44} + C_{q2}^{44} + C_{v2}^{44} + C_{a1}^{44} + C_{a2}^{44}
\end{aligned} \tag{3.73}
$$

where C_U^{44} is total user delay cost per maintained lane kilometer and C_a^{44} total accident cost per maintained lane kilometer for Alternative 4.4.

Optimized work zone lengths L^{*44} are then derived for four combinations of conditions defined by whether Q_1 and Q_2 are above or below the capacity c_w.

(1) If $Q_1 \le c_w$ & $Q_2 \le c_w$:

$$
L^{*44} = \sqrt{\frac{z_1}{z_4(Q_1 + Q_2)(v + \frac{n_a v_a}{10^8})(\frac{1}{V_w} - \frac{1}{V_a})}} \tag{3.74}
$$

(2) If $Q_1 > c_w$ & $Q_2 \le c_w$:

$$
L^{*44} = \sqrt{\frac{z_1 + \frac{z_3^2}{2}(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w)(v + \frac{n_a v_a}{10^8})}{z_4(c_w + Q_2)(v + \frac{n_a v_a}{10^8})(\frac{1}{V_w} - \frac{1}{V_a}) + \frac{z_4^2}{2}(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w)(v + \frac{n_a v_a}{10^8})}} \tag{3.75}
$$

(3) If $Q_1 \le c_w$ & $Q_2 > c_w$:

$$
L^{*44} = \sqrt{\frac{z_1 + \frac{z_3^2}{2}(1 + \frac{Q_2 - c_w}{c_0 - Q_2})(Q_2 - c_w)(v + \frac{n_a v_a}{10^8})}{z_4(c_w + Q_1)(v + \frac{n_a v_a}{10^8})(\frac{1}{V_w} - \frac{1}{V_a}) + \frac{z_4^2}{2}(1 + \frac{Q_2 - c_w}{c_0 - Q_2})(Q_2 - c_w)(v + \frac{n_a v_a}{10^8})}} \tag{3.76}
$$

(4) If $Q_1 > c_w$ & $Q_2 > c_w$:

$$
L^{*44} = \sqrt{\frac{z_1 + \frac{z_3^2}{2}(v + \frac{n_a v_a}{10^8})[(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w) + (1 + \frac{Q_2 - c_w}{c_0 - Q_2})(Q_2 - c_w)]}{2 z_4 c_w (v + \frac{n_a v_a}{10^8})(\frac{1}{V_w} - \frac{1}{V_a}) + \frac{z_4^2}{2}(v + \frac{n_a v_a}{10^8})[(1 + \frac{Q_1 - c_w}{c_0 - Q_1})(Q_1 - c_w) + (1 + \frac{Q_2 - c_w}{c_0 - Q_2})(Q_2 - c_w)]}} \tag{3.77}
$$

Because no alternate path is involved in Alternative 4.4, no detour parameters are shown in Eqs (74), (75), (76), and (77).

3.4 Determination of Work Zone and Detour Speeds

The relations between speed and flow have been extensively researched in past decades. In 1935 Greenshield proposed a parabolic equation for speed-flow curve on the basis of a linear speed-density relationship together with the equation, flow = speed * density. This model was widely used and appeared in the 1965 Highway Capacity Manual (HCM) and the 1985 HCM. However, some objections to for Greenshield's model have been made. One is that Greenshield's model did not work with freeway data. The second is that the curve-fitting of this model by current standards of research and empirical data would not acceptable (Messer et al., 1997). Many studies show that the relationship between speed and flow is divided into three stages: uncongested, queue discharge, and within a queue (Hall, et al., 1992). In the speed-flow curve, speed remains flat as flows increases between half and two-thirds of capacity values, and has a very small decrease in speeds at capacity from those values (Messer et al., 1997). Such a curve is also shown in the 1994 HCM. To simplify the analytic work zone optimization models, Greenshield's model is used below.

In traffic flow theory, the relation among flow Q, density K, and speed V is:

$$Q = KV \tag{3.78}$$

The speed function can be formulated by applying Greenshield's model (Gerlough and Huber, 1975):

$$V = V_f - \frac{V_f}{K_j} K \tag{3.79}$$

where V_f is free flow speed, K_j is jam density.

Substituting (3.79) into (3.78), we obtain

$$Q = K_j V - \frac{K_j}{V_f} V^2 \tag{3.80}$$

Solving the quadratic Eq.(3.80) for the speed V, we obtain two solutions. The first is:

$$V = \frac{K_j V_f + \sqrt{(K_j V_f)^2 - 4K_j V_f Q}}{2K_j} \tag{3.81}$$

Then, V_0, V_{d0}, V_d^{*3} and V_d^{*4} in Alternatives 2.2 and 2.3 or 4.2 and 4.3 can be determined from Eq.(3.81). The other solution of Eq.(3.80) is:

$$V = \frac{K_j V_f - \sqrt{(K_j V_f)^2 - 4K_j V_f Q}}{2K_j} \tag{3.82}$$

which is the speed under forced flow conditions (Gerlough and Huber, 1975). This speed is not used in Case 1 because V_0, V_{d0}, V_d^{*3} and V_d^{*4} are applied based on the assumption that the original road without work zone and detour has enough capacity for steady traffic inflows so that the speeds on the original road (V_0) and detour (V_{d0}) are free-flowing speeds. In Chapter 5, the congestion and delay along a detour will be considered when work zone optimization models for time-dependent inflows with a detour are developed.

3.5 Threshold Analysis

In this section the selection of the best alternatives is considered under different situations. Guidelines for selecting the best alternative for different traffic flows, roads and maintenance characteristics are developed by deriving thresholds among those alternatives.

C_T^{*1}, C_T^{*2}, C_T^{*3} and C_T^{*4} are the minimized total costs of Alternatives 2.1, 2.2, 2.3 and 2.4, (or Alternatives 4.1, 4.2, 4.3 and 4.4) computed with their respective optimized work zone lengths L^{*1}, L^{*2} L^{*3} and L^{*4}. The threshold between any two alternatives can be obtained by setting their two cost functions equal. For example, Figure 3.4 shows the relation between total cost and detour length. It indicates that Alternative 2.3 is preferable up to a detour length of T_{32}^{DL}, beyond which Alternative 2.2 is preferable up to T_{21}^{DL}.

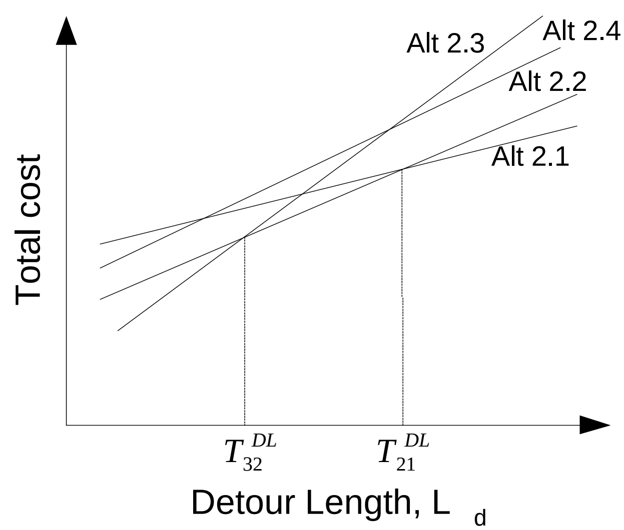


Figure 3.4 Total Cost vs. Detour Length

Thresholds with respect to the distance AB, setup cost, average maintenance time, and other input parameters, can be obtained similarly to the detour length thresholds. For some variables or alternatives, if the thresholds are not positive or not located within applicable ranges, then no threshold exists.

3.6 Numerical Analysis - Two-Lane Two-Way Highway

3.6.1 Sensitivity Analysis

The effects of various parameters on work zone length and the preferable alternatives are examined in this section. The baseline numerical values for each variable in this section are defined in Table 3.1.

The optimized solutions for work zone length and total cost are shown in Table 3.2 for various traffic flow combinations. For Alternatives 2.1 and 2.2, when Q_1 or Q_2 increases, the optimized zone length decreases. However, for Alternative 2.3, the optimized zone length increases slightly with Q_1 and decreases with Q_2, because increasing zone length decreases the delay cost of Q_1 in Eq.(3.15). The optimized zone length ranges from 1.54 to 0.49 km for Alternative 2.1, 2.17 to 0.20 km for Alternative 2.2, 2.3 to 0.74 km for Alternative 2.3, and 5 km for any Alternative 2.4. Table 3.2 shows that the optimized zone length increases with the diverted fraction to the detour from Q_1. The combined flow Q_1+Q_2 ranges from 100 to 2,000 vph. Note that the optimized zone length and minimized total cost are not available when the combined flow exceeds the work zone capacity 1,200 vph. At the baseline values, Alternative 2.4 dominates all others in Table 3.2, as its optimized total cost is the lowest for any flow combination Q_1 and Q_2.

Table 3.1 Inputs for Numerical Example and Sensitivity Analysis for Two-Lane Two-Way Highway Work Zones

Variable	Description	Values
H	Average headway through work zone area	3 s
K_j	Jam density along AB and detour	200 veh/lane·km
L_{d1}	Length of first detour segment	0.5 km
L_{d2}	Length of second detour segment	5 km
L_{d3}	Length of third detour segment	0.5 km
L_t	Entire Distance of Maintained Road from A to B	5 km
n_a	Number of accidents per 100 million vehicle hour	40 acc/100mvh
Q_3	Hourly flow rate in Direction 3	500 veh/hr
V	Average work zone speed	50 km/hr
V_f	Free flow speed along AB and detour	80 km/hr
v	Value of user time	12 $/veh·hr
z_1	Fixed setup cost	1,000 $/zone
z_2	Average maintenance cost per lane·kilometer	80,000 $/lane·km
z_3	Fixed setup time	2 hr/zone
z_4	Average maintenance time per lane·kilometer	6 hr/lane·km

To examine sensitivities to other factors, we fix the traffic flow rates Q_1 and Q_2 at 400 vehicles per hour (vph) each. Figure 3.5 shows increases in user cost as the zone length increases in Alternatives 2.1, 2.2, and 2.3. However, user cost decreases slightly as the zone length increases in Alternative 2.4 because no vehicle passes through the work zone and the longer zone decreases the moving delay per lane-kilometer.

Table 3.2 Optimized work zone lengths and Total Costs for Different Flow Rates

Q_1+Q_2	Q_1	Q_2	Alt.2.1		Alt.2.2 (p=0.3)		Alt.2.2 (p=0.6)		Alt.2.2 (p=0.9)		Alt.2.3		Alt.2.4	
			Optim. length	Min. total cost	Optim. length	Min. total cost	Optim. length	Min. total cost	Optim. length	Min. total cost	Optim. length	Min. total cost	Optim. length	Min. total cost
200	100	100	1.54	81,260	1.69	81,185	1.89	81,101	2.17	81,003	2.30	80,966	5.00	80,461
400	200	200	1.04	81,975	1.16	81,847	1.32	81,709	1.55	81,550	1.66	81,491	5.00	80,727
600	200	400	0.80	82,695	0.88	82,502	0.99	82,316	1.12	82,129	1.17	82,064	5.00	81,023
800	200	600	0.64	83,559	0.72	83,204	0.81	82,897	0.92	82,617	0.96	82,527	5.00	81,353
1000	200	800	0.48	85,162	0.58	84,245	0.67	83,596	0.79	83,085	0.83	82,933	5.00	81,723
1200	200	1000	-	-	0.34	88,747	0.51	85,172	0.68	83,659	0.74	83,302	5.00	82,136
600	400	200	0.80	82,693	0.95	82,442	1.16	82,194	1.53	81,908	1.73	81,792	5.00	80,992
800	400	400	0.61	83,846	0.73	83,303	0.89	82,888	1.12	82,512	1.22	82,383	5.00	81,277
1000	400	600	0.43	86,096	0.57	84,520	0.72	83,660	0.91	83,044	1.00	82,860	5.00	81,597
1200	400	800	-	-	0.37	87,872	0.57	84,886	0.78	83,595	0.86	83,277	5.00	81,957
1400	400	1000	-	-	-	-	0.28	92,322	0.65	84,444	0.77	83,657	5.00	82,359
800	600	200	0.64	83,556	0.81	83,048	1.05	82,673	1.51	82,275	1.80	82,106	5.00	81,268
1000	600	400	0.43	86,095	0.61	84,301	0.81	83,490	1.12	82,907	1.27	82,715	5.00	81,542
1200	600	600	-	-	0.42	86,921	0.64	84,549	0.91	83,487	1.04	83,206	5.00	81,852
1400	600	800	-	-	-	-	0.46	86,882	0.77	84,134	0.90	83,635	5.00	82,200
1600	600	1000	-	-	-	-	-	-	0.61	85,342	0.80	84,024	5.00	82,592
1000	800	200	0.49	85,159	0.70	83,736	0.97	83,154	1.49	82,652	1.87	82,433	5.00	81,558
1200	800	400	-!	-	0.49	85,866	0.74	84,146	1.11	83,315	1.32	83,061	5.00	81,821
1400	800	600	-	-	0.20	99,517	0.56	85,675	0.91	83,947	1.08	83,565	5.00	82,119
1600	800	800	-	-	-	-	0.32	91,391	0.76	84,704	0.94	84,006	5.00	82,456
1800	800	1000	-	-	-	-	-	-	0.57	86,401	0.84	84,406	5.00	82,836
1200	1000	200	-	-	0.60	84,658	0.90	83,649	1.48	83,041	1.95	82,777	5.00	81,865
1400	1000	400	-	-	0.33	90,098	0.68	84,891	1.11	83,737	1.38	83,423	5.00	82,115
1600	1000	600	-	-	-	-	0.48	87,296	0.91	84,426	1.12	83,942	5.00	82,400
1800	1000	800	-	-	-	-	-	-	0.75	85,311	0.97	84,394	5.00	82,725
2000	1000	1000	-	-	-	-	-	-	0.53	87,698	0.87	84,805	5.00	83,093

Table 3.3 compares the delay costs for different directional flows that add up to 1400 vph. For Alternative 2.2 (p=0.6), although the combined flow is the same, the combinations with larger Q_2 have shorter optimized zones and higher total costs. This occurs because the queue delay cost on the main road, C_q^{22}, which is the main part of the total delay costs, increases as Q_2 increases.

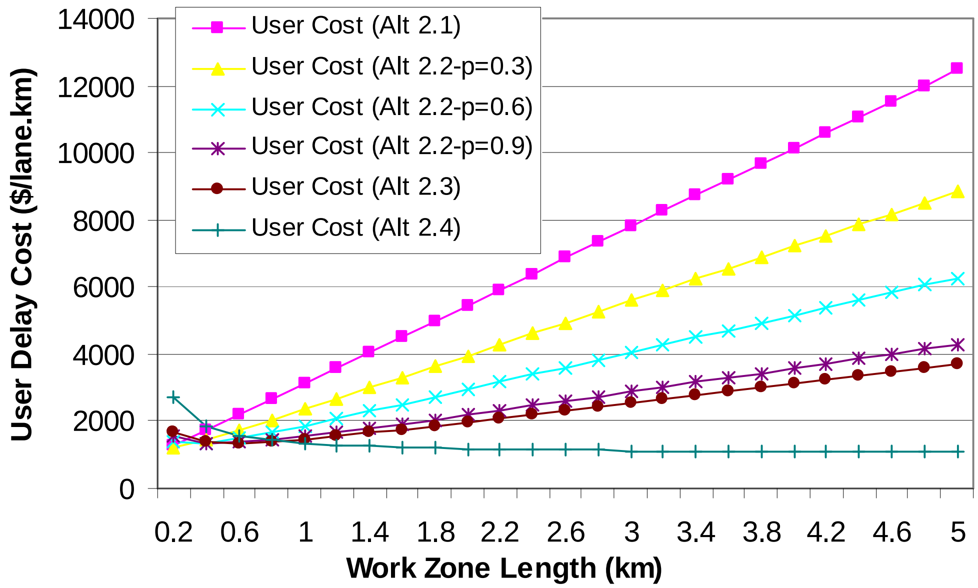


Figure 3.5 User Costs versus Various Zone Lengths (Q_1=400vph, Q_2=400vph)

Table 3.3. Comparison of Delay Costs with Different Directional Flows for Alternative 2.2 (*p*=o.6)

Q_1+Q_2 (vph)	Q1 (vph)	Q2 (vph)	Optimized Length (km)	Cost	C_T^{22} ($/km)	C_M ($/km)	C_U^{22} ($/km)			
							C_q^{22}	C_{vp}^{22}	C_{v3}^{22}	$C_{v(1-p)2}^{22}$
1,400	400	1,000	0.28	Value	92,322	83,546	8,116	542	87	31
				Percent of Cost	100%	90.49%	8.79%	0.59%	0.09%	0.03%
1,400	1,000	400	0.68	Value	84,891	81,474	2,331	897	157	32
				Percent of Cost	100%	95.97%	2.75%	1.10%	0.19%	0.04%

As the zone length increases, the maintenance costs per kilometer decreases due to fewer setups, but stays the same for all alternatives. Combined with the user cost in Figure 3.5, the zone lengths that minimize total costs are determined by trade-offs between the user and maintenance cost, show in Figure 3.6. The optimized zone lengths for Alternatives 2.1, 2.2 (*p*=0.3), 2.3, and 2.4 are 0.61 km, 0.73 km, 1.22 km, and 5.00 km, respectively. Faster increases in the user cost of Alternative 2.1 shorten its optimized zone.

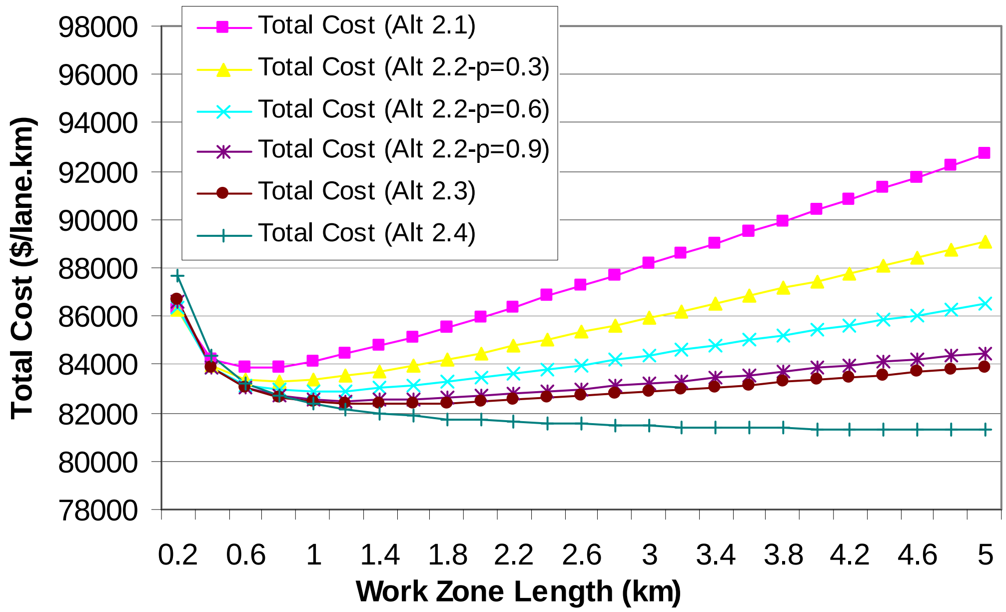


Figure 3.6 Total Costs versus Various Work Zone Lengths (Q_1=400vph, Q_2=400vph)

Figures 3.7 and 3.8 show how setup cost z_1 and average maintenance time z_4 affect the optimized zone length. Figure 3.7 shows that the optimized zone length increases when the setup cost z_1 increases for Alternatives 2.1, 2.2, and 2.3, because longer zones imply fewer setups and decreased total cost. In Alternative 2.4, total cost is minimized when zone length is 5 km, regardless of other variables. Then, the optimized zone length of Alternative 2.4 is entirely unaffected by setup cost. Figure 3.8 shows that the optimized zone length decreases when the average maintenance time increases, in order to avoid excessive increases in user delay. The optimized zone length of Alternative 2.4 is also entirely unaffected by average maintenance time. Additional sensitivity of the optimized zone length to setup duration, work zone speed, and other factors is provided in Chen and Schonfeld (2002).

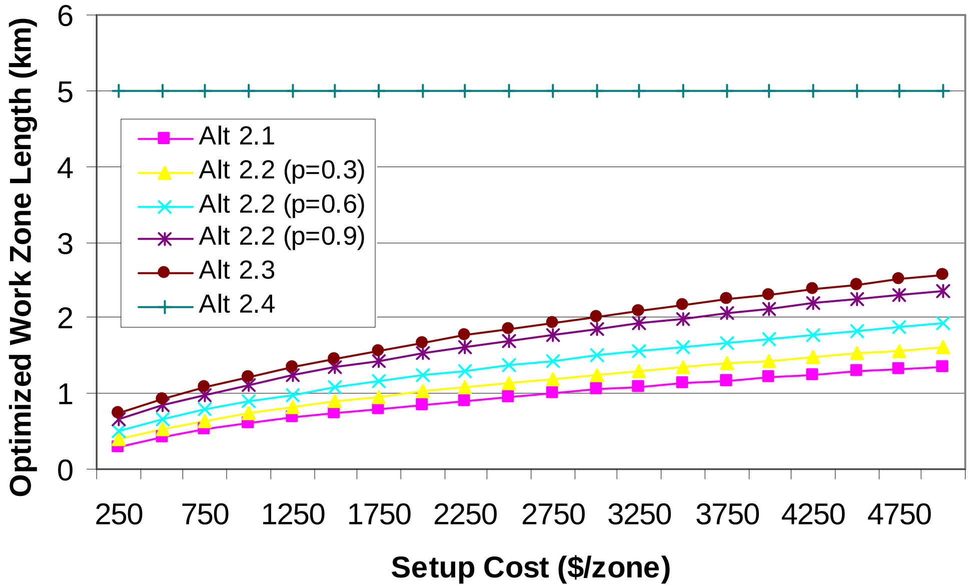


Figure 3.7 Optimized Zone Length versus Setup Cost z_1 (Q_1=400vph, Q_2=400vph)

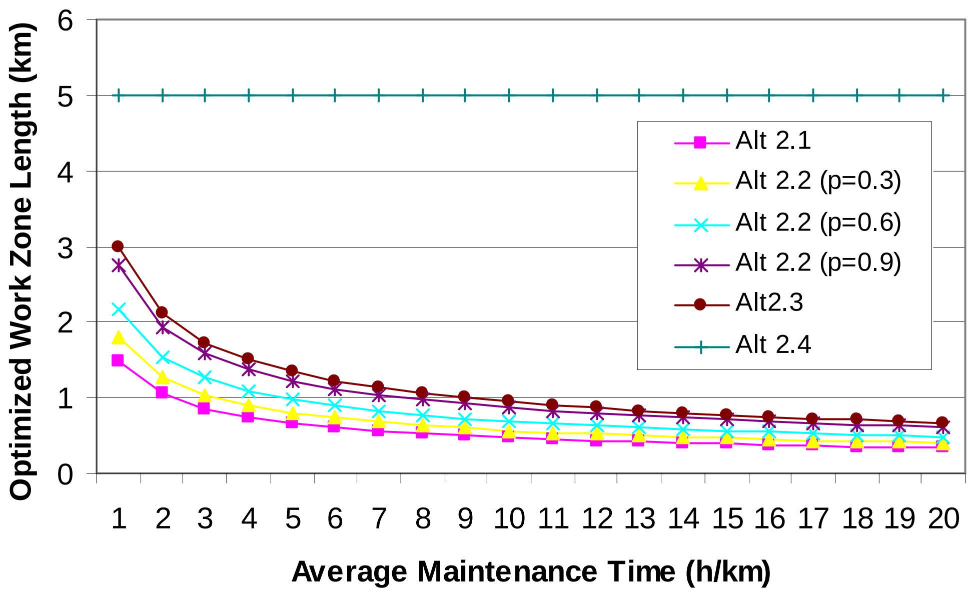


Figure 3.8 Optimized Zone Length versus Average Maintenance Time z_4 (Q_1=400vph, Q_2=400vph)

Figure 3.9 shows that the combined capacity of the maintained road and its detour increases as the diverted fraction increases. Here the capacity for Alternative 2.1 is 1200 vph. As the diverted fraction increases, the combined flow discharge increases. The combined capacity is about 1450 vph for Alternative 2.2 (p=0.3) and about 1700 vph for Alternative 2.2 (p=0.6). The capacity of the one lane through the zone in Alternative 2.1 can be also obtained by dividing one hour (3600 seconds) by the headway (3 seconds) through the zone. Starting from Alternative 2.1 as the baseline, the additional capacity in Alternatives 2.2 and 2.3 is contributed by the detour. Higher diverted fractions increase the capacity through the zone.

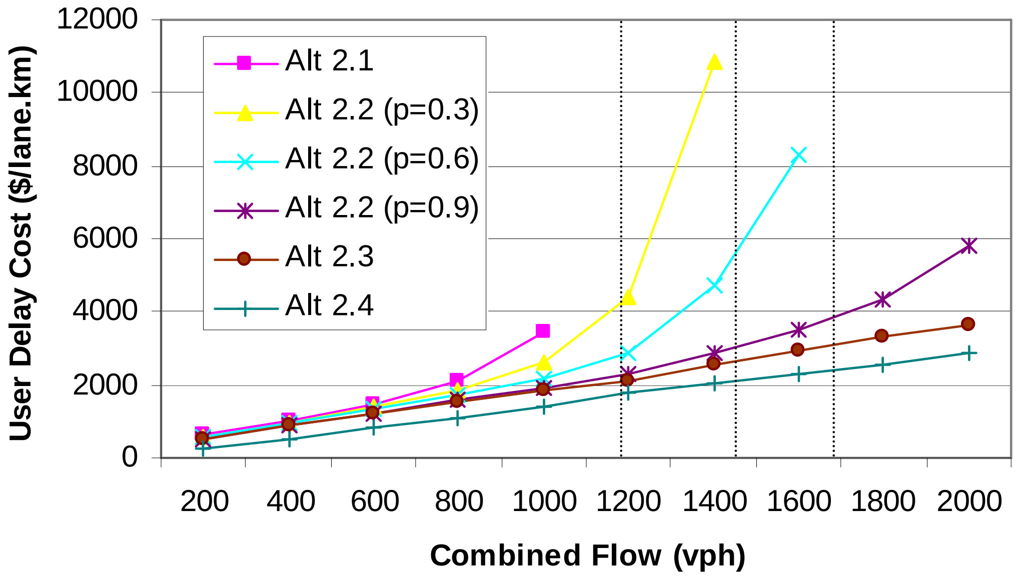


Figure 3.9 User Delay Costs versus Combined Flows

3.6.2 Selection Guidelines

Thresholds among alternatives with respect to four variables, namely, detour length (L_d), length of main road between the beginning and end of detour (L_t), setup cost (z_1), and average maintenance time per kilometer (z_4), are solved numerically and presented below.

Figure 3.10 shows the relation between total cost and detour length when Q_1 and Q_2 are each 200 vph. The detour length threshold is 9.00 km, beyond which Alternative 2.1 becomes preferable to Alternative 2.4.

Figure 3.11 shows that there are four detour length thresholds and Alternatives 2.1, 2.2, 2.3, and 2.4 are on the lowest cost envelope when Q_1 and Q_2 are each 400 and 600 vph. The first threshold occurs at 10 km, beyond which Alternative 2.3 becomes preferable to Alternative 2.4; beyond 11 km Alternative 2.2 (p=0.6) becomes preferable to Alternative 2.3; beyond 12 km Alternative 2.2 (p=0.3) becomes preferable to Alternative 2.2 (p=0.6); beyond 15 km Alternative 2.1 becomes preferable to Alternative 2.2 (p=0.3). Figure 3.12 shows the relation between total cost and detour length when Q_1 and Q_2 are each 800 and 600 vph. There are three detour length thresholds, 9 km, 12 km, and 14 km, and Alternatives 2.2 (p=0.6 and 0.9), Alternatives 2.3 and 2.4 are on the lowest cost envelope.

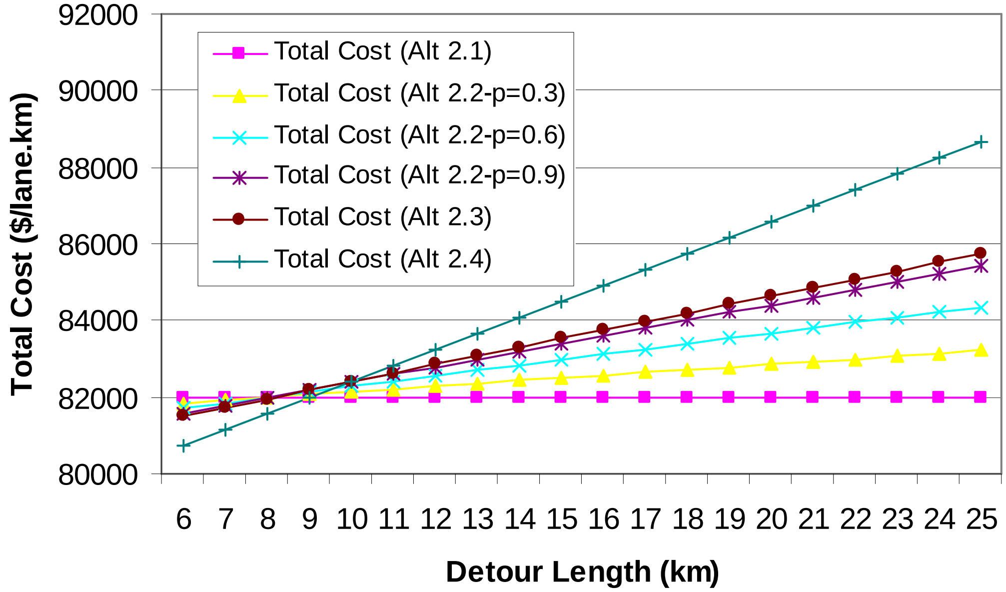


Figure 3.10 Total Cost versus Detour Length for Various Alternatives (Q_1=200vph, Q_2=200vph)

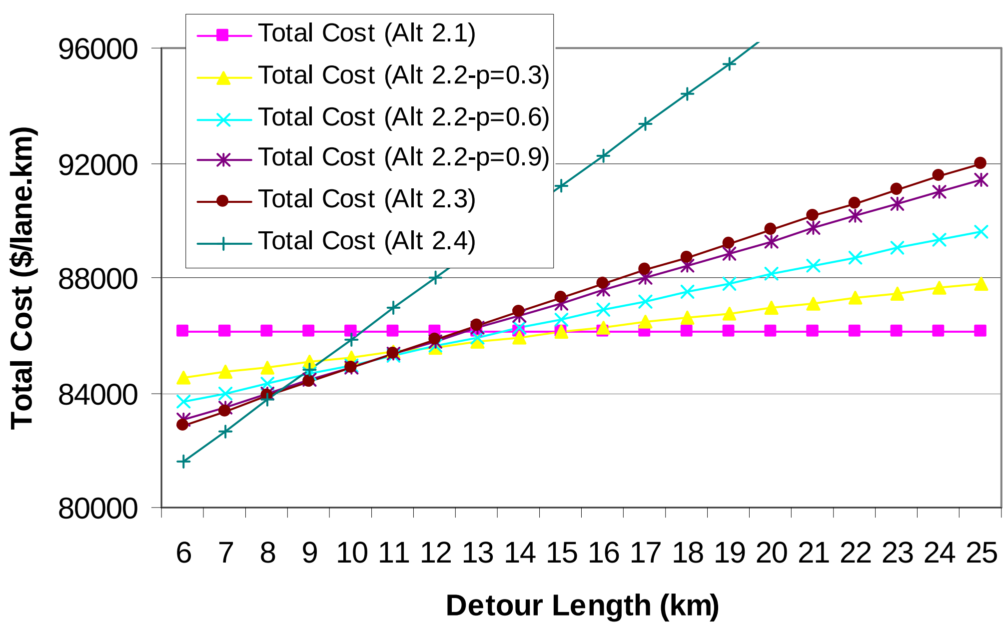


Figure 3.11 Total Cost versus Detour Length for Various Alternatives (Q_1=400vph, Q_2=600vph)

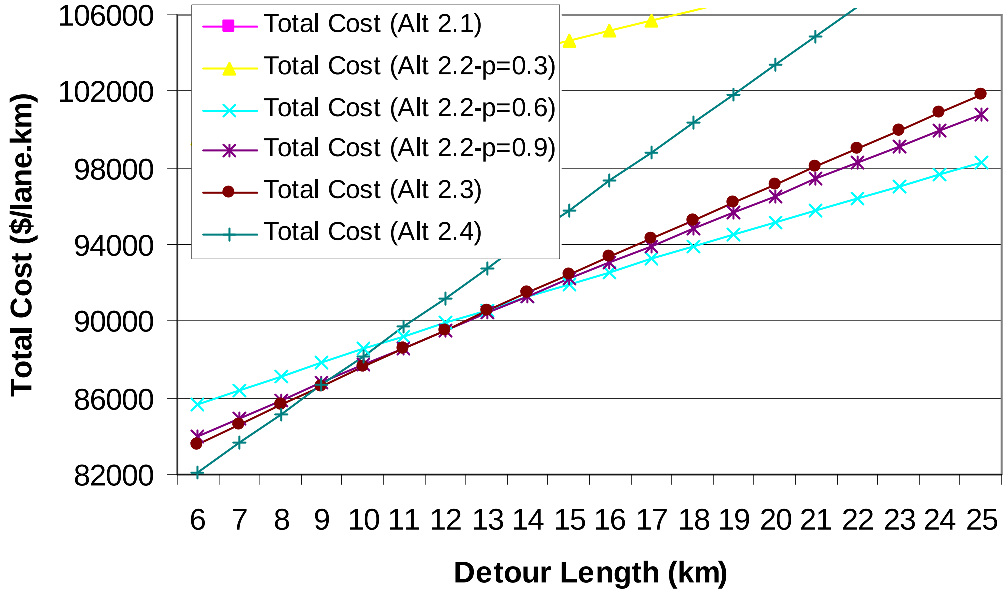


Figure 3.12 Total Cost versus Detour Length for Various Alternatives (Q_1=800vph, Q_2=600vph)

Defining circuity as the ratio of detour distance to maintained road distance = L_d / L_t, the circuity thresholds are shown for various traffic flows in Table 3.4. The numbers in Table 3.4 represent the preferred pair of alternatives that determine the threshold. If combined flow does not exceed 1000 vph, Alternatives 2.1 and 2.4 determine most thresholds, as illustrated in Figure 3.10.

As combined flow increases, Alternatives 2.2 and 2.3 may determine thresholds and additional detour length thresholds appear. Thus, Alternatives 2.1, 2.2, 2.3, and 2.4 all appear on the lowest cost envelope in Figure 3.11. As combined flow increases, e.g. beyond 1400 vph, Alternative 2.2 (whose diverted fraction is lower) is not preferable anymore, e.g. in Figure 3.12.

Table 3.4 Circuity Threshold at Different Flow Rates

Q_1+Q_2	Q_1	Q_2	Circuity threshold									
			Alt.2.1 & Alt.2.4	Alt.2.1 & Alt.2.2 (p=0.3)	Alt.2.1 & Alt.2.2 (p=0.6)	Alt.2.2 (p=0.3) & Alt.2.2 (p=0.6)	Alt.2.2 (p=0.3) & Alt.2.4	Alt.2.2 (p=0.6) & Alt.2.2 (p=0.9)	Alt.2.2 (p=0.6) & Alt.2.3	Alt.2.2 (p=0.6) & Alt.2.4	Alt.2.2 (p=0.9) & Alt.2.3	Alt.2.3 & Alt.2.4
200	100	100	2	-	-	-	-	-	-	-	-	-
400	200	200	1.8	-	-	-	-	-	-	-	-	-
600	200	400	1.8	-	-	-	-	-	-	-	-	-
600	400	200	1.8	-	-	-	-	-	-	-	-	-
800	200	600	-	-	2.2	-	-	-	2	-	-	1.6
800	400	400	-	2	-	-	1.8	-	-	-	-	-
800	600	200	1.8	-	-	-	-	-	-	-	-	-
1,000	200	800	-	3.4	-	3	-	2.8	-	-	2.6	1.6
1,000	400	600	-	3	-	2.4	-	-	2.2	-	-	1.8
1,000	600	400	-	2.6	-	-	2	-	-	-	-	-
1,000	800	200	-	2.2	-	-	1.8	-	-	-	-	-
1,200	200	1,000	-	-	-	-	-	-	-	-	5	1.6
1,200	400	800	-	-	-	-	-	3.4	-	-	3	1.6
1,200	600	600	-	-	-	3.6	-	-	2.4	-	-	1.8
1,200	800	400	-	-	-	2.6	-	-	-	1.8	-	-
1,200	1,000	200	-	-	-	-	2	-	-	-	-	-
1,400	400	1,000	-	-	-	-	-	-	-	-	-	1.6
1,400	600	800	-	-	-	-	-	5	-	-	3.4	1.6
1,400	800	600	-	-	-	-	-	2.8	-	-	2.4	1.8
1,400	1,000	400	-	-	-	-	-	-	-	2.2	-	-
1,600	600	1,000	-	-	-	-	-	-	-	-	-	1.6
1,600	800	800	-	-	-	-	-	-	-	-	3.8	1.8
1,600	1,000	600	-	-	-	-	-	3.6	-	-	2.4	2
1,800	800	1,000	-	-	-	-	-	-	-	-	-	1.6
1,800	1,000	800	-	-	-		-		-	-	4	1.8
2,000	1,000	1,000	-	-	-		-		-	-	-	1.6

The thresholds with respect to setup cost, z_1, average maintenance time per kilometer, z_4, and other factors at different flow rates can be obtained similarly to circuity ratio thresholds.

3.6.3 Optimizing the Diverted Fraction

Figures 3.13 and 3.14 show the relation between total cost and the diverted fraction of Q_1 at different flow rates for Alternatives 2.1, 2.2, and 2.3. (Alternative 2.4

with full diversion in both directions is not included). When the detour length L_d has its baseline value, 6 km, and Q_2 is 400 vph, the total costs are lowest as p approaches 1.0, which indicates Alternative 2.3 is preferable for various Q_1 flows, as illustrated in Figure 3.13. If the detour length L_d increases to 12 km, and Q_2 is 400 vph, the minimized total cost occurs at p=0 (Alternative 2.1, no diversion) for Q_1 of 200 and 400 vph; and at the lowest points of p, p=0.2, 0.4 for Q_1 of 600 and 800 vph, respectively. These indicate that full diversion is preferable when the detours are short; some or no diversion becomes preferable as detour length increases. The results of Figures 3.13 and 3.13 also can be obtained analytically, by setting to zero the partial derivatives of C_T with respect to p and solving for the optimal p value.

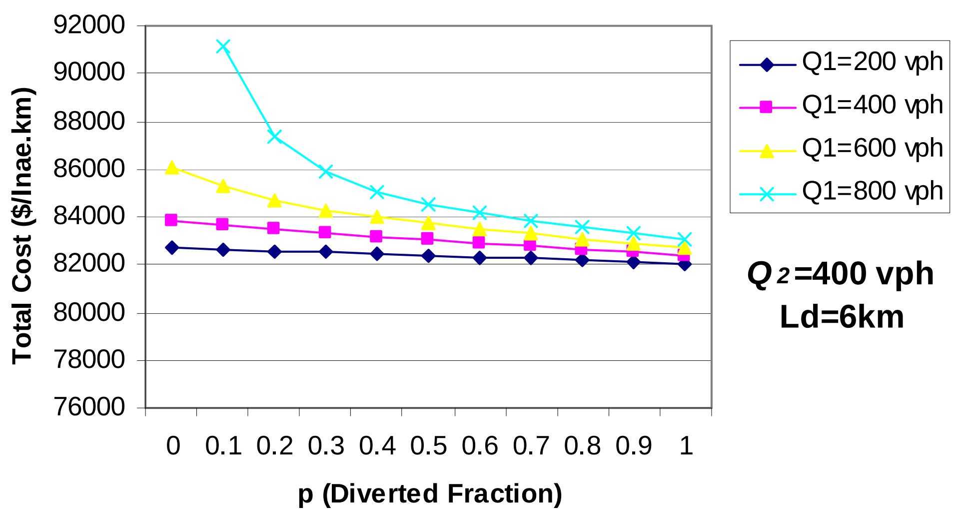


Figure 3.13. Total Cost versus Diverted Fraction (Q_2=400vph, L_d=6km)

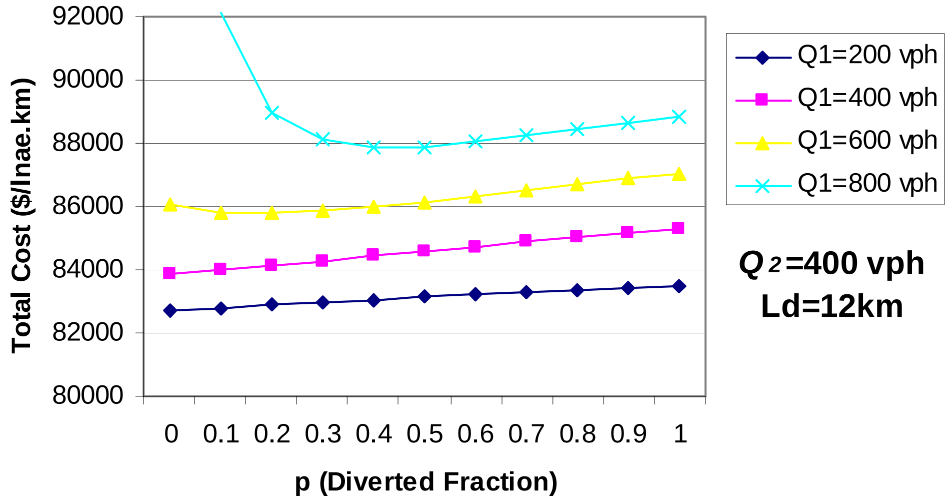


Figure 3.14 Total Cost versus Diverted Fraction (Q_2=400vph, L_d=12km)

3.6.4 Summary

In this section work zone cost models are developed for four alternative zone configurations with and without an alternate route. The optimized zone length and preferred alternative for various combinations of variables are determined with these cost models. When the traffic flows in two directions are steady, Alternative 2.1 has a higher user cost and shorter zone than other alternatives while Alternative 2.4 has a lower user cost and longer zone. As Q_1 or Q_2 increase, the optimized zone length decreases for Alternatives 2.1 and 2.2. However, for Alternative 2.3, the optimized zone length increases slightly as Q_1 increases, and decreases as Q_2 increases. The optimized zone length of Alternative 2.4 is unaffected by any other variables

In the threshold analysis presented, Alternative 2.4 is the preferred alternative in the baseline condition. As detour length L_d increases beyond its threshold, Alternatives 2.1, 2.2 or 2.3 may become preferable. This occurs because increasing L_d increases the user cost. Therefore, the preferred alternative changes when the total cost of Alternative 2.4 exceeds that of Alternatives 2.1, 2.2, or 2.3. Considering an optimized diverted fraction among Alternatives 2.1, 2.2, and 2.3, full diversion is preferable if the detour is short; partial or no division becomes preferable as detour length increases.

3.7 Numerical Analysis – Four-Lane Two-Way Highways

3.7.1 Sensitivity Analysis

The effects of various parameters on work zone length and the preferable alternatives are examined in this section. The baseline numerical values for each variable are the same as in Table 3.1. The baseline numerical values for additional variables in this section are defined in Table 3.5.

Table 3.5 Notation and Baseline Numerical Inputs Analysis for Four-Lane Two-Way Highway Work Zones

Variable	Description	Values
c_o	Maximum discharge rate without work zone	2,600vph
c_w	Maximum discharge rate along work zone	1,200vph
n_a	Number of accidents per 100 million vehicle hour	40 acc/100mvh
Q_2	Hourly flow rate in Direction 2	500 veh/hr
Q_3	Hourly flow rate in Direction 3	500 veh/hr
V_w	Average work zone speed	50 km/hr
v_a	Average accident cost	142,000 $/accident
v	Value of user time	12 $/veh·hr

The optimized solutions for work zone length and total cost are shown in Table 3.6 for various traffic flows Q_1, from 100 vph to 2,600 vph. Note that the optimized length and minimized total cost are available even if the remaining Q_1 on the main road exceeds the work zone capacity 1,200 vph. For Alternatives 4.1, 4.2 (p=0.3 and 0.6), and

4.4, as Q_1 increases, the optimized zone length L^* decreases. Figure 3.15 shows that for Alternatives 4.1, 4.2(p=0.3), and 4.4, L^* decreases sharply as the remaining flow of Q_1 in Direction 1 exceeds the work zone capacity, because a queue is then formed and a much shorter zone length L is needed to avoid higher queue delays. Q_1 in Alternative 4.2 (p=0.3) is higher when L^* decreases because 30% of Q_1 has been diverted and the remaining flow is approaching the zone capacity. For Alternatives 4.2 (p=0.9) and 4.3, L^* stays almost constant at 5 km because almost all of Q_1 has been diverted, and the very slight remaining flow of Q_1 on the main road has almost no effect on delays due to the work zone. Therefore, the optimized L^* is the entire distance from A to B because it has the lowest maintenance cost and total cost.

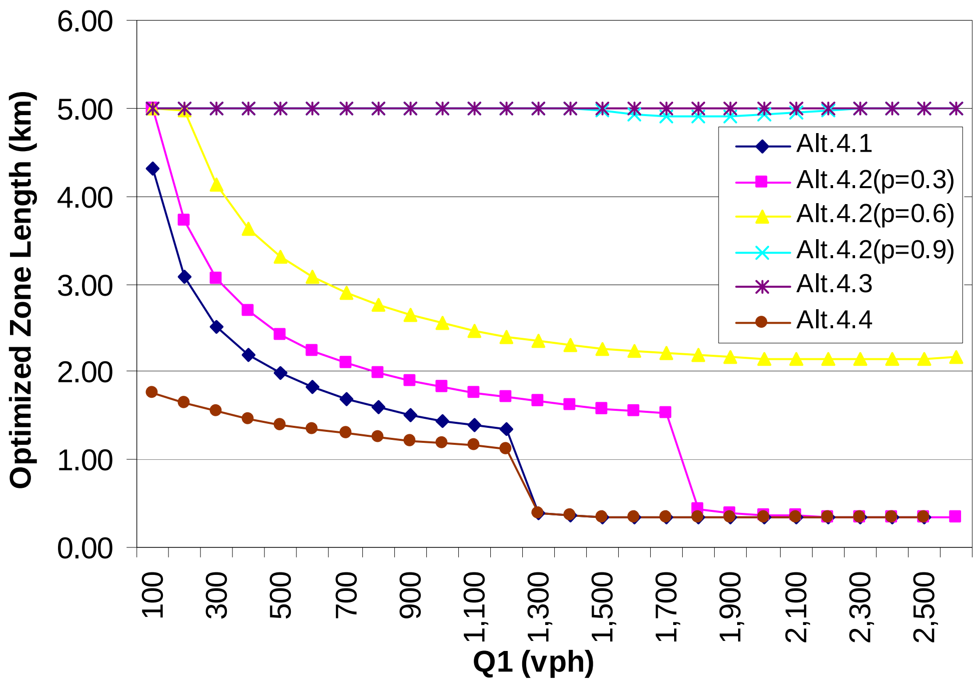


Figure 3.15 Optimized Zone Length vs. Q_1

To examine sensitivities to other factors, we fix the traffic flow rates Q_1 at 1,000 vehicles per hour (vph). Figure 3.16 shows increases in user cost as L increases in

Alternatives 4.1 and 4.4 because they have only one lane for discharging flow and no detours. A longer L only increases user delay costs. Alternatives 4.2 (p=0.3, 0.6, and 0.9) have their lowest user costs for zone lengths of 0.6 km, 1.0 km, and 2.5 km, respectively, since lower remaining flows on the maintained road justify longer L^* values. Alternative 4.3 has the lowest user delay cost and maximum L at 5 km since allof Q_1 has been diverted; the only moving delay occurs along the detour and it decreases due to reduced maintenance time per kilometer. Thus, a longer L shortens the maintenance time per kilometer and decreases user delay costs.

Table 3.6 Optimized work zone lengths (km) and Minimized Total Costs ($/lane.km) for Various Flow Rates

Q_1	Alt.4.1		Alt.4.2 (p=0.3)		Alt.4.2 (p=0.6)		Alt.4.2 (p=0.9)		Alt.4.3		Alt.4.4	
	Optimized length	Min. total cost	Optimized length	Min. total cost	Optimized length	Min. total cost	Optimized length	Min. total cost	Optimized length	Min. total cost	Optimized length	Min. total cost
100	4.32	80,481	5.00	80,439	5.00	80,392	5.00	80,346	5.00	80,331	1.76	81,242
200	3.07	80,687	3.71	80,648	4.97	80,582	5.00	80,493	5.00	80,465	1.64	81,343
300	2.52	80,846	3.06	80,818	4.13	80,758	5.00	80,642	5.00	80,601	1.54	81,436
400	2.20	80,980	2.68	80,968	3.63	80,921	5.00	80,791	5.00	80,741	1.46	81,522
500	1.98	81,098	2.43	81,103	3.31	81,073	5.00	80,942	5.00	80,883	1.40	81,602
600	1.82	81,203	2.24	81,227	3.07	81,218	5.00	81,094	5.00	81,029	1.34	81,677
700	1.69	81,299	2.10	81,343	2.89	81,357	5.00	81,247	5.00	81,179	1.29	81,747
800	1.59	81,386	1.99	81,451	2.75	81,491	5.00	81,402	5.00	81,333	1.25	81,813
900	1.51	81,467	1.90	81,552	2.64	81,620	5.00	81,559	5.00	81,490	1.21	81,874
1,000	1.45	81,541	1.82	81,647	2.54	81,746	5.00	81,717	5.00	81,652	1.18	81,932
1,100	1.39	81,610	1.76	81,736	2.47	81,867	5.00	81,876	5.00	81,819	1.15	81,985
1,200	1.34	81,674	1.70	81,819	2.40	81,984	5.00	82,038	5.00	81,991	1.13	82,035
1,300	0.39	114,198	1.65	81,896	2.35	82,097	5.00	82,201	5.00	82,169	0.39	114,476
1,400	0.36	150,510	1.61	81,967	2.30	82,206	5.00	82,367	5.00	82,352	0.36	150,921
1,500	0.35	193,334	1.58	82,033	2.27	82,311	4.97	82,534	5.00	82,543	0.35	193,914
1,600	0.34	244,690	1.55	82,092	2.23	82,411	4.94	82,704	5.00	82,740	0.34	245,483
1,700	0.34	307,441	1.52	82,145	2.21	82,506	4.92	82,877	5.00	82,946	0.34	308,501
1,800	0.34	385,866	0.44	101,978	2.19	82,597	4.91	83,052	5.00	83,161	0.34	387,270
1,900	0.34	486,686	0.38	125,530	2.17	82,681	4.91	83,231	5.00	83,385	0.34	488,541
2,000	0.34	621,103	0.36	151,665	2.16	82,760	4.92	83,413	5.00	83,621	0.34	623,570
2,100	0.34	809,276	0.36	180,984	2.15	82,832	4.94	83,598	5.00	83,869	0.34	812,611
2,200	0.34	1,091,527	0.35	214,147	2.14	82,896	4.98	83,788	5.00	84,132	0.34	,096,177
2,300	0.33	1,561,935	0.35	251,981	2.14	82,953	5.00	83,981	5.00	84,411	0.33	,568,791
2,400	0.33	2,502,739	0.34	295,559	2.15	82,999	5.00	84,180	5.00	84,710	0.33	,514,032
2,500	0.33	5,325,141	0.34	346,303	2.15	83,036	5.00	84,385	5.00	85,031	0.33	,349,777
2,600	-	-	0.34	406,147	-	-	5.00	84,596	5.00	85,380	-	-

As L increases, the maintenance costs per kilometer decreases due to fewer setups but stays the same for all alternatives. Combined with the user cost in Figure 3.16 and accident costs for four alternatives, the zone lengths that minimize total costs are determined by trade-offs among the maintenance, user, and accident costs. If we fix the traffic flow rates Q_1 at 1,000 vph, L^* is 1.45 km for Alternative 4.1, 1.82 km for Alternative 4.2 (p=0.3), 5.00 km for Alternative 4.3, and 1.18 km for Alternative 4.4, shown in Table 3.6 and Figure 3.17. Faster increases in the user cost of Alternative 4.4 shorten its L^*.

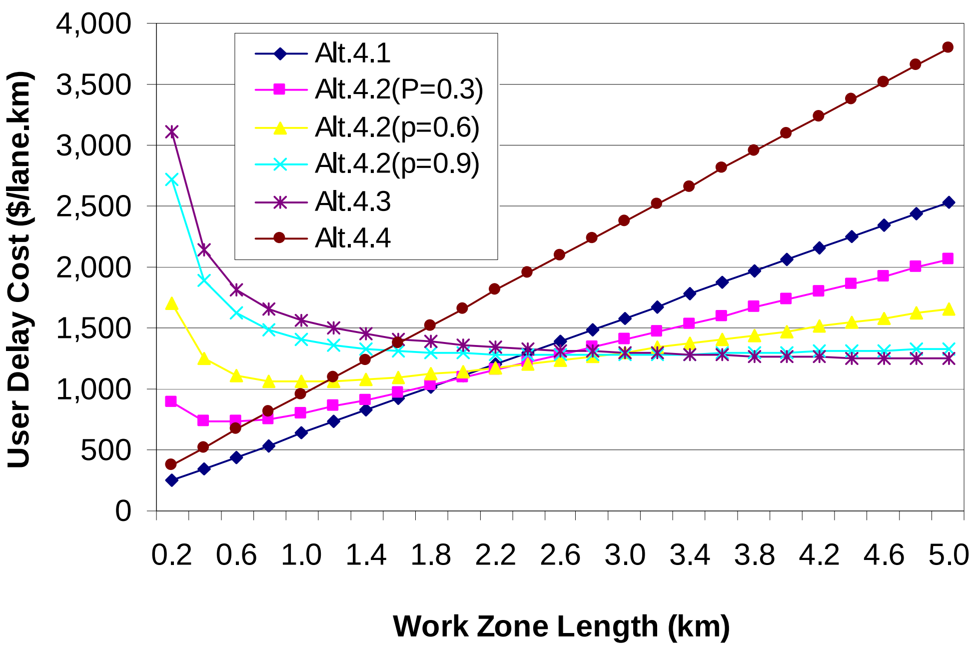


Figure 3.16 User Delay Cost vs. Work Zone Length (Q_1=1,000vph, Q_2=500vph, Q_3=500vph)

Figures 3.18 shows the relations between L^* and setup cost z_1. Thus, L^* increases when z_1 increases in Alternatives 4.1, 4.2 (p=0.3 and 0.6), and 4.4, because longer zones imply fewer setups and decreased total cost. In this case, the L^* of Alternatives 4.2 (p=0.9) and 4.3 are not sensitive to setup cost because L^* cannot exceed the full distance of the maintained road from A to B (5 km in this example) even though most theoretical

L^* values for Alternative 4.2 (p=0.9) exceed 5 km. In Alternative 4.3, total cost is minimized when L =5 km, regardless of other variables. Then, L^* of Alternative 4.3 is entirely unaffected by setup cost.

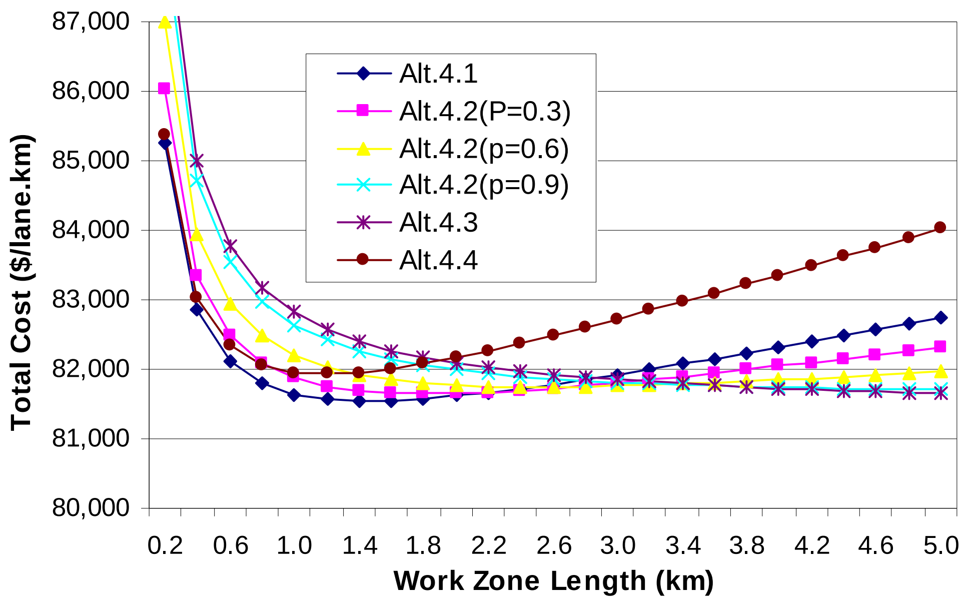


Figure 3.17 Total Cost vs. Work Zone Length (Q_1=1,000vph, Q_2=500vph, Q_3=500vph)

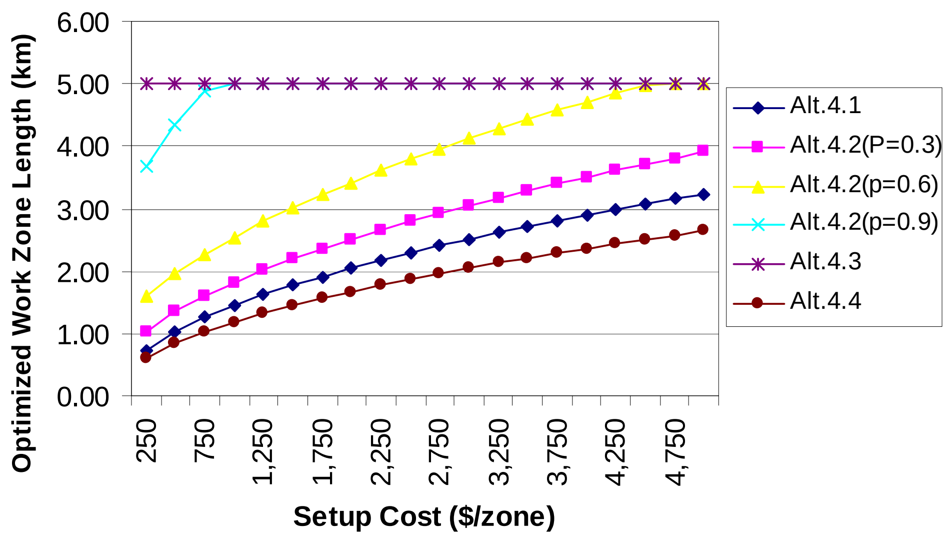


Figure 3.18 Optimized Work Zone Length vs. Setup Cost (Q_1=1,000vph, Q_2=500vph, Q_3=500vph)

Additional analysis of the sensitivity of L^* to setup duration, z_3, and average maintenance time, z_4, etc. is provided in Chen and Schonfeld (2001).

3.7.2 Selection Guidelines

Thresholds among alternatives with respect to several key variables, namely, traffic flow (Q_1), detour length (L_d), length of main road between the beginning and end of detour (L_t), setup cost (z_1), and average maintenance time per kilometer (z_4), etc. are solved numerically and presented below.

Figure 3.19 shows the relation between minimized total cost and Q_1. There are three flow thresholds and Alternatives 4.1, 4.2, 4.3 successively define the lowest cost envelope. The first threshold occurs at 800 vph, beyond which Alternative 4.1 becomes preferable to Alternative 4.3; beyond 1,200 vph Alternative 4.2 (p=0.3) becomes preferable to Alternative 4.1; beyond 1700 vph more diversion is preferable, such as Alternative 4.2 (p=0.6). This result can also be obtained from Table 3.6. The sharp increase occurs as Q_1 exceeds 1,200 vph in Alternative 4.1 and 1,700 vph in Alternative 4.2 (p=0.3) since the flow in Direction 1 exceeds work zone capacity and queue delays develop.

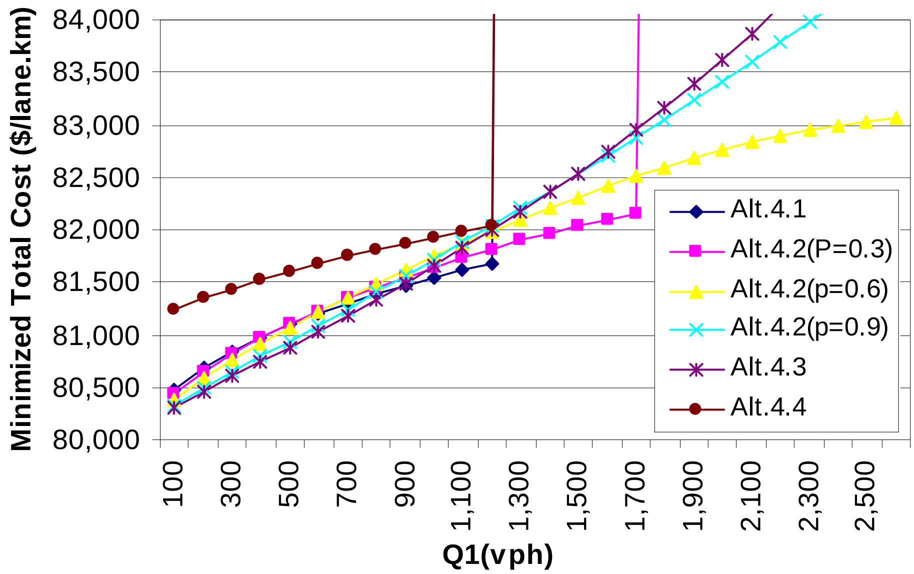
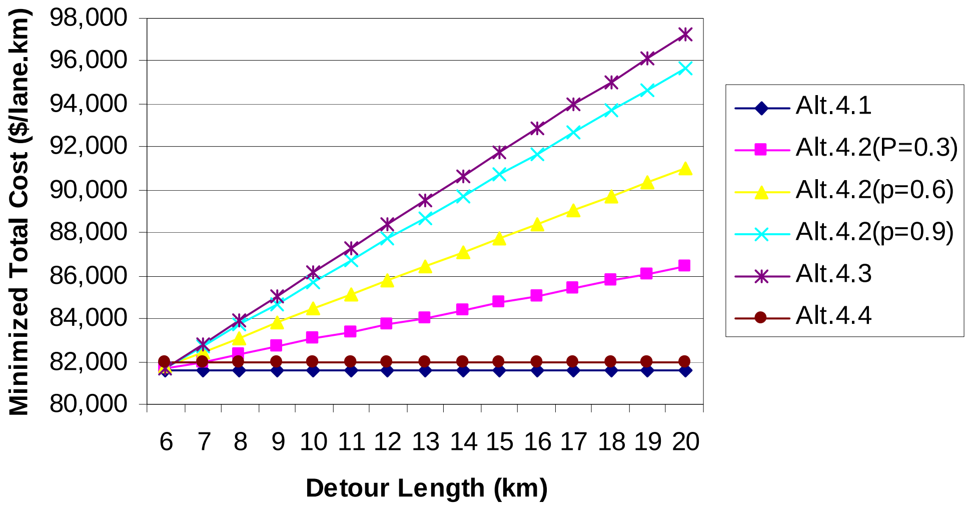


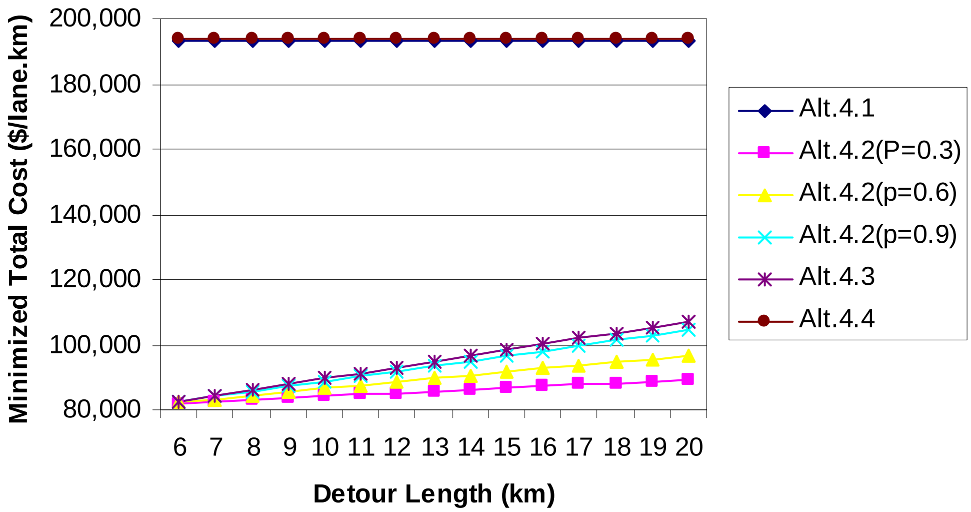
Figure 3.19 Minimized Total Cost vs. Q_1

Figure 3.20 shows the relation between minimized total cost and detour length in three cases: Q_1=1,000 vph, 1,500 vph, and 2,000 vph. There is no detour threshold in Figure 3.20; however, when Q_1 exceeds the maximum discharge rate along the work zone c_w, more diverted flow is preferable. The total costs in Alternatives 4.1 and 4.4, which have no detours, become quite high, as shown in Figures 3.20(b) and 3.20(c), as Q_1 exceeds c_w because queue delays develop and user delay costs increase sharply. Alternative 4.1 is preferable for Q_1=1,000 vph, Alternative 4.2 (p=0.3) is preferable for Q_1=1,500 vph and Alternative 4.2 (p=0.6) is preferable for Q_1=2,000 vph. Figure 3.20 shows that detour length affects the relative costs but not the rankings of alternatives.

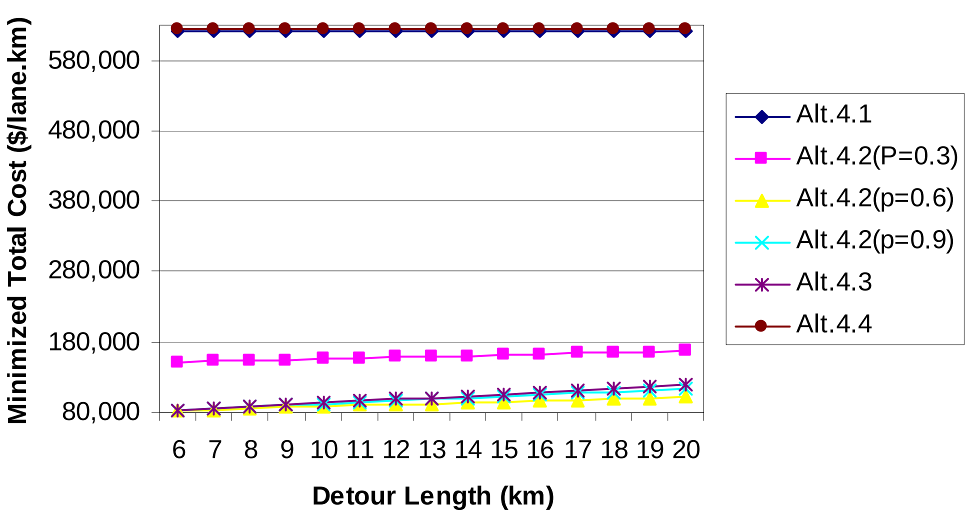
The thresholds with respect to other main variables, such as setup cost z_1, average maintenance time per kilometer, z_4, length of main road between the beginning and end of detour, L_t, etc. can be obtained similarly to traffic flow or setup cost thresholds.



(a)



(b)



(c)

Figure 3.20 Minimized Total Cost vs. Detour Length (a) Q_1=1,000 vph (b) Q_1=1,500 vph (c) Q_1=2,000 vph

3.7.3 Optimizing the Diverted Fraction

Figure 3.21 shows the relation between total cost and diverted fraction for different flow rates. When the flow Q_1 does not exceed maximum discharge rate along the work zone (1,200 vph) the total cost is lowest at boundary points of *p*, *p*=0 and 1.0. If Q_1 is between 0 and 800 vph, the minimized total cost occurs at *p*=1 (Alternative 4.3, diverted all Q_1 to detour); if Q_1 is between 800 vph and 1,200 vph, the minimized total cost occurs at *p*=0 (Alternative 4.1, no diversion). If the flow Q_1 exceeds the maximum discharge rate along the work zone (1,200 vph), the minimized total costs occur at the lowest points of *p*, *p*=0.2, 0.4, and 0.6 when flows Q_1 are 1,500, 2,000, and 2,500 vph, respectively. Note that 15,00*(1-0.2)=1,200 and 2,000*(1-0.4)=1,200, which indicate that total cost is minimized if any vehicles beyond 1,200 vph from Q_1 are detoured.

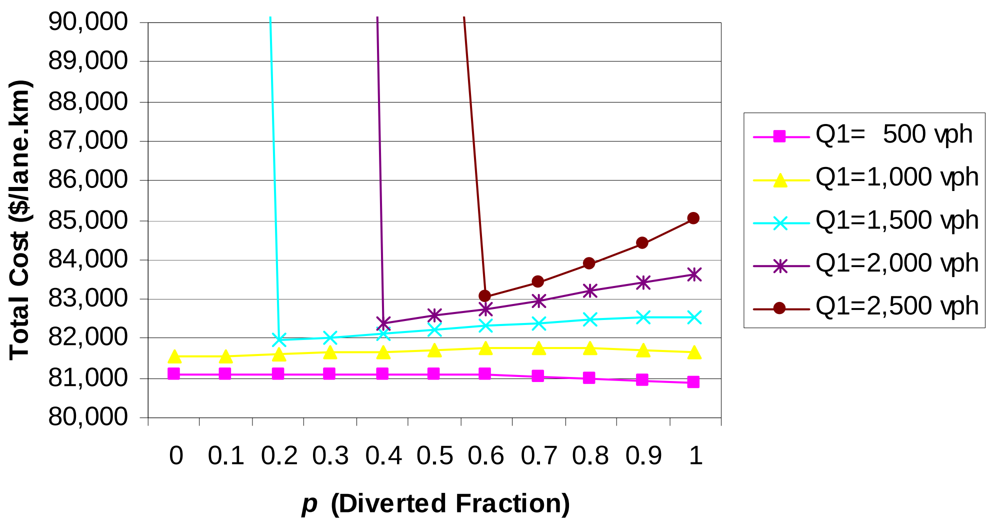


Figure 3.21 Total Cost vs. Diverted Fraction (Detour Length = 6km)

3.7.4 Summary

In this section work zone cost models are developed for four alternative zone configurations on four-lane roads, with and without an alternate route. The optimized zone length and preferred alternative are determined for various combinations of variables.

In the threshold analysis presented, traffic flow Q_1 and setup cost z_1 affect the rankings of alternatives. For example, in the flow threshold case, beyond the first threshold of 800 vph, Alternative 4.1 becomes preferable to Alternative 4.3; beyond the second threshold of 1,200 vph, Alternative 4.2 (p=0.3) becomes preferable to Alternative 4.1; beyond the third threshold of 1700 vph, Alternative 4.2 (p=0.6) becomes preferable to Alternative 4.2 (p=0.3). Alternative 4.4 might be selected only if an alternate road is unavailable and Q_2 is relatively low.

Chapter IV Work Zone Optimization for Time-Dependent Inflows

According to the previously developed steady-flow models (Sections 3.3 and 3.4), optimized work zone length is quite sensitive to traffic volume. A zone length and its related work duration optimized for one traffic level may be quite sub-optimal if traffic volumes change significantly before the work is completed. Therefore, a different methodology is needed to optimize the total cost under time-dependent inflows.

Chien et al. (2002) developed a model to optimize the scheduling of work zone activities associated with traffic control for two-lane two-way highways where one lane at a time is closed. However, their inflows are overly simplified and the "greedy" search approach used to determine each zone length tends to produce sub-optimal results. Jiang and Adeli (2003) used neural networks and simulated annealing to optimize only one work zone length and starting time for a four-lane freeway, considering factors such as darkness and numbers of lanes closed; however, a multiple-zone project were not considered. Complete scheduling plans for multiple-zone maintenance projects can be optimized with the method presented in this chapter. A methodology is developed here to optimize an entire work zone project under time-dependent inflows.

Efficient scheduling and traffic control through work zones may significantly reduce the total cost, including agency cost and user cost. Based on time-dependent inflows, the issues considered in this chapter include:

1. What is the best starting time for the project?
2. Into how many zones should the project be divided?
3. What are the best starting times for each zone?
4. What should be the length for each zone?

5. What should be the work duration for each zone?
6. Should the ending time of one work zone be the starting time of next work zone or should there be a work pause between some successive zones, based on the trade-offs among maintenance costs, user costs and idling costs?

One work zone plan example for time-dependent inflows is illustrated in Figure 4.1.

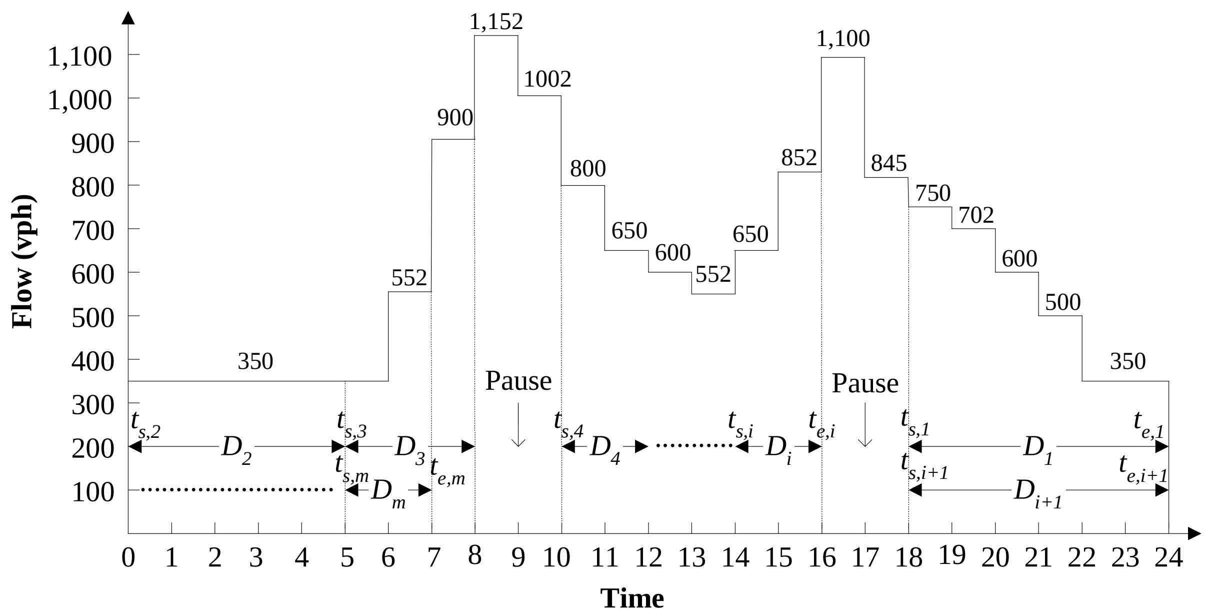


Figure 4.1 Work Zone Activities under Time-Dependent Inflows

A model for optimizing work plans, including zone lengths, work durations, starting times, pausing times (if any), and control cycle times (if two-lane highways) is presented in this chapter. This is done by minimizing total cost, including agency cost (maintenance cost and idling cost) and user cost (user delay cost and accident cost), while taking into account traffic demand variations over time. Two optimization methods, Powell's (Press et al., 1988) and Simulated Annealing (Kirkpatrick et al., 1983), are adapted for this problem and compared. In this chapter, work zone are optimized for

Alternative 2.1 (two-lane highways) and Alternative 4.1 (four-lane highways). Finally, the reliability of the Simulated Annealing algorithm is presented.

4.1 Work Zone Cost Function for Time-Dependent Inflows

4.1.1 Model Formulation– Two-Lane Two-Way Highways (Alternative 2.1)

Schonfeld and Chien (1999) developed a work zone cost function which includes user delay and maintenance cost for two-lane highways. Using deterministic queuing analysis for control cycles that alternate traffic directions past work zones, the queuing delays per cycle (each cycle having two phases, one for each direction of travel) incurred in the work zone are derived as follows:

$$Y_1 = \frac{1}{2} Q_1 (r + t_2)(t_1 + t_2) \tag{4.1}$$

$$Y_2 = \frac{1}{2} Q_2 (r + t_1)(t_1 + t_2) \tag{4.2}$$

$$t_1 = \frac{r(\frac{3600}{H} + Q_1 - Q_2)}{(\frac{3600}{H} - Q_1 - Q_2)} \tag{4.3}$$

$$t_2 = \frac{r(\frac{3600}{H} + Q_2 - Q_1)}{(\frac{3600}{H} - Q_1 - Q_2)} \tag{4.4}$$

Y_1 is delay per cycle in Direction 1 and Y_2 is delay per cycle in Direction 2. Note that t_1 is the discharge phase for servicing the traffic flow Q_1 in Direction 1, while t_2 is the discharge phase for servicing Direction 2. The average clearance time r is the work zone length L divided by the average vehicle moving speed V. Then:

$$Y_1 = \frac{2 \times 3600 \times Q_1 L^2 (\frac{3600}{H} - Q_1)}{HV^2 (\frac{3600}{H} - Q_1 - Q_2)^2} \tag{4.5}$$

$$Y_2 = \frac{2 \times 3600 \times Q_2 L^2 (\frac{3600}{H} - Q_2)}{HV^2 (\frac{3600}{H} - Q_1 - Q_2)^2} \tag{4.6}$$

Consider work zone *i* of length L_i, which is one of the zones on a maintained road. The number of cycles N_i for zone *i* is the maintenance duration for zone *i* divided by the cycle time. N_i can be obtained as:

$$N_i = \frac{D_i}{t_1^i + t_2^i} \tag{4.7}$$

In Eq.(4.7), t_1^i is the duration of the discharge phase in Direction 1 for work zone *i*, while t_2^i is the duration of the discharge phase in Direction 2 for zone *i*. D_i is the total maintenance duration for zone *i*, which is linear according to the assumption in Eq (3.3):

$$D_i = z_3 + z_4 L_i \tag{4.8}$$

The total queuing delay cost for work zone *i* is

$$C_{qi} = YN_i v = (Y_1 + Y_2)\frac{D_i}{t_1^i + t_2^i} v \tag{4.9}$$

where *Y* is total delay per cycle. Substituting Eqs.(4.1), (4.2), (4.3), (4.4) (4.8) into Eq.(4.9), we obtain:

$$C_{qi} = \frac{(z_3 + z_4 L_i) L_i [Q_1(\frac{3600}{H} - Q_1) + Q_2(\frac{3600}{H} - Q_2)] v}{V(\frac{3600}{H} - Q_1 - Q_2)} \tag{4.10}$$

The maintenance cost for work zone *i*, C_{mi}, is according to the assumption in Eq (3.4):

$$C_{mi} = z_1 + z_2 L_i \tag{4.11}$$

Then, the total cost for work zone i, C_{ti}, is

$$C_{ti} = C_{mi} + C_{qi} = z_1 + z_2 L_i + \frac{(z_3 + z_4 L_i) L_i [Q_1(\frac{3600}{H} - Q_1) + Q_2(\frac{3600}{H} - Q_2)] v}{V(\frac{3600}{H} - Q_1 - Q_2)} \tag{4.12}$$

where C_{ti} = total cost for work zone i; C_{mi} = maintenance cost for work zone i; C_{qi} = user queuing delay cost for zone i.

We consider the varying traffic flows in Directions 1 and 2 over one day. A maintenance project for a two-lane two-way road with total length L_T in one direction would be maintained by scheduling m work zones over the entire maintenance period. Assume that zone i (i = 1, 2, …., m) is resurfaced over n duration units (different zones would likely have different n values) and D_{ij} (j =1, 2, …., n) is a duration unit selected so that in it inflows stay appropriately constant, as shown in Figure 4.2. Then the duration for zone i, denoted D_i, is:

$$D_i = \sum_{j=1}^{n} D_{ij} \tag{4.13}$$

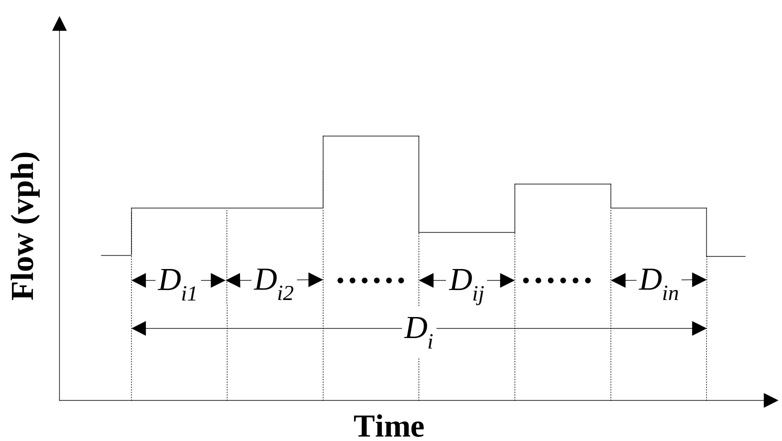


Figure 4.2 Duration for Work Zone *i* with Time-dependent Traffic Inflows

Here we assume that D_{ij} is a short duration unit for work zone activities that cannot be further subdivided, such as 0.06 hr. (Because the zone length unit is assumed to be 0.01 km in this study, duration unit = length unit * z_4 = 0.01 km * 6 hr/lane.km=0.06 hr.) Q_1^{ij} and Q_2^{ij} represent the varying traffic flows in Directions 1 and 2 during the period j for zone i. The number of cycles N_{ij} per traffic flow period is the duration of that period D_{ij} divided by the cycle time ($t_1^{ij} + t_2^{ij}$). N_{ij} can be obtained as:

$$N_{ij} = \frac{D_{ij}}{t_1^{ij} + t_2^{ij}} \tag{4.14}$$

where t_1^{ij} and t_2^{ij} are the discharge phases for traffic flows Q_1^{ij} in Direction 1 and Q_2^{ij} in Direction 2, respectively.

Then the user queuing delay cost for zone i can be formulated as:

$$C_{qi} = \sum_j^n Y^{ij} N_{ij} v \tag{4.15}$$

$$\begin{aligned} C_{qi} &= \sum_j^n (Y_1^{ij} + Y_2^{ij}) N_{ij} v \\ &= \sum_j^n \left[\frac{2 \times 3600 \times Q_1^{ij} L_i^2 (\frac{3600}{H} - Q_1^{ij})}{HV^2 (\frac{3600}{H} - Q_1^{ij} - Q_2^{ij})^2} + \frac{2 \times 3600 \times Q_2^{ij} L_i^2 (\frac{3600}{H} - Q_2^{ij})}{HV^2 (\frac{3600}{H} - Q_1^{ij} - Q_2^{ij})^2} \right] \frac{D_{ij}}{t_1^{ij} + t_2^{ij}} v \end{aligned} \tag{4.16}$$

where $$t_1^{ij} = \frac{r_i (\frac{3600}{H} + Q_1^{ij} - Q_2^{ij})}{(\frac{3600}{H} - Q_1^{ij} - Q_2^{ij})} \tag{4.17}$$

$$t_2^{ij} = \frac{r_i (\frac{3600}{H} + Q_2^{ij} - Q_1^{ij})}{(\frac{3600}{H} - Q_1^{ij} - Q_2^{ij})} \tag{4.18}$$

$$r_i = \frac{L_i}{V} \tag{4.19}$$

Eqs.(4.17) and (4.18) indicate that the one-way traffic control is time-dependent. The phases in Directions 1 and 2 are determined with the time-dependent flows Q_1^{ij} and Q_2^{ij}.

Substituting Eqs.(4.17), (4.18), (4.19) into Eq.(4.16), we obtain:

$$C_{qi} = \sum_{j}^{n} \frac{[\,Q_1^{ij}(\frac{3600}{H} - Q_1^{ij}) + Q_2^{ij}(\frac{3600}{H} - Q_2^{ij})\,]v}{V(\frac{3600}{H} - Q_1^{ij} - Q_2^{ij})} D_{ij} L_i \tag{4.20}$$

The moving delay cost of the traffic flows Q_1 and Q_2 in work zone i, denoted C_{vi}, is the cost increment due to the zone. The moving delay for zone i in each period D_{ij} of work zone duration D_i is equal to the flow ($Q_1 + Q_2$) multiplied by: (1) the period, D_{ij}, (2) the travel time difference over the zone length L_i with the work zone, $\frac{L_i}{V}$, and without the work zone, $\frac{L_i}{V_0}$, and (3) the value of time, v. Thus:

$$C_{vi} = \sum_{j}^{n} (Q_1^{ij} + Q_2^{ij}) D_{ij} (\frac{L_i}{V} - \frac{L_i}{V_0}) v \tag{4.21}$$

Idling cost is also considered in work zone activities with time-dependent inflows. This idling cost is equal to idling time multiplied by the average cost of idling time for crews and equipment. Idling time is a pause between two successive work zones, denoted $\Delta t_i = (t_{s,i} - t_{e,i-1})$. The idling cost per zone $C_{I\,i}$ is:

$$C_{I\,i} = v_d \Delta t_i \tag{4.22}$$

where v_d is average cost of idling time, $t_{s,i}$ is the starting time for zone i, and $t_{e,i-1}$ is the ending time for zone i-1. Note that Δt_i is 0 for i=1.

The accident cost incurred by the traffic passing the work zone can be determined from the number of accidents per 100 million vehicle hours n_a multiplied by the product of the increasing delay ($C_{qi}/v+C_{vi}/v$) and the average cost per accident v_a (Chien and Schonfeld, 2001), where C_{qi}/v is the queuing delay and C_{vi}/v is the moving delay for work zone i. The accident cost per work zone C_{ai} is formulated as:

$$C_{ai} = \frac{(C_{qi} + C_{vi})}{v} \frac{n_a v_a}{10^8} \tag{4.23}$$

The total cost for work zone i, C_{ti}, is

$$C_{ti} = (z_1 + z_2 L_i) + C_{qi} + C_{vi} + v_d \Delta t_i + \frac{(C_{qi} + C_{vi})}{v} \frac{n_a v_a}{10^8} \tag{4.24}$$

The total cost of the maintenance project for resurfacing road length L_T by scheduling m work zones, C_{PT} (\$/project), is expressed as:

$$\begin{aligned} C_{PT} &= \sum_{i}^{m} C_{ti} \\ &= \sum_{i}^{m} (z_1 + z_2 L_i) + \sum_{i}^{m} C_{qi} + \sum_{i}^{m} C_{vi} + \sum_{i}^{m} v_d \Delta t_i + \sum_{i}^{m} \frac{(C_{qi} + C_{vi})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{4.25}$$

The objective function is:

$$Min\ C_{PT} = Min \sum_{i}^{m} C_{ti} \tag{4.26}$$

subject to

$$\sum_{i}^{m} L_i = L_T \tag{4.27}$$

The total cost in Eq.(4.25) will be minimized with Powell's method as well as with the Simulated Annealing algorithm proposed in Section 4.2. Numerical analyses for two-lane highway work zones are presented in Section 4.3.

4.1.2 Model Formulation – Four-Lane Two-Way Highways (Alternative 4.1)

Chien and Schonfeld (2001) developed a work zone cost function, which includes the user delay, the accident, and the agency costs, for four-lane two-way highway without considering a detour (Figure 3.2(a)). The user delay cost consists of the queuing delay costs upstream of work zones and the moving delay costs through work zones. The equations of queuing delay and moving delay costs are shown in Section 3.4.2.

Consider the varying traffic flows in Directions 1 and 2 over one day. A maintenance project for a four-lane two-way road with total length L_T in one direction would be maintained by scheduling m work zones over the entire maintenance period. Assume that zone i (i = 1, 2, …., m) is resurfaced over n duration units (different zones would likely have different n values) and D_{ij} (j =1, 2, …., n) is a duration unit selected so that in it inflows stay appropriately constant, as shown in Figure 4.2.

Here we consider work zone i of length L, which is one of the zones along the total length L_T of a maintained road. Eq.(3.27), which estimates queuing delay cost for steady traffic inflows, cannot be applied directly for time-dependent inflows because it considers only one work zone, whose resulting queue might be dissipated after the zone is completed. In a multiple-zone project under time-dependent inflows, a new zone may begin immediately after the previous zone is completed; however, the queue is unlikely to be dissipated completely before next zone is started. In such a case, queuing delay costs for four-lane highway work zone are computed numerically. Queuing delay costs are illustrated here.

If flow Q_1^{ij} does not exceed c_w, the queuing delay is zero. Figure 4.3 shows the dissipation of queue length along zone duration if flow Q_1^{ij} exceeds c_w. Assume the queue due to work zone *i-1* has not been dissipated completely before zone *i* begins in Figure 4.3 and there exists queue length q_{i-1} as the zone *i* starts. The maximum queue length for zone *i* (area of A plus q_{i-1}) is:

$$q_{i,max} = q_{i-1} + (Q_1^{i1} - c_w)D_{i1} + (Q_1^{i2} - c_w)D_{i2} + \dots\dots..(Q_1^{i,j-1} - c_w)D_{i,j-1} \tag{4.28}$$

The area of A plus q_{i-1} is equal to the area of B, the number of dissipated vehicles. Figure 4.3 indicates that queue is dissipated completely before the next zone begins so that the work zone *i* is completed at $t_{e,i}$ while there is still a remaining dissipation time $t_{rd,i}$ for its zone. Then the queuing delay for work zone *i* is the area of C. The queuing delay cost for zone *i* is:

$$C_{qi} = (area\ of\ C)v \tag{4.29}$$

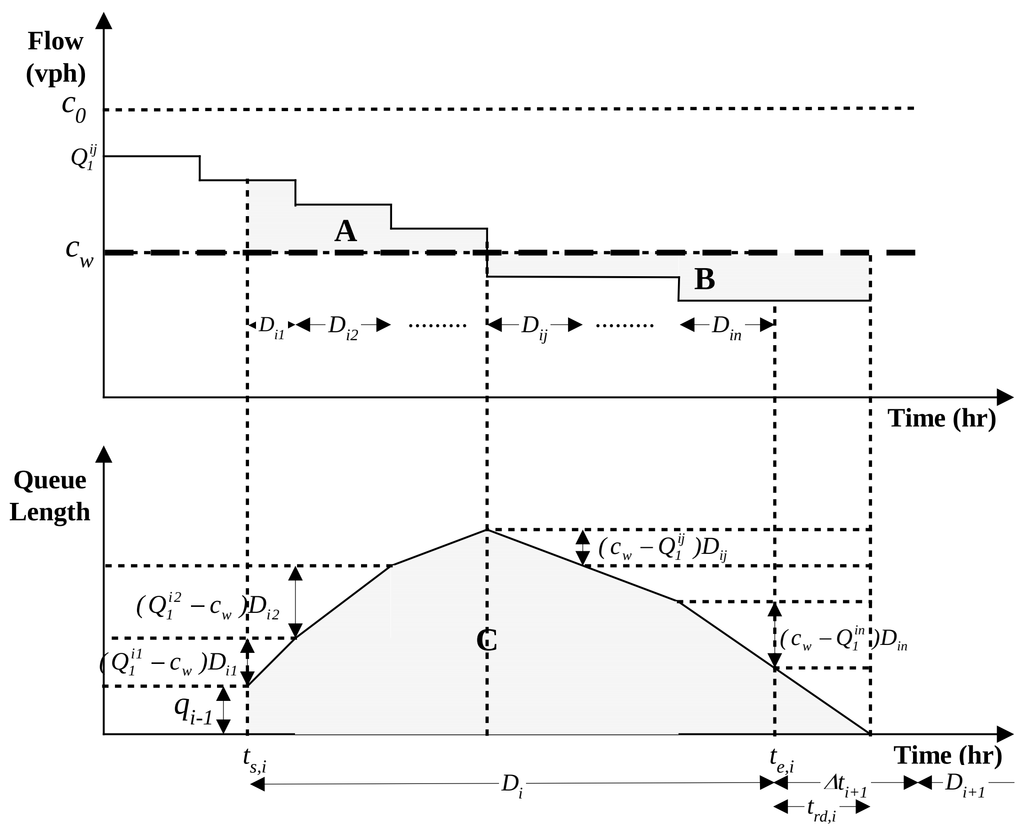


Figure 4.3 Queuing Delay and Queue Dissipation for Four-Lane Highway Work Zone

The moving delay cost of the traffic flows Q_1 in work zone i, denoted C_{vi}, is the cost increment due to the zone. It is the moving delay t_m^{ij} multiplied by the average delay cost v:

$$C_{vi} = \sum_{j=1}^{n} t_m^{ij} v \tag{4.30}$$

where t_m^{ij} = moving delay incurred by the approaching traffic flow Q_1^{ij} for zone i in each period D_{ij} of work zone duration D_i. t_m^{ij} is a function of the difference between the travel time on a road with and without a work zone:

$$t_m^{ij} = (\frac{L_i}{V_w} - \frac{L_i}{V_a}) Q_1^{ij} D_{ij} \qquad when \ Q_1^{ij} \le c_w \tag{4.31a}$$

$$t_m^{ij} = (\frac{L_i}{V_w} - \frac{L_i}{V_a}) c_w D_{ij} \qquad when \ Q_1^{ij} > c_w \tag{4.31b}$$

where V_a = average approaching speed; V_w = average work zone speed. If Q_1^{ij} is greater than c_w, the variable Q_1^{ij} is reduced by c_w (the maximum flow allowed to pass through the work zone).

Idling cost and accident cost have the same formulations as the Equations (4.22) and (4.23). The idling cost per zone C_{Ii} is:

$$C_{Ii} = v_d \Delta t_i \tag{4.22}$$

The accident cost per work zone C_{ai} is formulated as:

$$C_{ai} = \frac{(C_{qi} + C_{vi})}{v} \frac{n_a v_a}{10^8} \tag{4.23}$$

The maintenance cost for work zone i, C_{mi}, is according to assumption in Eq (3.4):

$$C_{mi} = z_1 + z_2 L_i \tag{4.11}$$

The total cost for work zone i, C_{ti}, is:

$$C_{ti} = (z_1 + z_2 L_i) + C_{qi} + C_{vi} + v_d \Delta t_i + \frac{(C_{qi} + C_{vi})}{v} \frac{n_a v_a}{10^8} \quad (4.32)$$

The total cost for resurfacing road length L_T by scheduling m work zones, C_{PT} (\$/project), is expressed as:

$$\begin{aligned} C_{PT} &= \sum_i^m C_{ti} \\ &= \sum_i^m (z_1 + z_2 L_i) + \sum_i^m C_{qi} + \sum_i^m C_{vi} + \sum_i^m v_d \Delta t_i + \sum_i^m \frac{(C_{qi} + C_{vi})}{v} \frac{n_a v_a}{10^8} \end{aligned} \quad (4.33)$$

The objective function is:

$$Min\ C_{PT} = Min \sum_i^m C_{ti} \quad (4.26)$$

subject to

$$\sum_i^m L_i = L_T \quad (4.27)$$

The total cost in Eq.(4.33) will be minimized with Powell's method and with the Simulated Annealing algorithm proposed in Section 4.2. Numerical analyses for four-lane highway work zones are presented in Section 4.4.

4.2 Optimization Methods

A good optimization method should usually reach a good solution quickly, without excessive memory requirements. Two optimization methods that were deemed suitable for this problem are adapted and compared here. One is a classic direction-set method, called Powell's Method (Press et al., 1988), and the other is a heuristic Simulated Annealing algorithm (Press et al., 1988, Kirkpatrick et al., 1983). The optimized variables of the total cost function include the work zones lengths L_i and starting times $t_{s,i}$ required to complete the project. The zone ending times $t_{e,i}$, the duration of maintenance pauses between two work zones Δt_i, and the time-dependent cycle lengths for discharging directional traffic over different time periods (if two-lane highways) can be uniquely determined from the optimized variables L_i and $t_{s,i}$.

4.2.1 Powell's Method

This method may be applied when derivatives of the objective function are difficult or impossible to specify. The basic concept of Powell's Method is as follows (Press et al., 1988): Take the unit vectors $\boldsymbol{e_1}$, $\boldsymbol{e_2}$, ….$\boldsymbol{e_N}$ as a set of directions. Using one-dimensional optimization, move along the first direction to the cost function's minimum, then from there along the second direction to its minimum, and so on, cycling through the whole set of directions as many times as necessary, until the function stops decreasing. The steps of Powell's Method are as follows:

Step 0: Initialize the set of directions $\boldsymbol{u_i}$ to basic vectors,

$\boldsymbol{u_i}=\boldsymbol{e_i}$ $\quad\quad$ $i=1, \ldots.., N$

Repeat the following sequence of steps until cost function stops decreasing.

Step 1: Save the starting position as $\boldsymbol{P_0}$.

Step 2: For i=1, …, N, move $\boldsymbol{P_{i-1}}$ to the minimum along direction $\boldsymbol{u_i}$ and call this point $\boldsymbol{P_i}$.

Step 3: For i=1, …, N-1, set $\boldsymbol{u_i} \leftarrow \boldsymbol{u_{i+1}}$.

Step 4: Set $\boldsymbol{u_N} \leftarrow \boldsymbol{P_N} - \boldsymbol{P_0}$.

Step 5: Move $\boldsymbol{P_N}$ to the minimum along direction $\boldsymbol{u_N}$ and call this point $\boldsymbol{P_0}$

In this study, work zone lengths and starting times are defined as vectors $\boldsymbol{e_i}$ because other variables, e.g. zone durations, ending times, can be derived from the relation between zone length and duration, shown in Assumption 3. The solution $\boldsymbol{P_i}$ is equal to (L_1, L_2,.., L_i,…, L_m, $t_{s,1}$, $t_{s,2}$,…, $t_{s,i}$, …, $t_{s,m}$), where m is the number of work zones. The sequence of directions for each successive iteration (step 1 to step 5) in searching for the minimized total cost is as follows: (L_1) → (L_2, $t_{s,2}$) → …. → (L_i, $t_{s,i}$) →… → (L_m, $t_{s,m}$). (L_i, $t_{s,i}$) indicates that zone length L_i and starting time $t_{s,i}$ are determined simultaneously. Note that $t_{s,1}$ is the project starting time, given from input data. The procedures from Step 1 to Step 5 are repeated until total cost stops decreasing.

4.2.2 Simulated Annealing Algorithm

Introduction

Simulated annealing (SA) is a stochastic computational technique derived from statistical mechanics for finding near globally optimum solutions to large optimization problems. It was developed by Metropolis (1953) to simulate the annealing process of crystals on a computer. Kirkpatrick et al. (1983) adapted this methodology to an algorithm exploiting the analogy between annealing solids and solving combinatorial optimization problems. The simulated annealing search process attempts to avoid becoming trapped at a local optimum by using a stochastic computational technique to find globally or near globally optimal solutions to combinatorial problems.

The original concept of SA from thermodynamics is that liquids freeze and crystallize, or metals cool and anneal. The SA algorithm is illustrated in pseudo-code in Table 4.1. Kirkpatrick et al. generalized an approach by introducing a multi-temperature approach in which the temperature is lowered slowly in stages. The outer loop (begin^1 …..end^1) in Table 4.1 indicates that the temperature T is lowered by updating T in each outer loop until T is less than or equal to T_f. The inner loop (begin^2 …..end^2) indicates that at each temperature the system repeats searching for a lower energy state until the system reaches equilibrium. A system in thermal equilibrium at temperature T has its energy probabilistically distributed, according to the Boltzmann probability distribution, $Prob(E) \sim \exp(-E/kT)$, where k is Boltzmann's constant (Metropolis, 1953). At each temperature a neighboring solution S' is chosen at random and the energy change (total cost change), Δ, is computed, where $\Delta = E(S') - E(S)$. $E(S')$ is the energy (total cost) of the new neighboring solution and $E(S)$ is the energy (total cost) of the previous solution. The new solution is accepted with the probability 1 if $\Delta \leq 0$, and with probability $e^{-\Delta/T}$ if $\Delta > 0$. Note that the simulated annealing procedure allows occasional "uphill moves" that have higher energy (total cost) than the current solution in order to avoid getting trapped at a locally optimal solution. These uphill moves are controlled probabilistically by the temperature T and become decreasingly likely toward the end of the process as T decreases (Press et al., 1988).

Table 4.1 Simulated Annealing Algorithm

```
Sub Anneal
    S = Initial solution S_0
    T = Initial temperature T_0
    Do while (T > T_f): (begin^1)
        Do while (not yet in equilibrium): (begin^2)
                S' := Some random neighboring solution of S
                Δ := E(S') – E(S) (or Δ := TC(S') – TC(S);)
                Prob := min (1, e^(-Δ/T))
                If  random(1,0) ≤ Prob then S:= S'
        Loop (end^2)
        Update T
    Loop (end^1)
    Output best solution
End Sub
```

[Wong, 1988, Modified by Chen, 2003]

Simulated Annealing Algorithm for Work Zone Optimization

The SA algorithm adapted here for work zone optimization is as follows:

Step 0. Generate an initial solution. Calculate average flow volume between two peak traffic periods, $\overline{Q}$. Given a project starting time, the initial work zone length L_i and duration D_i can be obtained by using the traffic volume $\overline{Q}$ for each stage and optimizing for steady traffic inflows using steady-demand model in Chapter 3. Here a stage is the period between two adjacent peak traffic volumes. The stage duration is denoted $D_{s,l}$, l=1, 2, …, as shown in Figure 4.13(b). The number of zones in each stage depends on how many D_i can be contained within the stage duration. The solution $S=(L_1, L_2,.., L_i,\ldots, L_m, t_{s,1}, t_{s,2},\ldots, t_{s,i}, \ldots, t_{s,m})$ is the initial solution for work zone lengths and starting times. Set j=1 and k=1, j=1 to J_{max} and k=1 to K_{max}. Set the values of T_0 and T_f .

Step 1. Generate a neighboring solution. Randomly generate four numbers: n_1, n_2, n_3, and n_4. n_1 and n_2 are two zones chosen randomly from all work zones in the previous solution. n_1 or n_2 is equal to $1+int(m*r)$, where int is a function that takes only the integer part of a real number; r is a uniform random number between 0 and 1. n_3 is a binary random number; in it 0 indicates that zone length decreases by one unit in zone n_1 and increases by one unit in zone n_2 while 1 indicates zone length increases by one unit in zone n_1 and decreases by one unit in zone n_2. n_4 is a binary random number, in which 0 or 1 indicates that an "increasing event" or "decreasing event" occurs in the end or in the beginning of zones, respectively. When zone n_1 is randomly chosen, $i=n_1$, and that zone length increases or decreases by one unit, from L_i to $L_i^{'}$, while zone n_2 will decrease or increase by one unit, from L_j to $L_j^{'}$, to keep the total project length unchanged. Other zone lengths stay unchanged. The details for "Increase" (including "Increase in end" and "Increase in begin"), "Decrease" (including "Decease in end" and "Decrease in begin"), "Check last zone", and "Delete zone", are shown from Figures 4.5 to 4.12. The neighboring solution $S'=(L_1, L_2,.., L_i^{'},.. L_j^{'},.., t_{s,1}, t_{s,2}, \ldots t_{s,i}^{'}, \ldots. t_{s,j}^{'}, \ldots t_{s,m})$ is generated after one "Decrease" event and one "Increase" event. Compute the objective function value and the difference between the new and previous total costs, $\Delta TC = TC(S^{'}) - TC(S)$. If $\Delta TC<0$, go to Step 3. Otherwise, go to Step 2.

Step 2. ($\Delta TC>0$) Select a random variable $\alpha \in U(0,1)$. If $\alpha < Prob(\Delta TC) \equiv exp(-\Delta TC / T_j)$, then go to Step 3. If $\alpha \geq Prob(\Delta TC) \equiv exp(-\Delta TC / T_j)$, then reject this new solution and go to Step 4.

Step 3 ($\Delta TC<0$ or $\alpha < Prob(\Delta TC)$) Accept the new solution *S'* and new total cost *TC*(*S'*). Store the new solution and total cost.

Step 4 If $T_j>T_f$ and $k<K_{max}$, then $k=k+1$ and go to Step 1, else if $T_j>T_f$ and $k=K_{max}$, then reduce T_j, $j=j+1$, $k=1$, and go to Step1. Otherwise, stop.

The flow chart of simulated annealing algorithm for work zone optimization is shown in Figure 4.4.

The new variables shown in Figures 4.4 to 4.12 are defined as follows:

$D_{s,l}$: duration of Stage *l*;

J_{max}: number of iterations for reducing temperature from T_0 to T_f;

K_{max}: maximum number of iterations for temperature T_j to equilibrium;

L_{assign}: deleted last zone length divided by *m-1*, which is averagely assigned to the previous *m-1* zones;

L_{avg}: average zone length in current solution;

L_{min}: minimum zone length in current solution;

L_R: project remaining length;

L_T: project length;

m: number of work zones of a maintained project;

N_{limit}: maximum number of successful iterations for temperature T_j to equilibrium;

N_{succ}: cumulative number of successful iterations for temperature T_j to equilibrium;

$N_{r,succ}$: cumulative number of successful iterations for repeating generating neighbor solution using the same random numbers under temperature T_j;

T_f : final temperature;

T_0: initial temperature;

ΔD: duration unit for increasing or decreasing a unit length, $\Delta D=\Delta L*z_4$;

ΔD_r: duration difference between new $t_{e,\ i}$ and old $t_{s,\ i+1}$ when new $t_{e,\ i}$ exceeds old $t_{s,\ i+1}$;

ΔL: length unit for increasing or decreasing, baseline=0.01km;

ΔL_r: length difference between length unit and the remaining length of the deleted zone;

Δt_i: idle time between zone i and zone $i\text{-}1$;

$\sum_i \Delta t_i$: cumulative idle times from zone 1 to zone i;

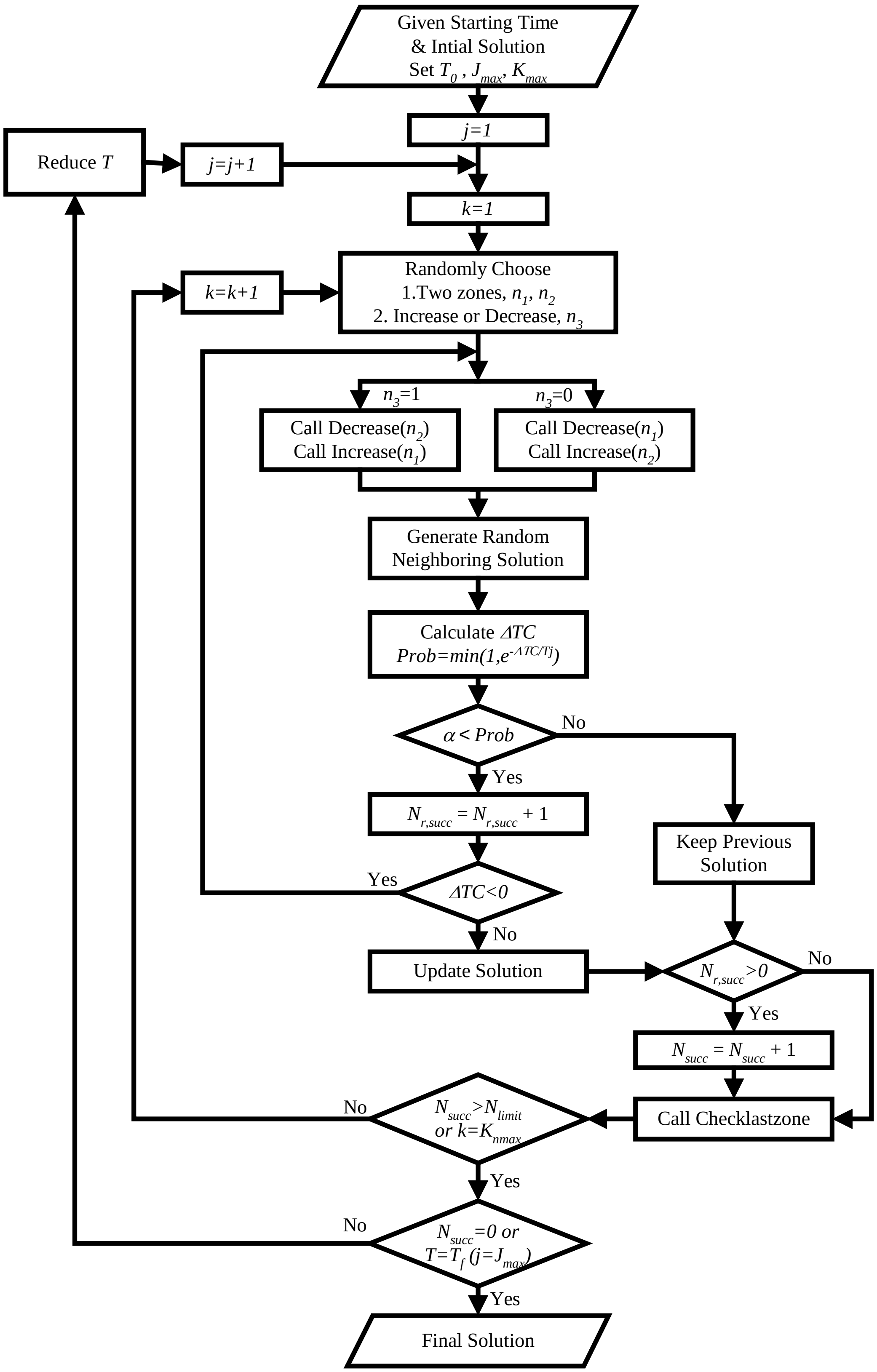


Figure 4.4 Flow Chart of Simulated Annealing Algorithm for Work Zone Optimization

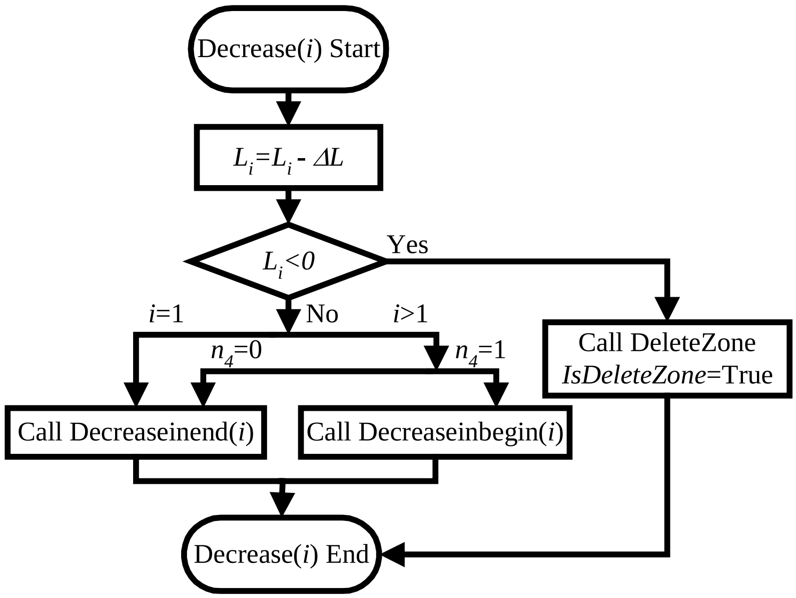


Figure 4.5 Decrease Event

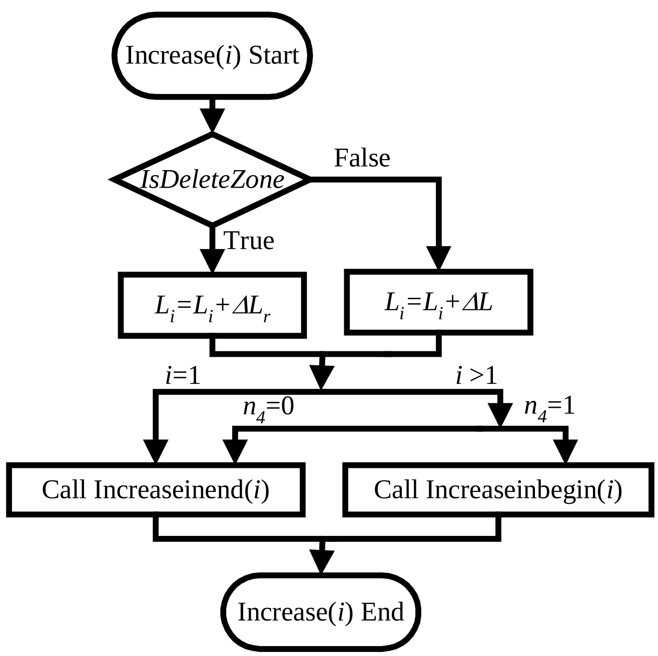


Figure 4.6 Increase Event

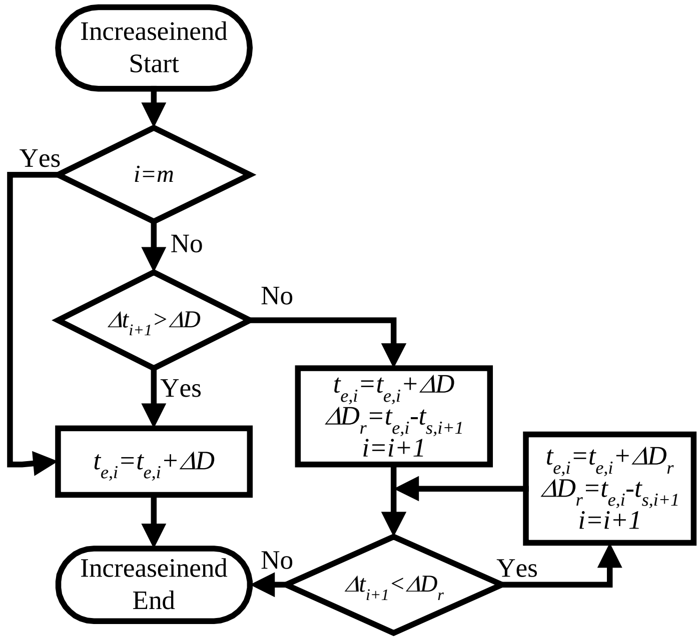


Figure 4.7 “Increaseinend” Event

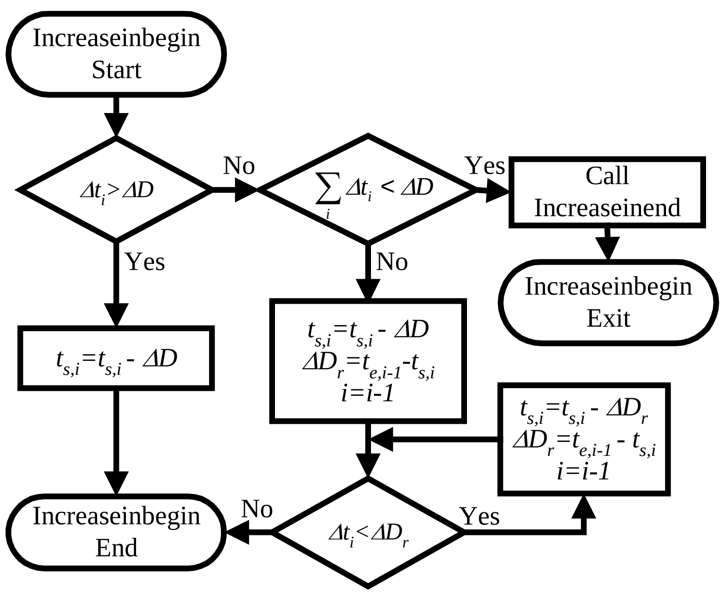


Figure 4.8 “Increaseinbegin” Event

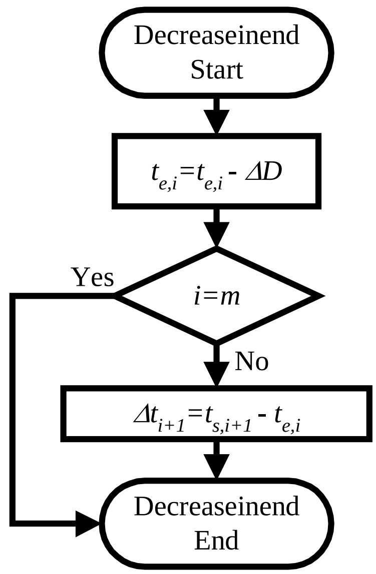


Figure 4.9 "Decreasinend" Event

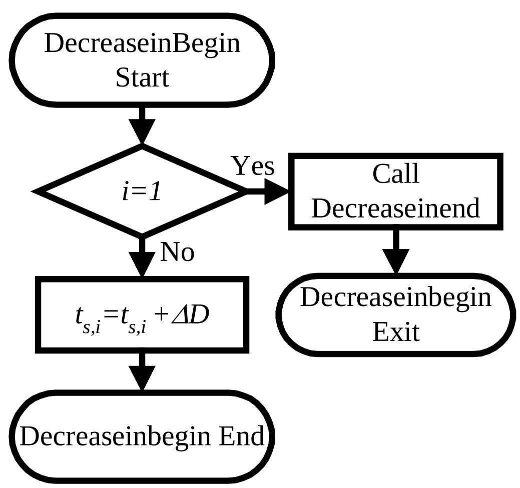


Figure 4.10 "Decreaseinbegin" Event

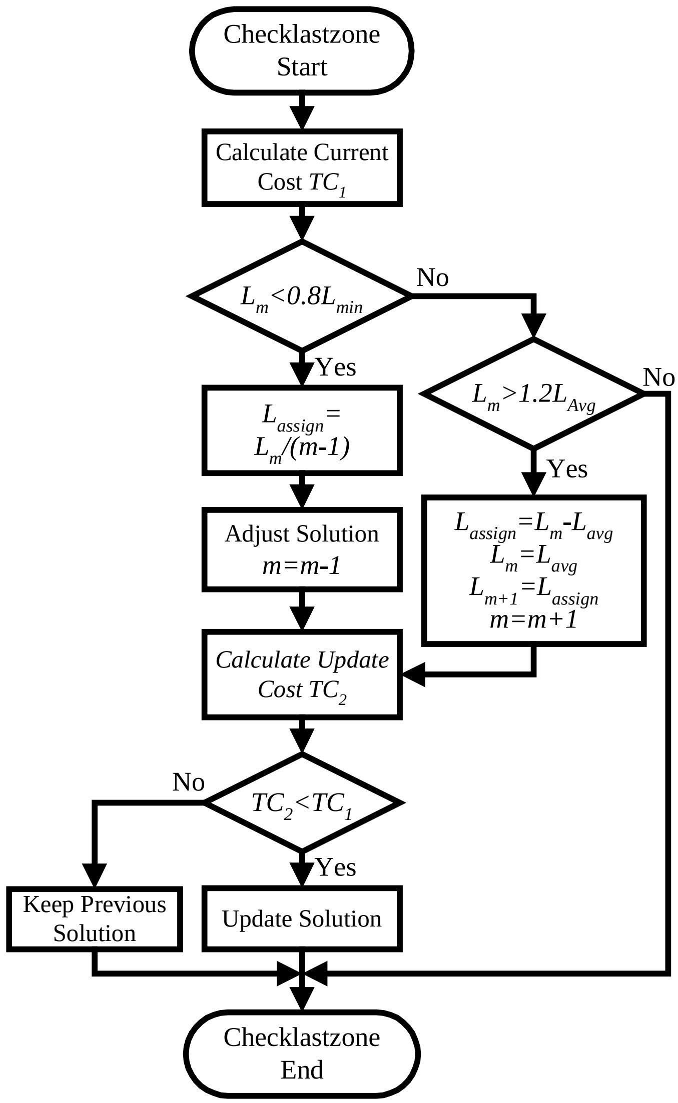


Figure 4.11 "Checklastzone" Event

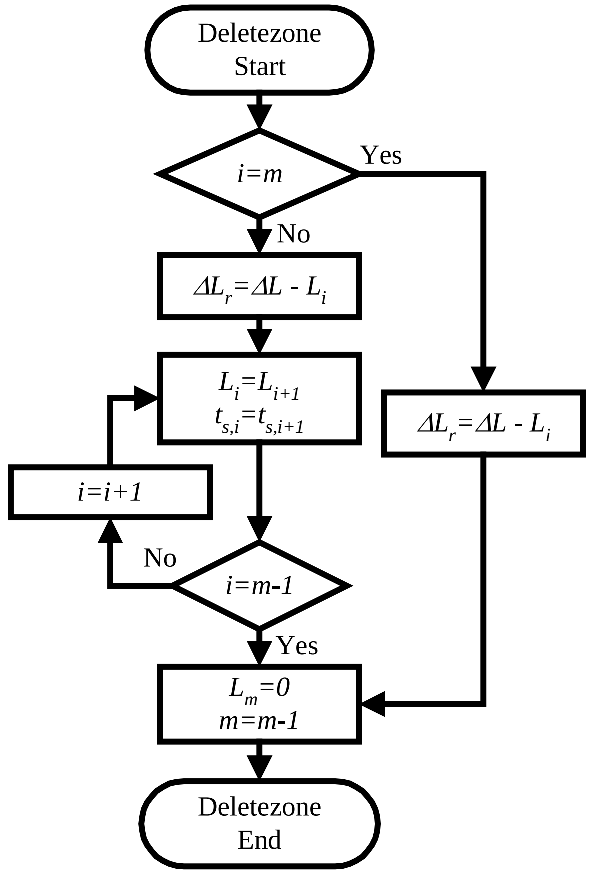
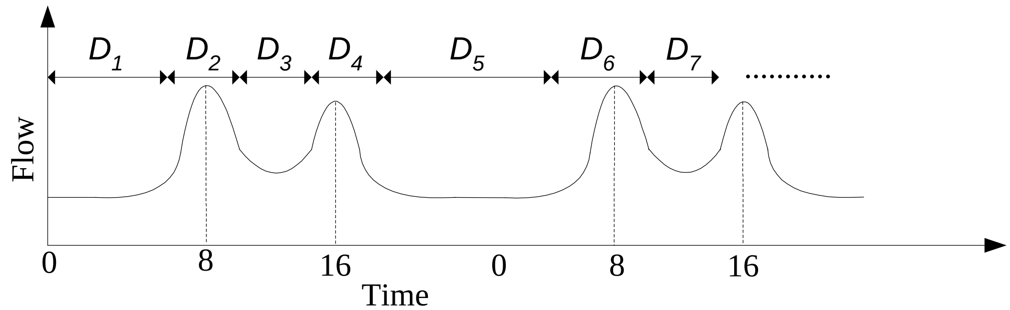


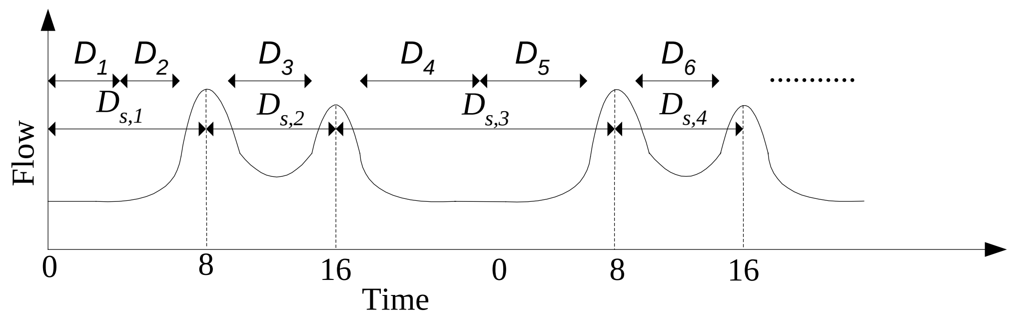
Figure 4.12 "Deletezone" Event

Optimization Solutions

One policy for work zone optimization is to work continuously over time without any pause between successive zones, as shown in Figure 4.13(a). The alternative policy is that pauses between zones during peak traffic periods are allowable, as shown in Figure 4.13(b).



(a) **Without Pauses**



(b) **With Pauses**

Figure 4.13 Work Zone Durations

Optimizing Best Project Starting Time

The proposed SA algorithm is based on a given project starting time. Figure 4.13 shows the procedure for finding the best project starting time, indicated by $t_{s,1}$, the starting time of first zone. The best start time of the entire project can be determined by comparing all minimized total costs corresponding to different project start times.

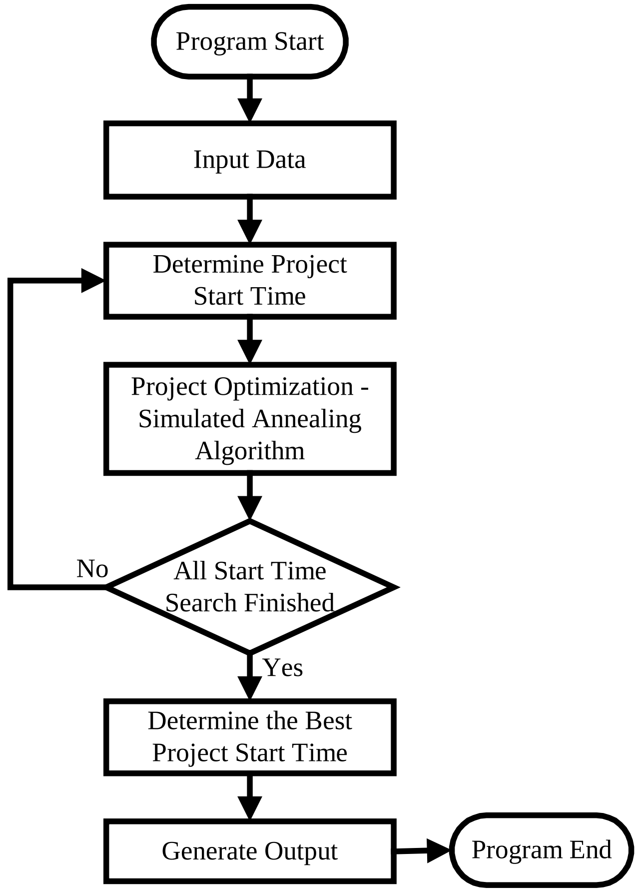


Figure 4.14 Search for Best Project Starting Time

4.3 Numerical Analysis – Two-Lane Two-Way Highway

The effects of various parameters on work zone lengths and starting times for two-lane highway work zones are examined in this section. The baseline numerical values for each variable in this section are defined in Table 4.2. A numerical example sequences and schedules unequal work zones for a 7.5-km maintenance project on a two-lane highway. Table 4.3 shows the hourly traffic distribution on the maintained road. The annual average daily traffic (AADT) is 15,000 vehicles. Two daily peak periods are shown in Figure 4.15.

Table 4.2 Notation and Baseline Numerical Inputs for Two-Lane Two-Way Highway Work Zones

Variables	Description	Input Values
H	Average headway through work zone area	3 s
$AADT$	Annual average daily traffic on Main Road	15,000
L_i	Zone length for zone i	-
L_T	Project road length	7.5 km
n_a	Number of accidents per 100 million vehicle hours	40 acc/100mvh
V	Average work zone speed	50 km/hr
v	Value of user time	12 \$/veh·hr
v_a	Average accident cost	142,000 \$/accident
v_d	Average Cost of Idling Time	800 \$/hr
z_1	Fixed setup cost	1,000 \$/zone
z_2	Average maintenance cost per lane·kilometer	80,000 \$/lane·km
z_3	Fixed setup time	2 hr/zone
z_4	Average maintenance time per lane·kilometer	6 hr/lane·km

Table 4.3 AADT and Hourly Traffic Distribution on a Two-Lane Two-Way Highway

Hour	Volume (Both Direction)	% of AADT	% of Direction1	Q_1 (vph)	Q_2 (vph)
0	349	2.33%	0.48	167	182
1	350	2.33%	0.48	168	182
2	349	2.33%	0.45	157	192
3	350	2.33%	0.53	185	165
4	349	2.33%	0.53	185	164
5	350	2.33%	0.53	186	164
6	552	3.68%	0.57	315	237
7	900	6.00%	0.56	504	396
8	1,152	7.68%	0.56	645	507
9	1,002	6.68%	0.54	541	461
10	800	5.33%	0.51	408	392
11	649	4.33%	0.51	331	318
12	600	4.00%	0.50	300	300
13	552	3.68%	0.52	287	265
14	650	4.33%	0.51	332	318
15	852	5.68%	0.53	452	400
16	1,100	7.33%	0.49	539	561
17	844	5.63%	0.47	397	447
18	750	5.00%	0.47	353	397
19	702	4.68%	0.47	330	372
20	600	4.00%	0.46	276	324
21	500	3.33%	0.48	240	260
22	349	2.33%	0.48	167	182
23	349	2.33%	0.48	167	182
AADT	**15,000**	**100.00%**	**-**	**7,632**	**7,368**

Compared to Powell's Method, we find in Figure 4.15 that for most project starting times considered (18 of 24) SA finds lower total costs while using less computer time (3 minutes with SA vs. 20 minutes with Powell's). Two algorithms are implemented in Visual Basic 6.0 and tested on a personal computer with a 1.8GHz Pentium 4 CPU and 512 MB memory. Two different but almost equally good project starting times are found by using the SA optimization process. The first best project starting time is 11:00. Its minimized total cost is $627,714/project, with nine work zones whose optimized lengths of 0.53, 0.76, 1.07, 0.82, 1.76, 1.08, 0.71, 0.45, and 1.34 km add up to 7.5 km, and whose idling time is 3.96 hours, as shown in Table 4.4(a). The second best project starts at 17:00. Its minimized total cost is $627,753/project, with eight zones whose optimized lengths of 0.80, 1.03, 0.77, 0.55, 1.50, 0.77, 0.56, and 1.49 km add up to 7.5 km, and whose idling time is 1.97 hours, as shown in Table 4.4(b). Thus, the solution starting at 17:00 has fewer (8 vs. 9) but longer work zones. When starting at 11:00 the first zone is shortened to avoid the afternoon peak period, during which there is a pause. The 17:00 start has already avoided the afternoon peak; it schedules less idling than the 11:00 start. Both cases have pauses during the morning peak, which has the highest traffic flow of the day. In Table 4.4(a) and (b), the agency cost, including maintenance cost and idling cost, is higher if starting at 11:00 ($612,167) than at 17:00 ($609,580). However, the user cost, including queuing delay cost, moving delay cost, and accident cost, is lower ($15,547) for starting at 11:00 than at 17:00 ($18,174). Such tradeoffs between agency costs and user costs should be carefully considered in project scheduling.

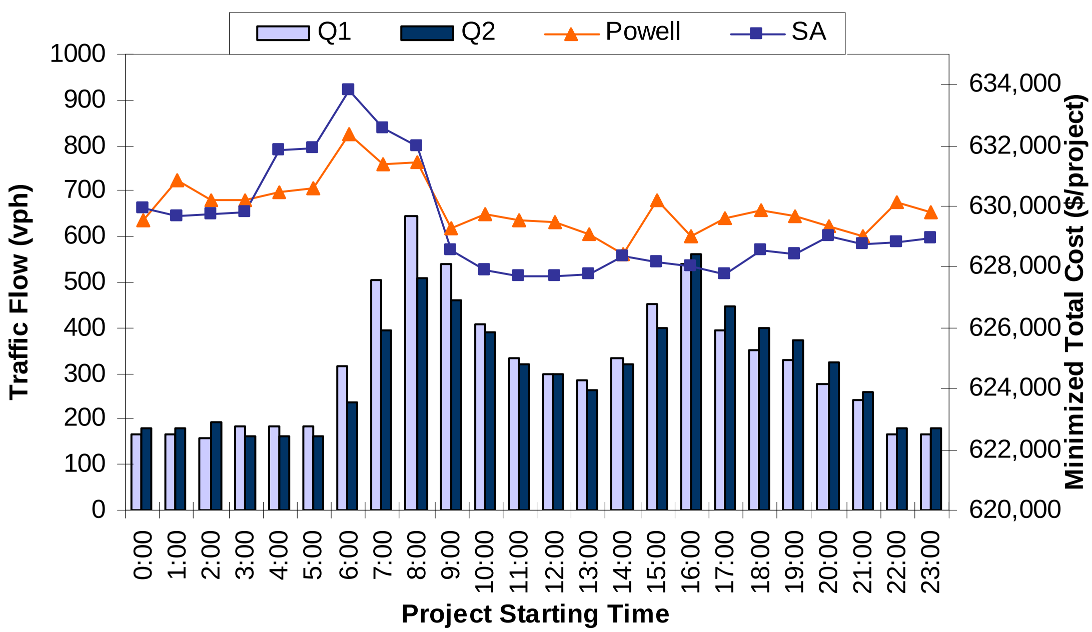


Figure 4.15 Hourly Traffic Distributions on Two-Lane Highway and Minimized Total Cost vs. Project Starting Time

Table 4.4(a) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 11:00, v_d=\$800/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.53	5.17	11.00	16.17	-	44,251
2	0.76	6.55	17.01	23.55	0.84	63,766
3	1.07	8.41	23.55	7.96	0.00	88,218
4	0.82	6.91	9.06	15.96	1.10	69,723
5	0.76	6.55	16.99	23.53	1.02	63,933
6	1.08	8.47	23.53	8.00	0.00	89,063
7	0.71	6.25	9.00	15.25	1.00	60,292
8	0.45	4.69	15.25	19.93	0.00	38,366
9	1.34	10.03	19.93	5.96	0.00	110,103
Total	7.50	63.00			3.96	627,714
Maintenance cost						609,000
Queuing delay cost						12,862
Moving delay cost						2,612
Idling cost						3,167
Accident Cost						73
Total cost						627,714
Total cost/project-km ($/lane-km)						83,695

Table 4.4(b) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 17:00, v_d=\$800/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0~23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.80	6.82	17.00	23.82	-	66,893
2	1.03	8.20	23.82	8.02	0.00	85,521
3	0.77	6.64	9.02	15.67	1.00	65,894
4	0.55	5.32	15.67	20.99	0.00	47,246
5	1.50	11.02	20.99	8.01	0.00	124,550
6	0.77	6.64	8.99	15.63	0.98	65,915
7	0.56	5.38	15.63	21.01	0.00	48,095
8	1.49	10.96	21.01	7.97	0.00	123,639
Total	7.50	61.00			1.97	627,753
Maintenance cost						608,000
Queuing delay cost						15,182
Moving delay cost						2,906
Idling cost						1,580
Accident Cost						86
Total cost						627,753
Total cost/project-km ($/lane-km)						83,699

Figure 4.16 shows that the total project duration decreases as the average cost of idling time v_d increases. However, for projects starting at 11:00 the project duration is not sensitive to v_d when v_d exceeds \$1,900/hr. Because there are no pauses, total project duration cannot decrease even when v_d increases. If v_d is below \$1,900/hr, maintenance activities should be interrupted during peak periods to avoid user delay costs that exceed idling costs. 9 zones and 63 hours without pauses are scheduled when v_d exceeds \$1,900/hr. Table 4.5(a) shows the optimized solution when v_d is \$2,000/hr. This solution corresponds to the first policy shown in Figure 4.13(a), in which work zones are worked continuously without any pause. Table 4.5(b) shows the optimized solution when v_d is \$200/hr. This corresponds to the second policy shown in Figure 4.13(b), which allows pauses between zones during peak traffic periods.

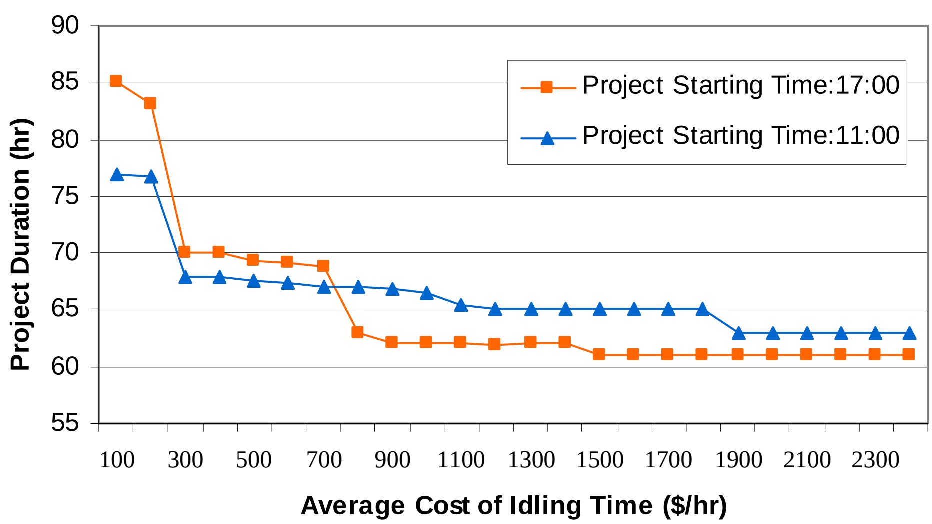


Figure 4.16 Project Duration vs. Average Cost of Idling Time

Table 4.5(a) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 11:00, v_d = $2000/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.54	5.25	11.00	16.25	-	45520
2	0.46	4.77	16.25	21.03	0.00	39313
3	1.36	10.17	21.03	7.20	0.00	112187
4	0.30	3.81	7.20	11.01	0.00	26853
5	0.94	7.65	11.01	18.67	0.00	80367
6	1.72	12.33	18.67	7.00	0.00	142662
7	0.22	3.33	7.00	10.33	0.00	19969
8	0.78	6.69	10.33	17.03	0.00	66529
9	1.16	8.97	17.03	2.00	0.00	96600
Total	7.50	63.00			0.00	630,000
Maintenance cost						609,000
Queuing delay cost						17,849
Moving delay cost						3,053
Idling cost						0
Accident Cost						99
Total cost						630,000
Total cost/project-km ($/lane-km)						84,000

Table 4.5(b) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 11:00, v_d = $200/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0~23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.50	4.99	11.00	15.99	-	41,741
2	0.53	5.17	16.98	22.15	0.98	44,469
3	1.15	8.89	22.15	7.05	0.00	94,420
4	0.67	6.01	9.98	16.00	2.94	56,523
5	0.53	5.17	16.98	22.15	0.98	44,468
6	1.15	8.89	22.15	7.05	0.00	94,420
7	0.67	6.01	9.98	16.00	2.94	56,523
8	0.53	5.17	16.98	22.15	0.98	44,468
9	1.15	8.89	22.15	7.05	0.00	94,420
10	0.63	5.77	9.98	15.76	2.94	53,159
Total	7.50	65.00			11.76	624,612
Maintenance cost						610,000
Queuing delay cost						9,880
Moving delay cost						2,323
Idling cost						2,351
Accident Cost						58
Total cost						624,612
Total cost/project-km ($/lane-km)						83,282

Figure 4.17 shows that fewer and longer zones are optimized as v_d increases. This reduces maintenance costs but increases user costs. We find a decrease from 10 to 9 zones (in Figure 4.17) and a decrease in project duration (in Figure 4.16) when v_d increases from \$200/hr to \$300/hr, because fewer zones decrease the setup duration and total project duration. In Figure 4.15, with baseline inputs, we find 11:00 to be the best project starting time. Figure 4.18 indicates that for v_d between \$300/hr and \$900/hr, 11:00 is the best project starting time. Outside that range, 17:00 is preferable. Note that three total cost drops occur at v_d = 700 to 800, 1400 to 1500 (project starting time =11:00), and 1800 to 1900 (project starting time=17:00). These total cost drops are consistent with the project duration drops in Figure 4.16 because increased v_d decreases idling time thereby decreasing idling costs more than it increases user delay cost.

Table 4.4(a) and Tables 4.6 (a)-(c) show the optimized results for two-lane highway work zones using values for z_2 (the average maintenance cost per lane-kilometer) of \$80,000, \$10,000, \$5,000, and \$100 per lane-km, respectively. These tables indicate that z_2 has very slight influence on optimized zone length and user delay cost. Similar results have been obtained from Equation (3.5), in which the optimal zone length is affected by both traffic volumes Q_1, Q_2, fixed setup cost z_1, average maintenance time per lane-kilometer z_4. Although that equation applies to steady traffic inflows, similar trends can be expected under time-dependent inflows.

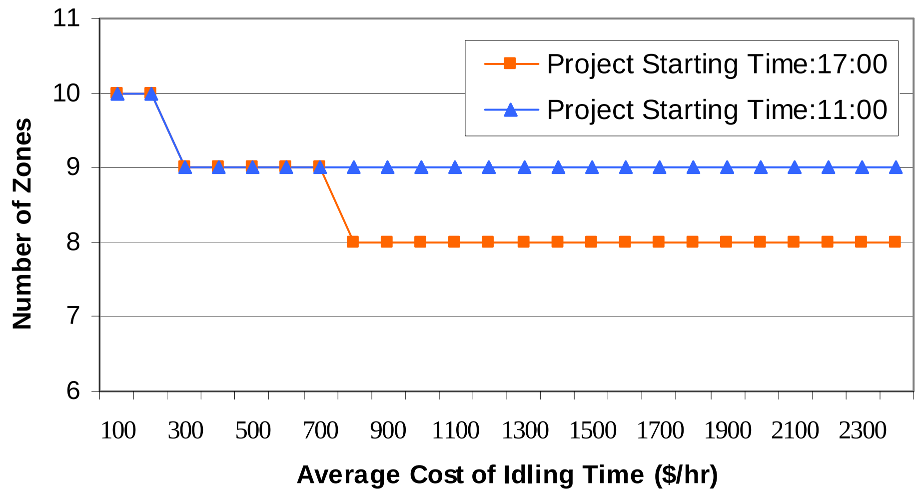


Figure 4.17 Number of Zones vs. Average Cost of Idling Time

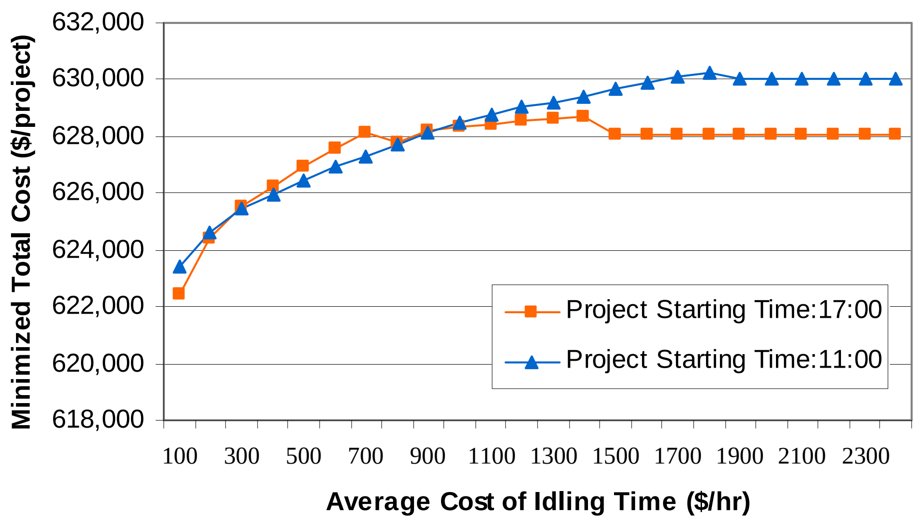


Figure 4.18 Minimized Total Cost vs. Average Cost of Idling Time

Table 4.6(a) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 11:00, z_2=$10,000/km

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0~23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.52	5.13	11.00	16.13	-	7,189
2	0.74	6.45	16.97	23.41	0.84	10,556
3	1.09	8.55	23.41	7.96	0.00	13,766
4	0.82	6.93	9.05	15.97	1.09	12,526
5	0.74	6.45	16.97	23.41	0.99	10,680
6	1.09	8.55	23.41	7.96	0.00	13,765
7	0.70	6.21	9.00	15.20	1.04	10,685
8	0.45	4.71	15.20	19.91	0.00	7,073
9	1.34	10.05	19.91	5.96	0.00	16,507
Total	7.50	63.00			3.96	102,748
Maintenance cost						84,000
Queuing delay cost						12,896
Moving delay cost						2,615
Idling cost						3,164
Accident Cost						73
Total cost						102,748
Total cost/project-km ($/lane-km)						13,700

Table 4.6(b) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 11:00, z_2=$5,000/km

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.52	5.13	11.00	16.13	-	4,584
2	0.74	6.45	16.97	23.41	0.84	6,851
3	1.09	8.55	23.41	7.96	0.00	8,310
4	0.82	6.93	9.05	15.97	1.09	8,420
5	0.74	6.45	16.97	23.41	0.99	6,975
6	1.09	8.55	23.41	7.96	0.00	8,310
7	0.70	6.21	9.00	15.20	1.04	7,180
8	0.45	4.71	15.20	19.91	0.00	4,817
9	1.34	10.05	19.91	5.96	0.00	9,802
Total	7.50	63.00			3.96	65,248
Maintenance cost						46,500
Queuing delay cost						12,896
Moving delay cost						2,615
Idling cost						3,164
Accident Cost						73
Total cost						65,248
Total cost/project-km ($/lane-km)						8,700

Table 4.6(c) Optimized Results for Numerical Example (Two-Lane Highway), Project Starting Time: 11:00, z_2=$100/km

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0~23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.52	5.13	11.00	16.13	-	2,030
2	0.74	6.45	16.97	23.41	0.84	3,219
3	1.09	8.55	23.41	7.96	0.00	2,964
4	0.82	6.93	9.05	15.97	1.09	4,397
5	0.74	6.45	16.97	23.41	0.99	3,343
6	1.09	8.55	23.41	7.96	0.00	2,963
7	0.70	6.21	9.00	15.20	1.04	3,744
8	0.45	4.71	15.20	19.91	0.00	2,607
9	1.34	10.05	19.91	5.96	0.00	3,230
Total	7.50	63.00			3.96	28,498
Maintenance cost						9,750
Queuing delay cost						12,896
Moving delay cost						2,615
Idling cost						3,164
Accident Cost						73
Total cost						28,498
Total cost/project-km ($/lane-km)						3,800

4.4 Numerical Analysis – Four-Lane Two-Way Highway

The effects of various parameters on work zone lengths and starting times for four-lane highway work zones are examined in this section. The baseline numerical values for each variable in this section are defined in Table 4.7. A numerical example sequences and schedules unequal work zones for a 7.5-km maintenance project on a two-lane highway. Table 4.8 shows the hourly traffic distribution on the maintained road. The annual average daily traffic (AADT) is 35,000 vehicles. Two daily peak periods are shown in Figure 4.19.

Table 4.7 Notation and Baseline Numerical Inputs for Four-Lane Two-Way Highway Work Zones

Variables	Description	Input Values
c_o	Maximum discharge rate without work zone	2,600vph
c_w	Maximum discharge rate along work zone	1,200vph
H	Average headway through work zone area	3 s
$AADT$	Annual average daily traffic on main road	3,5000
L_T	Project road length	7.5 km
n_a	Number of accidents per 100 million vehicle hours	40 acc/100mvh
V_w	Average work zone speed	50 km/hr
V_a	Average approaching speed	80 km/hr
v	Value of user time	12 \$/veh·hr
v_a	Average accident cost	142,000 \$/accident
v_d	Average cost of idling time	800 \$/hr
z_1	Fixed setup cost	1,000 \$/zone
z_2	Average maintenance cost per lane·kilometer	80,000 \$/lane·km
z_3	Fixed setup time	2 hr/zone
z_4	Average maintenance time per lane·kilometer	6 hr/lane·km

Compared to Powell's Method, we find in Figure 4.19 that for most project starting times considered (17 of 24) SA finds lower total costs while using less computer time (3 minutes with SA vs. 20 minutes with Powell's) for four-lane highway work zones. The best project starting time is found at 21:00 by using the SA optimization process. Its minimized total cost is $612,908/project, with five work zones whose optimized lengths of 1.52, 1.35, 1.80, 0.91, and 1.90 km add up to 7.5 km, and whose idling time is 2.03 hours, as shown in Table 4.9.

Table 4.8 AADT and Hourly Traffic Distribution on a Four-Lane Two-Way Highway

Hour	Volume (Both Direction)	% of AADT	% of Direction1	Q_1 (vph)	Q_2 (vph)
0	816	2.33%	0.48	392	424
1	815	2.33%	0.48	391	424
2	816	2.33%	0.45	367	449
3	816	2.33%	0.53	432	384
4	816	2.33%	0.53	432	384
5	816	2.33%	0.53	432	384
6	1,288	3.68%	0.57	734	554
7	2,100	6.00%	0.56	1,176	924
8	2,688	7.68%	0.56	1,505	1,183
9	2,338	6.68%	0.54	1,263	1,075
10	1,865	5.33%	0.51	951	914
11	1,515	4.33%	0.51	772	743
12	1,400	4.00%	0.50	700	700
13	1,288	3.68%	0.52	670	618
14	1,516	4.33%	0.51	773	743
15	1,988	5.68%	0.53	1,054	934
16	2,565	7.33%	0.49	1,257	1,308
17	1,970	5.63%	0.47	926	1,044
18	1,750	5.00%	0.47	822	928
19	1,638	4.68%	0.47	770	868
20	1,400	4.00%	0.46	644	756
21	1,165	3.33%	0.48	559	606
22	816	2.33%	0.48	392	424
23	815	2.33%	0.48	391	424
AADT	**35,000**	**100.00%**	**-**	**17,805**	**17,195**

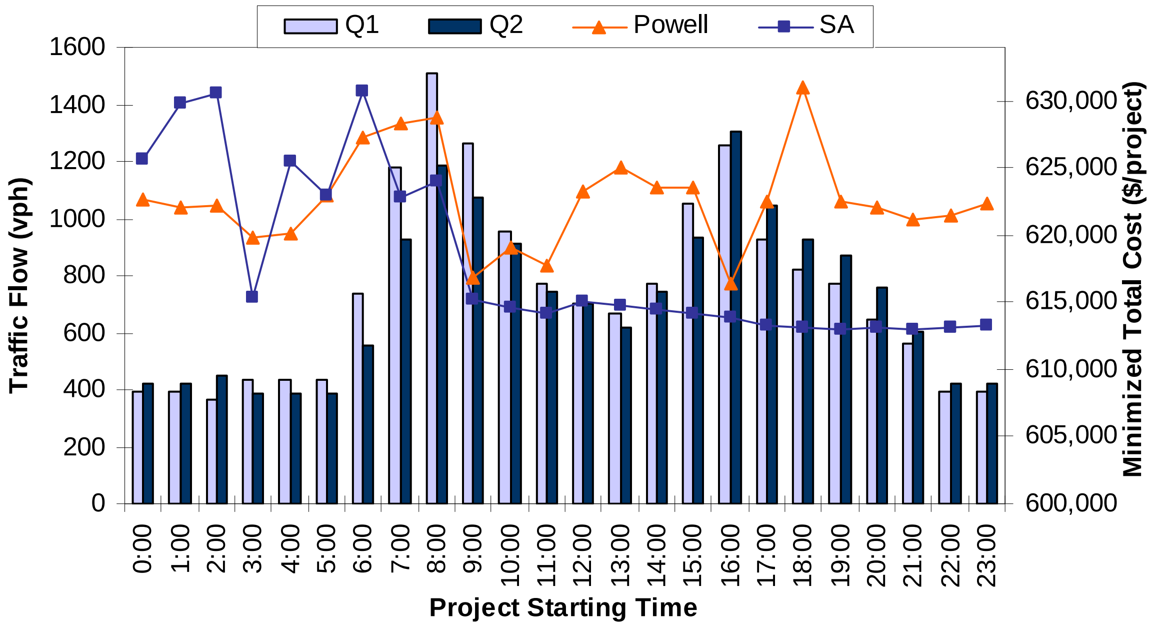


Figure 4.19 Hourly Traffic Distributions on Four-Lane Highway and Minimized Total Cost vs. Project Starting Time

Table 4.9 Optimized Results for Numerical Example (Four-Lane Highway), Project Starting Time: 21:00, v_d=$800/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	1.52	11.14	21.00	8.14	-	123,778
2	1.35	10.12	9.22	19.35	1.08	111,994
3	1.80	12.82	19.35	8.17	0.00	146,540
4	0.91	7.48	9.12	16.60	0.95	75,929
5	1.90	13.42	16.60	6.03	0.00	154,667
Total	7.50	55.00			2.03	612,908
Maintenance cost						605,000
Queuing delay cost						1,373
Moving delay cost						4,883
Idling cost						1,623
Accident Cost						30
Total cost						612,908
Total cost/project-km ($/lane-km)						81,721

Figure 4.20 shows that the total project duration decreases as the average cost of idling time v_d increases. The numbers of zones always keep five zones as v_d increases and project durations not including idling time always are 55 hours. This indicates that idling time is necessary even if v_d increases. Table 4.10(a) shows the optimized solution with idling time 1.01 hours when v_d is $2,400/hr. Table 4.10(b) shows the optimized solution with idling time 3.90 hours when v_d is $100/hr. These two solutions correspond to the second policy shown in Figure 4.13(b), which allows pauses between zones during peak traffic periods. In two-lane highway case, the idling time is avoided if v_d exceeds $1,900/hr (Figure 4.16). However, in four-lane highway work zone, even v_d reaches $2,400/hr, idling time is mandatory. This is because queuing delay will be cumulative during peak periods (Figure 4.3) and the queue will dissipate after the peak period in this four-lane highway numerical example. Even if v_d increases up to $2,400/hr, that cannot compensate for the high queuing delay costs at these four-lane highway work zones.

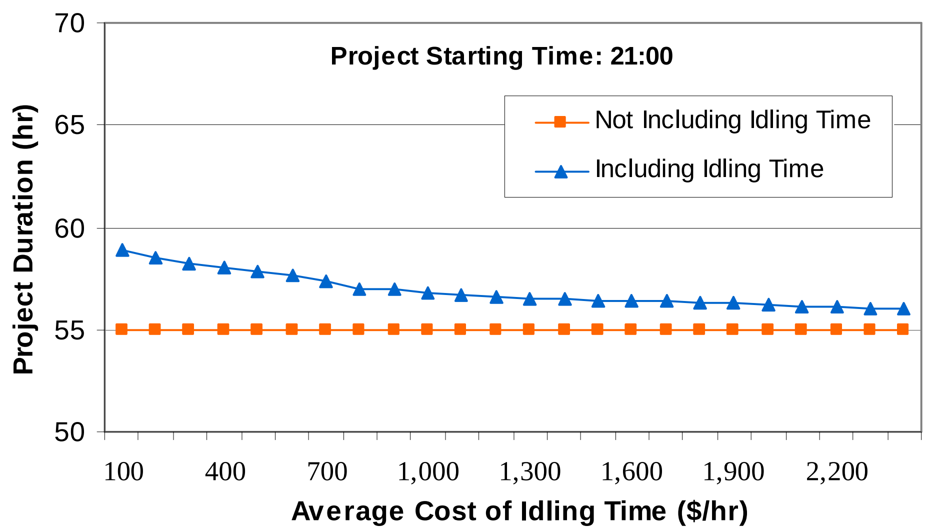


Figure 4.20 Project Duration vs. Average Cost of Idling Time

Table 4.10(a) Optimized Results for Numerical Example (Four-Lane Highway), Project Starting Time: 21:00, v_d = $2400/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	1.58	11.50	21.00	8.50	-	129,221
2	1.40	10.42	8.98	19.41	0.48	116,628
3	1.84	13.06	19.41	8.47	0.00	150,272
4	1.13	8.80	9.00	17.80	0.53	94,703
5	1.53	11.20	17.80	5.01	0.00	124,521
Total	7.50	55.00			1.01	615,345
Maintenance cost						605,000
Queuing delay cost						2,958
Moving delay cost						4,930
Idling cost						2,421
Accident Cost						37
Total cost						615,345
Total cost/project-km ($/lane-km)						82,046

Table 4.10(b) Optimized Results for Numerical Example (Four-Lane Highway), Project Starting Time: 21:00, v_d = $100/hr

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0~23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	1.50	11.02	21.00	8.02	-	122100
2	1.30	9.82	9.82	19.65	1.80	106942
3	1.73	12.40	19.65	8.05	0.00	140773
4	0.78	6.70	9.67	16.38	1.62	64392
5	2.17	15.04	16.86	7.90	0.48	176730
Total	7.50	55.00			3.90	610937
Maintenance cost						605,000
Queuing delay cost						546
Moving delay cost						4,976
Idling cost						390
Accident Cost						26
Total cost						610,937
Total cost/project-km ($/lane-km)						83,282

4.5 Reliability of Simulated Annealing

Figure 4.15 shows that the Simulated Annealing algorithm yields better solutions than Powell's Method. Powell's is a deterministic method whose solutions are reproducible. However, Simulated Annealing is a stochastic method whose solutions vary with different random numbers. To test the reliability of SA, 50 replications of the cost minimization for a project starting at 11:00 are performed with 50 different groups of random numbers. The random numbers are generated by using PMMLCG (Prime Modules Multiplicative Linear Congruential Generators) (Law, 2000). PMMLCG is probably the most widely used and best understood kind of random-number generator (Law, 2000) and this generator applied for SA can cover the most random-number range. Figure 4.21 shows that the total costs of those 50 replications range very tightly between \$627,688 and \$627,747. The minimized total costs of the 50 replications have a mean (μ) of \$627,720 and a standard deviation (σ) of \$12.62. With such a small relative variance (the coefficient of variation (c.v.) = $\sigma / \mu = 2.01 \times 10^{-5}$), we are quite unlikely to find a value much below the mean. Thus we are likely to be very near in total cost to the best possible solution (the "global optimum"). Table 4.11 shows the optimized solution of the 10th replication, which has the lowest minimized total cost \$627,688 among the 50 replications. The optimized work zone lengths in Tables 4.4(a) and 4.11 are almost the same and only the zone starting times or ending times differ very slightly. The statistical analysis and our numerical examples indicate that Simulated Annealing is very likely to find solutions that are very close in value to the global optimum.

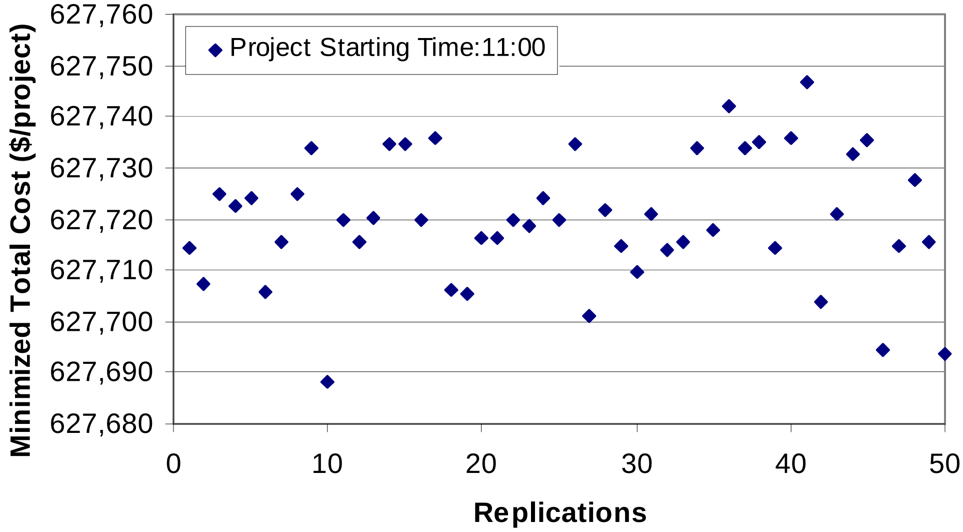


Figure 4.21. Minimized Total Costs in 50 Replications (Two-lane Highway)

Table 4.11 Optimized Results for Numerical Example, Project Starting Time: 11:00, v_d=$800/hr (10th replication), Alternative 2.1

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.53	5.16	11.00	16.16	-	44,154
2	0.77	6.60	17.00	23.60	0.84	64,502
3	1.07	8.40	23.60	8.00	0.00	88,147
4	0.82	6.90	9.09	15.99	1.09	69,612
5	0.76	6.54	16.99	23.53	1.00	63,817
6	1.08	8.46	23.53	7.99	0.00	88,968
7	0.71	6.24	9.01	15.25	1.02	60,207
8	0.45	4.68	15.25	19.93	0.00	38,271
9	1.34	10.02	19.93	5.95	0.00	110,011
Total	7.50	63.00			3.95	627,688
Maintenance cost						609,000
Queuing delay cost						12,842
Moving delay cost						2,612
Idling cost						3,162
Accident Cost						73
Total cost						627,688
Total cost/project-km ($/lane-km)						83,691

Chapter V Work Zone Optimization with a Detour

The cost models for Alternative 2.1 (two-lane two-way highway work zones without detour) and Alternative 4.1 (four-lane highway work zones without detour) are extended to analyze time-dependent inflows in Chapter 4. Work zone optimization for other time-dependent alternatives with detours are developed in this chapter.

Methods and solutions will be developed to address the following questions about work zone traffic management plans:

1. When should additional lanes be closed or reopened?

2. What fraction of the traffic should be diverted to specific alternate paths?

3. When should there be pauses in maintenance activities?

In minimizing total costs, different alternatives may be preferable for various traffic levels on the main road and on the detour(s). In Section 5.1, cost functions are developed that are applicable to each alternative for two-lane and four-lane highway work zones. In Section 5.2, optimization methods for a single alternative and mixed alternatives are developed. An improved search method, SAMASD (Simulated Annealing algorithm for mixed alternatives with a single detour), is developed that allows different alternatives to be used for successive zones within a single project. Such mixed alternatives may yield lower minimized total cost than uniform alternatives. Thus, two traffic management plans are developed with uniform alternatives and with mixed alternatives within a single project. In Section 5.3 and 5.4, numerical examples are analyzed for two-lane highway, four-lane highway. Finally, numerical examples for mixed alternatives are presented in Section 5.5.

5.1 Work Zone Cost Functions with a Detour

5.1.1 Queuing Delay on a Detour

In Chapter 3 the congestion and possible queuing delay along a detour are neglected for steady traffic inflows. However, possible queuing delays due to detour capacity and intersections along detour are considered for time-dependent inflows and are derived in this section.

Figure 5.1 shows the queuing delay and queue dissipation on the detour. For Alternative 2.1, no diverted flow affects the original flow Q_3^{ij} in Direction 3 (shown in Figure 3.1) under time-dependent traffic inflows. If Q_3^{ij} exceeds the detour capacity c_{d3}, a queue develops. The queuing delay is represented by the area D in Figure 5.1.

Then we consider what happens if pQ_1^{ij} is diverted to the detour and the flow in Direction 3 is $pQ_1^{ij} + Q_3^{ij}$ for work zone i. Figure 5.1 shows that the detour queue resulting from work zone $i\text{-}1$ is not dissipated completely and the queue length is q_{i-1} as work in zone i starts. The maximum queue length reached in work zone i is:

$$
\begin{aligned}
q_{i,max} &= q_{i-1} + (pQ_1^{i1} + Q_3^{i1} - c_{d3})D_{i1} + (pQ_1^{i2} + Q_3^{i2} - c_{d3})D_{i2} \\
&\quad + \ldots\ldots (pQ_1^{i,j-1} + Q_3^{i,j-1} - c_{d3})D_{i,j-1}
\end{aligned}
\tag{5.1}
$$

which is equal to the area A plus q_{i-1}. The area A plus q_{i-1} is equal to the area B, which represents the number of dissipated vehicles. Note that work in zone i is completed at $t_{e,i}$ and there is still a remaining dissipation time $t_{rd,i}$ for zone i. Figure 5.1 indicates that queue is dissipated completely before next zone begins so that the work zone is completed at $t_{e,i}$ and there is still a remaining dissipation time $t_{rd,i}$ for work zone i. Then the detour queuing delay for zone i is the area C. The queuing delay cost for zone i is:

$$C_{qd,i} = (\,area\ of\ C\,)v \tag{5.2}$$

Note that the queuing delay represented by area D, which results from Q_3^{ij} only in Direction 3, is not included in the queuing delay due to the diverted flow pQ_1^{ij} in Direction 3 represented by area C.

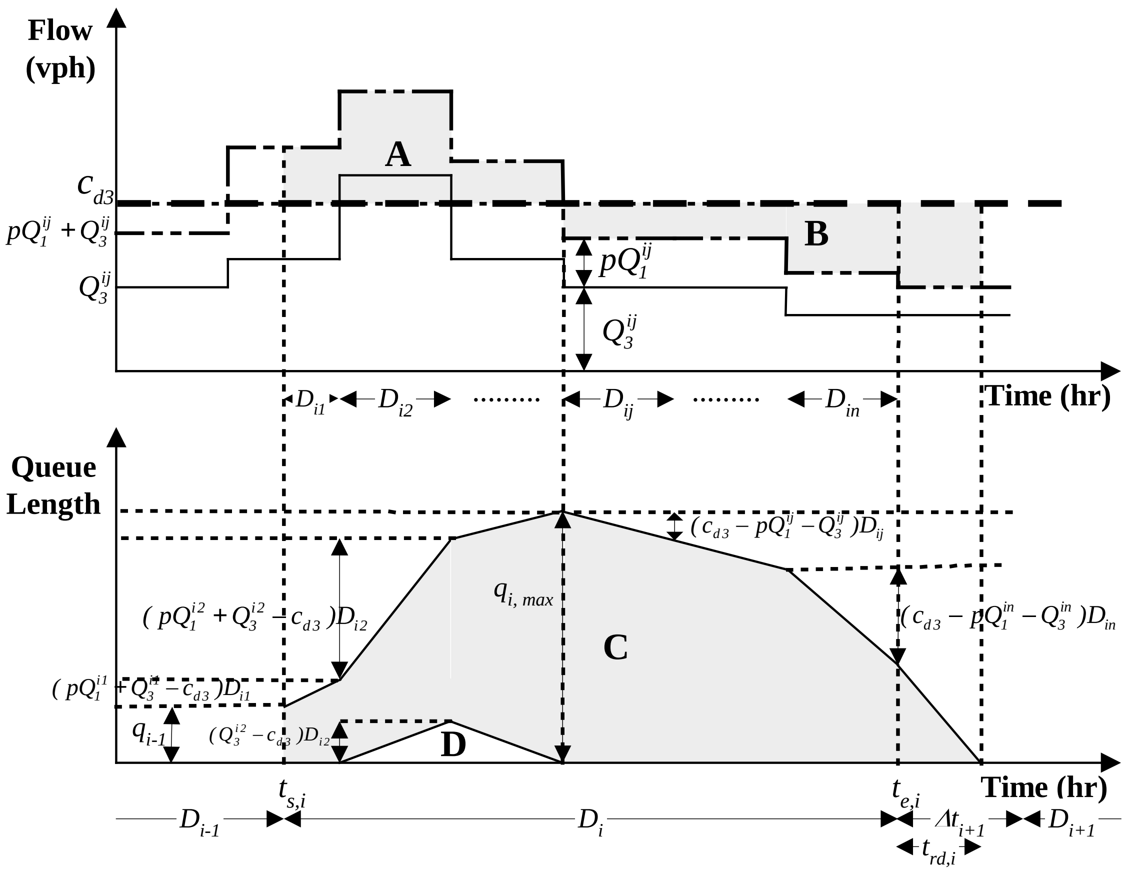


Figure 5.1 Queuing Delay and Dissipation of Queue Length along Detour

Here, the user delay cost for work zone i of the diverted flow pQ_1^{ij} along the detour due to intersection signal or stop delay, denoted as $C_{int,i}$, is equal to the flow pQ_1^{ij} multiplied by: (1) the maintenance duration per zone, D_i, (2) the number of intersections along detour, N_{int}, (3) average waiting time per intersection, t_{int}, and (4) the value of time, v. Thus:

$$C_{\text{int},i} = \sum_{j}^{n} pQ_1^{ij} D_i N_{\text{int}} \frac{t_{\text{int}}}{3600} v \tag{5.3}$$

5.1.2 Two-Lane Highway Work Zone with a Detour

The derivation processes for the cost functions of Alternatives 2.2,2.3 and 2.4 are similar to the process for Alternative 2.1. However, possible queuing delay costs along detour routes and signal or stop delays are added in this chapter while Assumptions 4 and 6 in Section 3.2.1 are released. The cost functions of Alternatives 2.2, 2.3, and 2.4 are derived as follows.

Alternative 2.2 – Flow on one lane as well as a detour

Figure 3.1(b) shows that the fraction *p* of the flow pQ_1^{ij} in Direction 1 is diverted to the alternate route. The user queuing delay cost for work zone *i* for Alternative 2.2, $C_{q(1-p)2,i}^{22}$, can be expressed as:

$$C_{q(1-p)2,i}^{22} = \sum_{j}^{n} \frac{[(1-p)Q_1^{ij}(\frac{3600}{H} - (1-p)Q_1^{ij}) + Q_2^{ij}(\frac{3600}{H} - Q_2^{ij})]v}{V(\frac{3600}{H} - (1-p)Q_1^{ij} - Q_2^{ij})} D_{ij} L_i \tag{5.4}$$

The possible queuing delay cost of the diverted flow pQ_1^{ij} and Q_3^{ij} in Direction 3 for zone *i*, denoted $C_{qd,i}^{22}$, is:

$$C_{qd,i}^{22} = (\text{area of } C)v \tag{5.5}$$

where the area of C is shown in Figure 5.1. The user delay cost of the diverted flow pQ_1^{ij} from Direction 1 along detour due to intersection signal or stop delay, denoted as $C_{int,i}^{22}$, is:

$$C^{22}_{int,i} = \sum_{j}^{n} pQ^{ij}_{1} D_i N_{int} \frac{t_{int}}{3600} v \tag{5.6}$$

The combined queuing delay cost for the maintained road AB and the detour C^{22}_{qi} is:

$$C^{22}_{qi} = C^{22}_{q(1-p)2,i} + C^{22}_{qd,i} + C^{22}_{int,i} \tag{5.7}$$

The moving delay cost of the remaining traffic flow in Direction 1, $(1-p)Q^{ij}_{1}$, and Q^{ij}_{2}, for zone *i*, denoted $C^{22}_{v(1-p)2,i}$, is formulated as:

$$C^{22}_{v(1-p)2,i} = \sum_{j}^{n} ((1-p)Q^{ij}_{1} + Q^{ij}_{2}) D_{ij} (\frac{L_i}{V} - \frac{L_i}{V_0}) v \tag{5.8}$$

The moving delay cost of the diverted flow pQ^{ij}_{1} from Direction 1, denoted as $C^{22}_{vp,i}$, is:

$$C^{22}_{vp,i} = \sum_{j}^{n} pQ^{ij}_{1} D_i [\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V^{*3}_{d}} - \frac{L_t}{V_0}] v \tag{5.9}$$

The moving delay cost $C^{22}_{v3,i}$ to the original flow on the detour, Q^{ij}_{3}, as affected by the pQ^{ij}_{1}, is:

$$C^{22}_{v3,i} = \sum_{j}^{n} Q^{ij}_{3} D_i (\frac{L_{d2}}{V^{*3}_{d}} - \frac{L_{d2}}{V_{d0}}) v \tag{5.10}$$

The combined moving delay cost for the maintained road AB and the detour C^{22}_{vi} is:

$$C^{22}_{vi} = C^{22}_{v(1-p)2,i} + C^{22}_{vp,i} + C^{22}_{v3,i} \tag{5.11}$$

The idling cost for zone *i* $C^{22}_{I\,i}$ is:

$$C^{22}_{I\,i} = v_d \Delta t_i \tag{5.12}$$

The accident cost for zone *i*, C_{ai}^{22}, is expressed as:

$$C_{ai}^{22} = \frac{(C_{qi}^{22} + C_{vi}^{22})}{v} \frac{n_a v_a}{10^8} \tag{5.13}$$

The maintenance cost for zone *i*, C_{mi}^{22}, is:

$$C_{mi}^{22} = z_1 + z_2 L_i \tag{5.14}$$

The total cost for work zone *i*, C_{ti}^{22}, is:

$$C_{ti}^{22} = (z_1 + z_2 L_i) + C_{qi}^{22} + C_{vi}^{22} + v_d \Delta t_i + \frac{(C_{qi}^{22} + C_{vi}^{22})}{v} \frac{n_a v_a}{10^8} \tag{5.15}$$

The total cost for resurfacing road length L_T by scheduling *m* work zones, C_{PT}^{22}, is expressed as:

$$\begin{aligned} C_{PT}^{22} &= \sum_i^m C_{ti}^{22} \\ &= \sum_i^m (z_1 + z_2 L_i) + \sum_i^m C_{qi}^{22} + \sum_i^m C_{vi}^{22} + \sum_i^m v_d \Delta t_i + \sum_i^m \frac{(C_{qi}^{22} + C_{vi}^{22})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{5.16}$$

Because the results in Chapter 4 were better with Simulated Annealing than with Powell's method, the total cost in Eq.(5.16) will be minimized with the Simulated Annealing algorithm proposed in Section 4.2.

Alternative 2.3 – One direction along the work zone and the other detoured

Figure 3.1(c) shows that the entire flow Q_1^{ij} in Alternative 2.1 is diverted to the alternate route. The possible queuing delay cost of the diverted flow Q_1^{ij} and Q_3^{ij} in Direction 3 for zone *i*, denoted $C_{qd,i}^{23}$, is:

$$C_{qd,i}^{23} = (\textit{area of } C)v \tag{5.17}$$

where the area of C is shown in Figure 5.1 and the *p* value is 1 (full diversion). The user delay cost of the diverted flow Q_1^{ij} from Direction 1 along detour due to intersection signal or stop delay, denoted as $C_{int,i}^{23}$, is:

$$C_{int,i}^{23} = \sum_{j}^{n} Q_1^{ij} D_i N_{int} \frac{t_{int}}{3600} v \tag{5.18}$$

The combined queuing delay cost for the maintained road AB and the detour C_{qi}^{23} can be derived as:

$$C_{qi}^{23} = C_{qd,i}^{23} + C_{int,i}^{23} \tag{5.19}$$

The user moving delay cost in Direction 1 for zone *i*, denoted as $C_{v1,i}^{23}$, has the same formulation as Eq. (5.9) but with Q_1^{ij} substituted for pQ_1^{ij}.

$$C_{v1,i}^{23} = \sum_{j}^{n} Q_1^{ij} D_i [\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0}] v \tag{5.20}$$

The moving delay cost of Q_2^{ij} along work zone for zone *i*, denoted $C_{v2,i}^{23}$, is formulated as:

$$C_{v2,i}^{23} = \sum_{j}^{n} Q_2^{ij} D_{ij} (\frac{L_i}{V} - \frac{L_i}{V_0}) v \tag{5.21}$$

The moving delay cost $C_{v3,i}^{23}$ of the original flow on the detour, Q_3^{ij}, as affected by the Q_1^{ij}, has the same formulation as Eq.(5.10).

$$C_{v3,i}^{23} = \sum_{j}^{n} Q_3^{ij} D_i (\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}}) v \tag{5.22}$$

The combined moving delay cost for the maintained road AB and the detour C_{vi}^{23} is:

$$C_{vi}^{23} = C_{v1,i}^{23} + C_{v2,i}^{23} + C_{v3,i}^{23} \tag{5.23}$$

The idling cost for zone i $C^{23}_{I\,i}$ is

$$C^{23}_{I\,i} = v_d \Delta t_i \tag{5.24}$$

The accident cost for zone i, C^{23}_{ai}, is formulated as

$$C^{23}_{ai} = \frac{C^{23}_{qi} + C^{23}_{vi}}{v} \frac{n_a v_a}{10^8} \tag{5.25}$$

The maintenance cost for zone i, C^{23}_{mi}, is $z_1 + z_2 L_i$. Then the total cost for zone i, C^{23}_{ti}, is

$$C^{23}_{ti} = (z_1 + z_2 L_i) + C^{23}_{qi} + C^{23}_{vi} + v_d \Delta t_i + \frac{(C^{23}_{qi} + C^{23}_{vi})}{v} \frac{n_a v_a}{10^8} \tag{5.26}$$

The total cost for resurfacing road length L_T, C^{23}_{PT}, is expressed as:

$$\begin{aligned} C^{23}_{PT} &= \sum_i^m C^{23}_{ti} \\ &= \sum_i^m (z_1 + z_2 L_i) + \sum_i^m C^{23}_{qi} + \sum_i^m C^{23}_{vi} + \sum_i^m v_d \Delta t_i + \sum_i^m \frac{(C^{23}_{qi} + C^{23}_{vi})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{5.27}$$

The total cost in Eq.(5.27) will be minimized with a Simulated Annealing algorithm.

Alternative 2.4 – Both directions detoured and both lanes closed for work

In Alternative 2.4, as shown in Figure 3.1(d), the entire flows Q^{ij}_1 and Q^{ij}_2 are diverted to the alternate route and both lanes between A and B are entirely closed for maintenance. The queuing delay cost of the diverted flow Q^{ij}_1 plus Q^{ij}_3 in Direction 3 and the diverted flow Q^{ij}_2 plus Q^{ij}_4 in Direction 4, denoted $C^{24}_{qd,i}$, is:

$$C^{24}_{qd,i} = (\mathit{area\ of\ } C + \mathit{area\ of\ } C')v \tag{5.28}$$

where the area of C is shown in Figure 5.1 and the *p* value is 1 (full diversion). The area of C' is the queuing delay of the diverted flow Q_2^{ij} plus Q_4^{ij} in Direction 4. The calculation of area of C' is similar to the area of C but with Q_2^{ij} substituted for Q_1^{ij}, with Q_4^{ij} substituted for Q_3^{ij}, and with c_{d4} substituted for c_{d3}, where c_{d4} is the detour capacity in Direction 4.

The user delay cost of the diverted flow Q_1^{ij} from Direction 1 and the diverted flow Q_2^{ij} from Direction 2 along detour due to intersection signal or stop delay, denoted as $C_{int,i}^{24}$, is:

$$C_{int,i}^{24} = \sum_{j}^{n} (Q_1^{ij} + Q_2^{ij}) D_i N_{int} \frac{t_{int}}{3600} v \tag{5.29}$$

The combined queuing delay cost for the maintained road AB and the detour C_{qi}^{24} can be derived as:

$$C_{qi}^{24} = C_{qd,i}^{24} + C_{int,i}^{24} \tag{5.30}$$

The user moving delay cost in Direction 1 for zone *i*, denoted as $C_{v1,i}^{24}$, has the same formulation as Eq.(5.20).

$$C_{v1,i}^{24} = \sum_{j}^{n} Q_1^{ij} D_i [\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0}] v \tag{5.31}$$

The user moving delay cost of the flow Q_2^{ij} for zone *i*, denoted as C_{v2}^{24}, has the same formulation as Eq.(5.31) but with Q_2 substituted for pQ_1^{ij} and with V_d^{*4} substituted for V_d^{*3}.

$$C_{v2,i}^{24} = \sum_{j}^{n} Q_2^{ij} D_i [\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*4}} - \frac{L_t}{V_0}] v \tag{5.32}$$

The moving delay cost $C^{24}_{v3,i}$ to the original flow on the detour in Direction 3, Q^{ij}_{3}, as affected by the Q^{ij}_{1}, has the same formulation as Eq. (5.22).

$$C^{24}_{v3,i} = \sum_{j}^{n} Q^{ij}_{3} D_i \left(\frac{L_{d2}}{V^{*3}_{d}} - \frac{L_{d2}}{V_{d0}} \right) v \tag{5.33}$$

Similarly, the moving delay cost $C^{24}_{v4,i}$ of the original flow on the detour in Direction 4, Q^{ij}_{4}, as affected by the Q^{ij}_{2}, has the same formulation as Eq. (5.33) but with Q^{ij}_{4} substituted for Q^{ij}_{3} and V^{*4}_{d} substituted for V^{*3}_{d}.

$$C^{24}_{v4,i} = \sum_{j}^{n} Q^{ij}_{4} D_i \left(\frac{L_{d2}}{V^{*4}_{d}} - \frac{L_{d2}}{V_{d0}} \right) v \tag{5.34}$$

The combined moving delay cost for the maintained road AB and the detour C^{24}_{vi} is:

$$C^{24}_{vi} = C^{24}_{v1,i} + C^{24}_{v2,i} + C^{24}_{v3,i} + C^{24}_{v4,i} \tag{5.35}$$

The idling cost for zone i $C^{24}_{I\,i}$ is:

$$C^{24}_{I\,i} = v_d \Delta t_i \tag{5.36}$$

The accident cost for zone i, C^{24}_{ai}, is formulated as:

$$C^{24}_{ai} = \frac{C^{24}_{qi} + C^{24}_{vi}}{v} \frac{n_a v_a}{10^8} \tag{5.37}$$

The maintenance cost for zone i, C^{24}_{mi}, is $z_1 + z_2 L_i$. Then the total cost for zone i, C^{24}_{ti}, is:

$$C^{24}_{ti} = (z_1 + z_2 L_i) + C^{24}_{qi} + C^{24}_{vi} + v_d \Delta t_i + \frac{(C^{24}_{qi} + C^{24}_{vi})}{v} \frac{n_a v_a}{10^8} \tag{5.38}$$

The total cost for resurfacing road length L_T, C^{24}_{PT}, is expressed as:

$$
\begin{aligned}
C_{PT}^{24} &= \sum_{i}^{m} C_{ti}^{24} \\
&= \sum_{i}^{m} (z_1 + z_2 L_i) + \sum_{i}^{m} C_{qi}^{24} + \sum_{i}^{m} C_{vi}^{24} + \sum_{i}^{m} v_d \Delta t_i + \sum_{i}^{m} \frac{(C_{qi}^{24} + C_{vi}^{24})}{v} \frac{n_a v_a}{10^8}
\end{aligned} \tag{5.39}
$$

5.1.3 Four-Lane Highway Work Zone with a Detour

The derivation processes for the cost functions of Alternatives 4.2,4.3 and 4.4 are similar to the process for Alternative 4.1. Possible queuing delay costs along detour routes and signal or stop delays are added and Assumptions 3 and 5 in Section 3.3.1 are released. Queuing delay on the detour is developed as shown in Section 5.1.1 and cost functions for Alternatives 4.2, 4.3, and 4.4 are derived as follows.

Alternative 4.2 – A fraction of Q_1 traffic through detour

Figure 3.2(b) shows that the fraction *p* of the flow pQ_1^{ij} in Direction 1 is diverted to the alternate route. The user queuing delay cost of the remaining flow $(1-p)Q_1^{ij}$ in Direction 1 for work zone *i* for Alternative 4.2, $C_{q(1-p),\,i}^{42}$, is the area of C in Figure 4.3 multiplied by *v* but with $(1-p)Q_1^{ij}$ substituted for Q_1^{ij}.

$$C_{q(1-p),i}^{42} = (\text{area of } C)v \tag{5.40}$$

The possible queuing delay cost of the diverted flow pQ_1^{ij} and Q_3^{ij} in Direction 3 for zone *i*, denoted $C_{qd,i}^{42}$, is the area C in Figure 5.1 multiplied by *v*.

$$C_{qd,i}^{42} = (\text{area of } C)v \tag{5.41}$$

The user delay cost of the diverted flow pQ_1^{ij} from Direction 1 along detour due to intersection signal or stop delay, denoted as $C_{int,\,i}^{42}$, is:

$$C^{42}_{int,i} = \sum_{j}^{n} pQ^{ij}_{1} D_i N_{int} \frac{t_{int}}{3600} v \tag{5.42}$$

The combined queuing delay cost for the maintained road AB and the detour C^{42}_{qi} can be derived as:

$$C^{42}_{qi} = C^{42}_{q(1-p),i} + C^{42}_{qd,i} + C^{42}_{int,i} \tag{5.43}$$

The moving delay cost of the traffic flows $(1-p)Q^{ij}_{1}$ in work zone *i*, denoted $C^{42}_{v(1-p),i}$, is the cost increment due to the zone. It is the moving delay $t^{ij}_{m(1-p)}$ multiplied by the average delay cost *v*:

$$C^{42}_{v(1-p),i} = \sum_{j=1}^{n} t^{ij}_{m(1-p)} v \tag{5.44}$$

where $t^{ij}_{m(1-p)}$ = moving delay incurred by the approaching traffic flow $(1-p)Q^{ij}_{1}$ for zone *i* in each period D_{ij} of work zone duration D_i. The $t^{ij}_{m(1-p)}$ is a function of the difference between the travel time on a road with and without a work zone:

$$t^{ij}_{m(1-p)} = \left(\frac{L_i}{V_w} - \frac{L_i}{V_a}\right)(1-p)Q^{ij}_{1} D_{ij} \qquad when\ (1-p)Q^{ij}_{1} \le c_w \tag{5.45a}$$

$$t^{ij}_{m(1-p)} = \left(\frac{L_i}{V_w} - \frac{L_i}{V_a}\right) c_w D_{ij} \qquad when\ (1-p)Q^{ij}_{1} > c_w \tag{5.45b}$$

The moving delay cost of the diverted flow pQ^{ij}_{1} from Direction 1, denoted as $C^{42}_{vp,i}$, is:

$$C^{42}_{vp,i} = \sum_{j}^{n} pQ^{ij}_{1} D_i \left[\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V^{*3}_{d}} - \frac{L_t}{V_0}\right] v \tag{5.46}$$

The moving delay cost $C^{42}_{v3,i}$ to the original flow on the detour, Q^{ij}_{3}, as affected by the pQ^{ij}_{1} is:

$$C^{42}_{v3,i} = \sum_{j}^{n} Q^{ij}_{3} D_i \left(\frac{L_{d2}}{V^{*3}_{d}} - \frac{L_{d2}}{V_{d0}} \right) v \tag{5.47}$$

The combined moving delay cost for the maintained road AB and the detour C^{42}_{vi} can be derived as:

$$C^{42}_{vi} = C^{42}_{v(1-p),\,i} + C^{42}_{vp,i} + C^{42}_{v3,i} \tag{5.48}$$

The idling cost for zone i $C^{42}_{I\,i}$ is:

$$C^{42}_{I\,i} = v_d \Delta t_i \tag{5.49}$$

The accident cost for zone i, C^{42}_{ai}, is formulated as:

$$C^{42}_{ai} = \frac{(C^{42}_{qi} + C^{42}_{vi})}{v} \frac{n_a v_a}{10^8} \tag{5.50}$$

The maintenance cost for zone i, C^{42}_{mi}, is $z_1 + z_2 L_i$. Then the total cost for zone i, C^{42}_{ti}, is:

$$C^{42}_{ti} = (z_1 + z_2 L_i) + C^{42}_{qi} + C^{42}_{vi} + v_d \Delta t_i + \frac{(C^{42}_{qi} + C^{42}_{vi})}{v} \frac{n_a v_a}{10^8} \tag{5.51}$$

The total cost for resurfacing road length L_T by scheduling m work zones, C^{42}_{PT}, is expressed as:

$$\begin{aligned} C^{42}_{PT} &= \sum_{i}^{m} C^{42}_{ti} \\ &= \sum_{i}^{m} (z_1 + z_2 L_i) + \sum_{i}^{m} C^{42}_{qi} + \sum_{i}^{m} C^{42}_{vi} + \sum_{i}^{m} v_d \Delta t_i + \sum_{i}^{m} \frac{(C^{42}_{qi} + C^{42}_{vi})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{5.52}$$

Alternative 4.3 – All Q_1 traffic through detour, allowing work zone on both lanes in Direction 1

Figure 3.2(c) shows the entire flow Q_1^{ij} in Direction 1 being diverted to the alternate route. There is no queuing delay in Direction 1. The possible queuing delay cost of the diverted flow Q_1^{ij} and Q_3^{ij} in Direction 3 for zone *i*, denoted $C_{qd,i}^{43}$, is the area of C in Figure 5.1 multiplied by *v*.

$$C_{qd,i}^{43} = (\, area\ of\ C\,)v \tag{5.53}$$

The user delay cost of the diverted flow Q_1^{ij} from Direction 1 along detour due to intersection signal or stop delay, denoted as $C_{int,\,i}^{43}$, is:

$$C_{int,i}^{43} = \sum_{j}^{n} Q_1^{ij} D_i N_{int} \frac{t_{int}}{3600} v \tag{5.54}$$

The combined queuing delay cost for the maintained road AB and the detour C_{qi}^{43} can be derived as:

$$C_{qi}^{43} = C_{qd,i}^{43} + C_{int,i}^{43} \tag{5.55}$$

The moving delay cost of the diverted flow Q_1^{ij} from Direction 1, denoted as $C_{v1,i}^{43}$, is:

$$C_{v1,i}^{43} = \sum_{j}^{n} Q_1^{ij} D_i [\frac{L_{d1} + L_{d3}}{V_0} + \frac{L_{d2}}{V_d^{*3}} - \frac{L_t}{V_0}]v \tag{5.56}$$

The moving delay cost $C_{v3,i}^{43}$ to the original flow on the detour, Q_3^{ij}, as affected by the Q_1^{ij} is:

$$C_{v3,i}^{43} = \sum_{j}^{n} Q_3^{ij} D_i (\frac{L_{d2}}{V_d^{*3}} - \frac{L_{d2}}{V_{d0}})v \tag{5.57}$$

The combined moving delay cost for the maintained road AB and the detour C_{vi}^{43} can be derived as:

$$C_{vi}^{43} = C_{v1,i}^{43} + C_{v3,i}^{43} \tag{5.58}$$

The idling cost for zone i $C_{idle,i}^{43}$ is:

$$C_{idle,i}^{43} = v_d \Delta t_i \tag{5.59}$$

The accident cost for zone i, C_{ai}^{43}, is formulated as:

$$C_{ai}^{43} = \frac{(C_{qi}^{43} + C_{vi}^{43})}{v} \frac{n_a v_a}{10^8} \tag{5.60}$$

The maintenance cost for zone i, C_{mi}^{43}, is $z_1 + z_2 L_i$. Then the total cost for zone i, C_{ti}^{43}, is:

$$C_{ti}^{43} = (z_1 + z_2 L_i) + C_{qi}^{43} + C_{vi}^{43} + v_d \Delta t_i + \frac{(C_{qi}^{43} + C_{vi}^{43})}{v} \frac{n_a v_a}{10^8} \tag{5.61}$$

The total cost for resurfacing road length L_T by scheduling m work zones, C_{PT}^{43}, is expressed as:

$$\begin{aligned} C_{PT}^{43} &= \sum_i^m C_{ti}^{43} \\ &= \sum_i^m (z_1 + z_2 L_i) + \sum_i^m C_{qi}^{43} + \sum_i^m C_{vi}^{43} + \sum_i^m v_d \Delta t_i + \sum_i^m \frac{(C_{qi}^{43} + C_{vi}^{43})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{5.62}$$

Alternative 4.4 – Crossover of all Q_1 traffic into one opposite lane, allowing work on both lanes in Direction 1

Figure 3.2(d) shows that the entire flow Q_1^{ij} in Direction 1 crosses over to one lane in the opposite direction. Both lanes in Direction 1 are closed for a work zone. The flow Q_2^{ij} in Direction 2 only uses the remaining lane. The user queuing delay cost of the flow Q_1^{ij} in Direction 1 for work zone *i*, $C_{q1,\,i}^{44}$, is:

$$C_{q1,i}^{44} = (\,area\ of\ C\,)v \tag{5.63}$$

where the area C is the queuing delay of the flow Q_1^{ij}, as shown in Figure 4.3.

The user queuing delay cost of the flow Q_2^{ij} in Direction 2 for work zone *i*, $C_{q2,\,i}^{44}$, is:

$$C_{q2,i}^{44} = (\,area\ of\ C'\,)v \tag{5.64}$$

where the area of C' is the queuing delay of the flow Q_2^{ij}. Area C' is determined as area C but with Q_2^{ij} substituted for Q_1^{ij}.

The combined queuing delay cost for the maintained road AB and the detour C_{qi}^{44} can be derived as:

$$C_{qi}^{44} = C_{q1,i}^{44} + C_{q2,i}^{44} \tag{5.65}$$

The moving delay cost of the traffic flows Q_1^{ij} in work zone *i*, denoted $C_{v1,i}^{44}$, is:

$$C_{v1,i}^{44} = (\frac{L_i}{V_w} - \frac{L_i}{V_a})Q_1^{ij} D_{ij} v \qquad when\ Q_1^{ij} \le c_w \tag{5.66a}$$

$$C_{v1,i}^{44} = (\frac{L_i}{V_w} - \frac{L_i}{V_a})c_w D_{ij} v \qquad when\ Q_1^{ij} > c_w \tag{5.66b}$$

The moving delay cost of the traffic flows Q_2^{ij} in work zone *i*, denoted $C_{v2,i}^{44}$, is:

$$C_{v2,i}^{44} = (\frac{L_i}{V_w} - \frac{L_i}{V_a})Q_2^{ij} D_{ij} v \qquad when\ Q_2^{ij} \le c_w \tag{5.67a}$$

$$C_{v2,i}^{44} = (\frac{L_i}{V_w} - \frac{L_i}{V_a})c_w D_{ij} v \qquad when\ Q_2^{ij} > c_w \tag{5.67b}$$

The combined moving delay cost for the maintained road AB and the detour C_{vi}^{44} can be derived as:

$$C_{vi}^{44} = C_{v1,i}^{44} + C_{v2,i}^{44} \tag{5.68}$$

The idling cost for zone i $C_{idle,i}^{44}$ is:

$$C_{idle,i}^{44} = v_d \Delta t_i \tag{5.69}$$

The accident cost for zone i, C_{ai}^{44}, is formulated as:

$$C_{ai}^{44} = \frac{(C_{qi}^{44} + C_{vi}^{44})}{v} \frac{n_a v_a}{10^8} \tag{5.70}$$

The maintenance cost for zone i, C_{mi}^{44}, is $z_1 + z_2 L_i$. Then the total cost for zone i, C_{ti}^{44}, is:

$$C_{ti}^{44} = (z_1 + z_2 L_i) + C_{qi}^{44} + C_{vi}^{44} + v_d \Delta t_i + \frac{(C_{qi}^{44} + C_{vi}^{44})}{v} \frac{n_a v_a}{10^8} \tag{5.71}$$

The total cost for resurfacing road length L_T by scheduling m work zones, C_{PT}^{44}, is expressed as:

$$\begin{aligned} C_{PT}^{44} &= \sum_i^m C_{ti}^{44} \\ &= \sum_i^m (z_1 + z_2 L_i) + \sum_i^m C_{qi}^{44} + \sum_i^m C_{vi}^{44} + \sum_i^m v_d \Delta t_i + \sum_i^m \frac{(C_{qi}^{44} + C_{vi}^{44})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{5.72}$$

5.2 Optimization Methods

5.2.1 Uniform Alternatives and Mixed Alternatives

Until now, the same alternative was assumed to be applied in all zones of one project, which is called uniform alternatives here. Numerical examples for uniform alternatives will be analyzed in Sections 5.3 and 5.4 for two-lane highway and four-lane highway work zones based on the Simulated Annealing algorithm developed in Chapter 4, which is called “SAUA” (Simulated Annealing algorithm for Uniform Alternatives). If the alternatives consider a single detour, i.e. Alternatives 2.2, 2.3, 2,4, 4.2, and 4.3, the SA algorithm is called “SAUASD” (SAUA with a Single Detour). In this section, we consider the possible advantages of using different alternatives for different zones within a project.

Sections 5.1 and its numerical examples indicate that the optimization for uniform alternatives is developed and alternative selection is determined by which alternative (and what diverted fraction if Alternative 2.2 or 4.2 is preferable) yields the lowest total cost. Each project is optimized by a given single alternative, with or without a detour. However, lower minimized total cost for a project may be obtained by mixing several alternatives within a project. A traffic management plan combining different alternatives is shown in Figure 5.2. For example, Alternatives 2.3 and 2.2 might have minimized total cost during the off-peak period and Alternative 2.1 might have minimized total cost during the peak period.

An improved Simulated Annealing algorithm is developed here to search through possible mixed alternatives and diverted fractions for all zones within a project in order to minimize total cost. The improved method is called “SAMASD” (Simulated Annealing algorithm for Mixed Alternatives with a Single Detour). Thus, two traffic management plans are developed with uniform alternatives and with mixed alternatives within a single project.

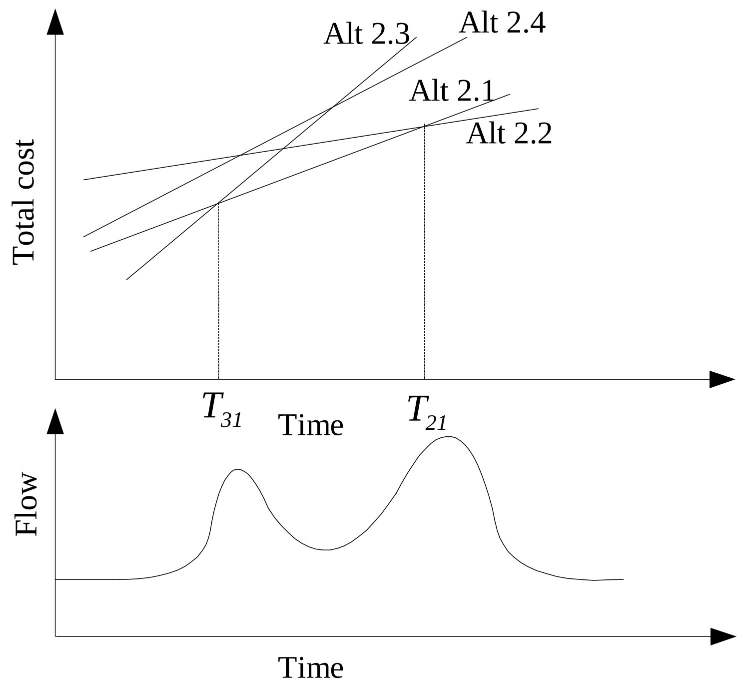


Figure 5.2 Traffic Management Plan Combining Different Alternatives

5.2.2 Simulated Annealing Algorithm for Mixed Alternatives with a Single Detour-SAMASD

Figure 5.3 shows the improved Simulated Annealing algorithm for integrating mixed alternatives within a project. This SAMASD algorithm is developed by modifying the Simulated Annealing algorithm developed in Section 4.2.2. The SAMASD algorithm is shown as follows:

1. Add new variables A_i, p_i, $A_{opt,i}$, $p_{opt,i}$ in the Step 0 in Section 4.2.2, where

A_i: Alternative for zone i, A_i=2.1, 22, 23,and 24, i=*1*, …., m;

p_i: diverted fraction for zone i, $p_i = 0 - 1$, $i=1, \ldots, m$;

$A_{opt,i}$: final optimal Alternative for zone i, $A_{opt,i}$=21, 22, 23,and 24, $i=1, \ldots, m$;

$p_{opt,i}$: final optimal diverted fraction for zone i, $p_{opt,i} = 0 - 1$, $i=1, \ldots, m$.

The notation for two-lane highway alternatives is applied here. “21” represents Alternative 2.1. For four-lane highway work zones, 21, 22, 23, and 24 can be substituted for 41, 42, 43, and 44, respectively.

Set the initial $A_{opt,i}$, =21, $p_{opt,i}$ =0, $i=1, \ldots, m$ for all zones.

2. Add “Determine alternatives and diverted fraction for n_1 and n_2” after generating random neighboring solution in Step 1. Test all possible A_i and p_i combinations and calculate the total cost. If the total cost for the current combination is lower than for the previous combination, update $A_{opt,i}$ and $p_{opt,i}$; otherwise, keep the previous solution. This procedure terminates when all possible A_i and p_i combinations are tested. Figure 5.4 shows the flow chart for determining alternatives and diverted fractions in SAMASD.

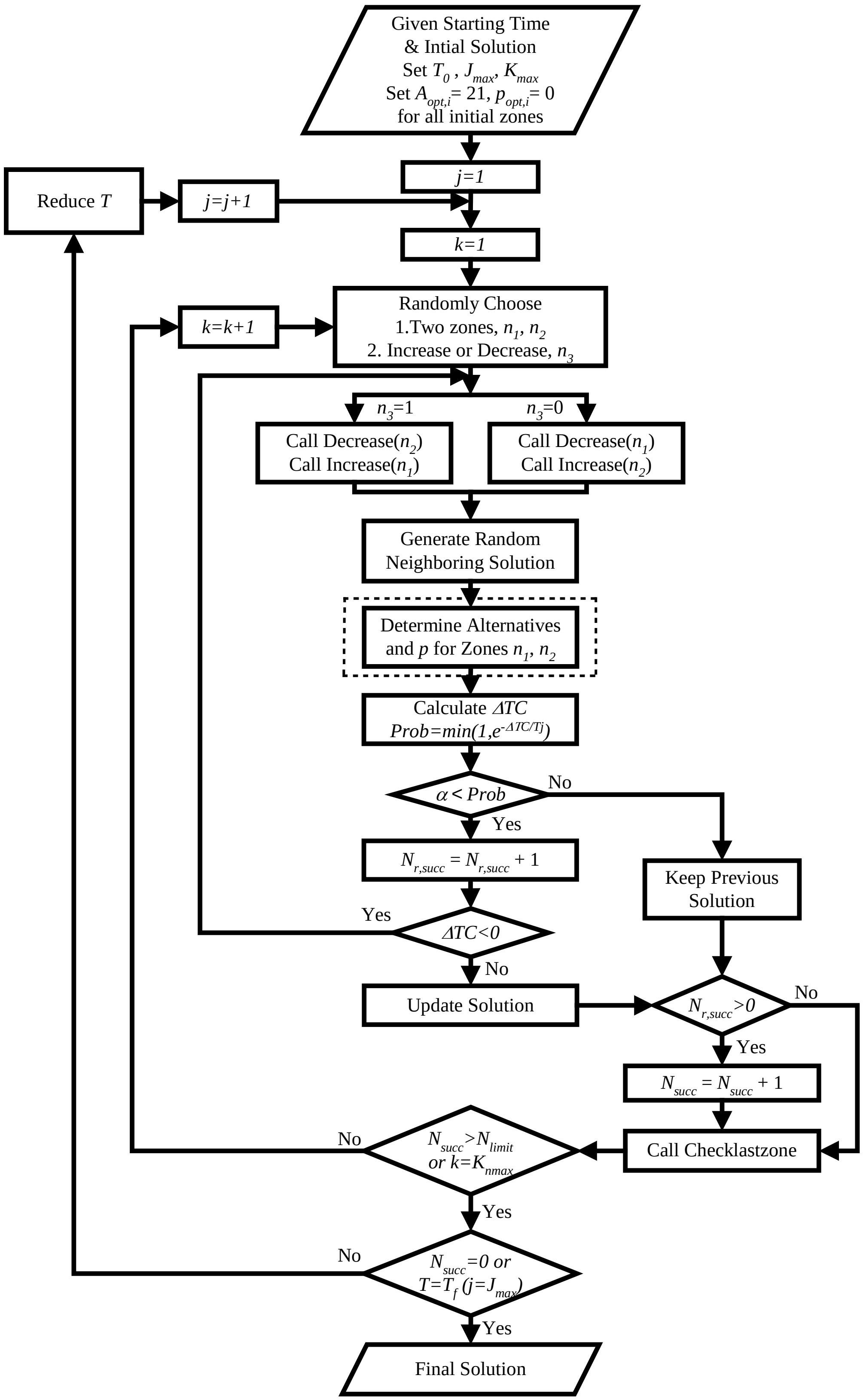


Figure 5.3 SAMASD Algorithm

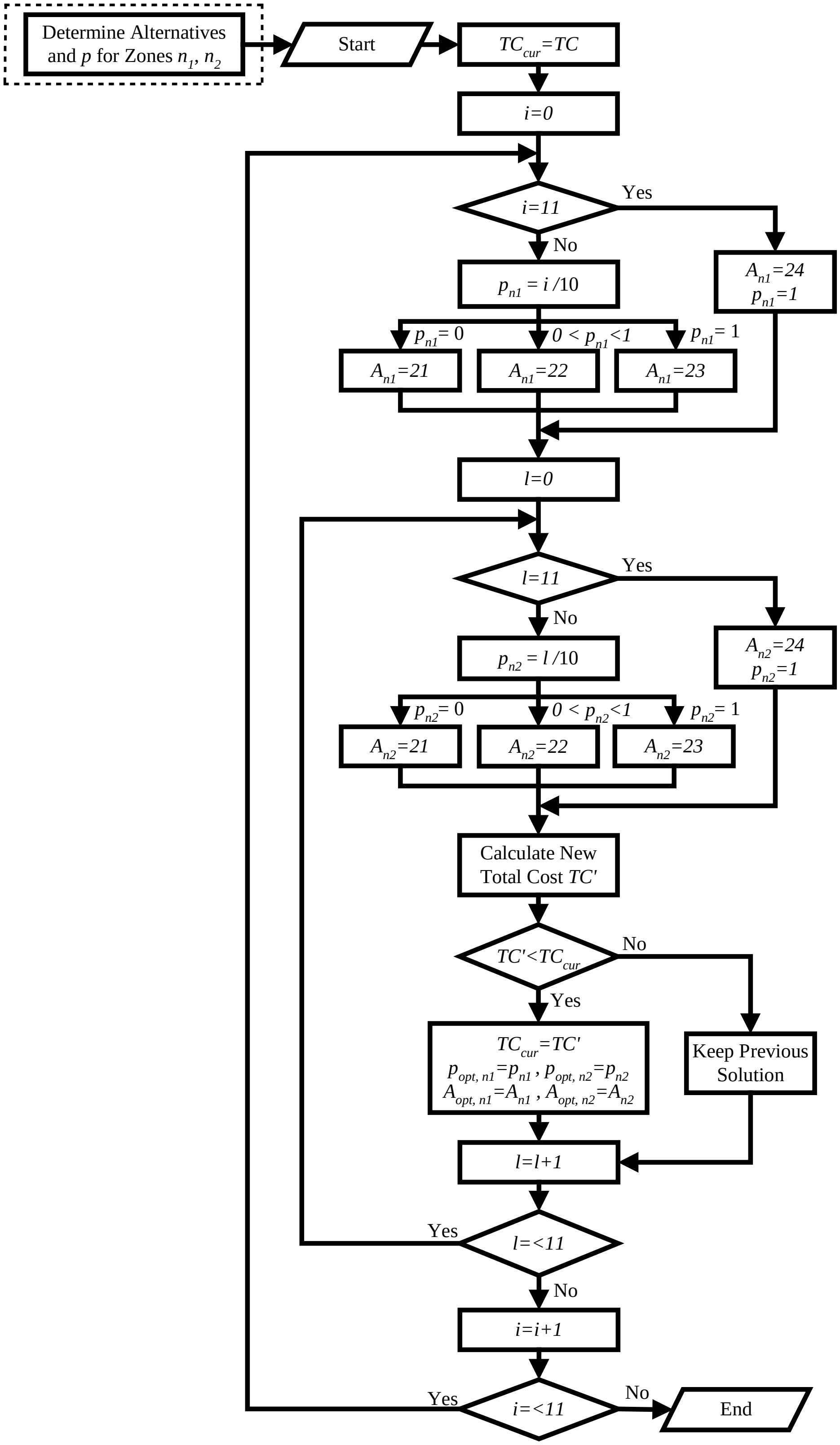


Figure 5.4 Determining Alternatives and Diverted Fractions in SAMASD

5.3 Numerical Examples - Two-Lane Highway Work Zone with a Detour

The effects of various parameters on work zone scheduling for two-lane highway and on the preferable alternatives are examined in this section. The baseline numerical values for each variable in this section are defined in Table 5.1.A numerical example sequences and schedules unequal work zones for a 7.5-km maintenance project on a two-lane highway with a detour. Table 4.3 shows the hourly traffic distribution on the maintained road. The annual average daily traffic (AADT) on the main road is 15,000 vehicles, as shown in Figure 4.15 for Alternative 2.1. The annual average daily traffic on the detour is 5,000 vehicles, as shown in Table 5.2.

Table 5.1 Inputs for Numerical Example for Two-Lane Highway Work Zones with Detour

Variable	Description	Values
c_{d3}	Maximum discharge rate along detour L_{d2}	1,300vph
$AADT_m$	Annual average daily traffic on main Road	1,5000
$AADT_d$	Annual average daily traffic on detour	5,000
L_T	Project road length	7.5 km
L_{d1}	Length of first detour segment	0.5 km
L_{d2}	Length of second detour segment	7.5 km
L_{d3}	Length of third detour segment	0.5 km
L_t	Entire Distance of Maintained Road from A to B	7.5 km
N_{int}	Number of intersections along detour	3
n_a	Number of accidents per 100 million vehicle hour	40 acc/100mvh
t_{int}	Average waiting time per intersection	30 sec
V	Average work zone speed	50 km/hr
V_f	Free flow speed along AB and detour	80 km/hr
v	Value of user time	12 \$/veh·hr
v_a	Average accident cost	142,000 \$/accident
v_d	Average Cost of Idling Time	800 \$/hr
z_1	Fixed setup cost	1,000 \$/zone
z_2	Average maintenance cost per lane·kilometer	80,000 \$/lane·km
z_3	Fixed setup time	2 hr/zone
z_4	Average maintenance time per lane·kilometer	6 hr/lane·km

Table 5.2 AADT and Hourly Traffic Distribution on Detour (Two-lane Highway)

Hour	Volume (Both Direction)	% of AADT	% of Direction3	% of Direction4	Q_3 (vph)	Q_4 (vph)
0	117	2.33%	0.48	0.52	56	61
1	116	2.33%	0.48	0.52	55	61
2	117	2.33%	0.45	0.55	53	64
3	116	2.33%	0.53	0.47	61	55
4	117	2.33%	0.53	0.47	62	55
5	116	2.33%	0.53	0.47	61	55
6	184	3.68%	0.57	0.43	105	79
7	300	6.00%	0.56	0.44	168	132
8	383	7.68%	0.56	0.44	214	169
9	334	6.68%	0.54	0.46	180	154
10	267	5.33%	0.51	0.49	136	131
11	217	4.33%	0.51	0.49	110	107
12	200	4.00%	0.5	0.5	100	100
13	184	3.68%	0.52	0.48	96	88
14	217	4.33%	0.51	0.49	110	107
15	283	5.68%	0.53	0.47	150	133
16	367	7.33%	0.49	0.51	180	187
17	282	5.63%	0.47	0.53	132	150
18	250	5.00%	0.47	0.53	117	133
19	233	4.68%	0.47	0.53	110	123
20	200	4.00%	0.46	0.54	92	108
21	167	3.33%	0.48	0.52	80	87
22	116	2.33%	0.48	0.52	55	61
23	117	2.33%	0.48	0.52	56	61
AADT	**5,000**	**100.00%**	**-**	**-**	**2,539**	**2,461**

Figure 5.5 shows the Minimized Total Cost and project starting time for Alternatives 2.1, 2.2 (p=0.3, 0.6, 0.9), 2.3 (p=1) and 2.4. The best project starting times are 11:00, 11:00, and 9:00, respectively, for Alternatives 2.1, 2.3, and 2.4; and 20:00 for Alternative 2.2 (p=0.3, 0.6,0.9). Based on the baseline values in Table 5.1, the cost of Alternative 2.3 is minimized by starting the work at 11:00, as shown in Table 5.3. Its minimized total cost is $614,073/project, with three work zones whose optimized lengths of 2.65, 2.22, and 2.62 km add up to 7.5 km, and whose idling time is 0.

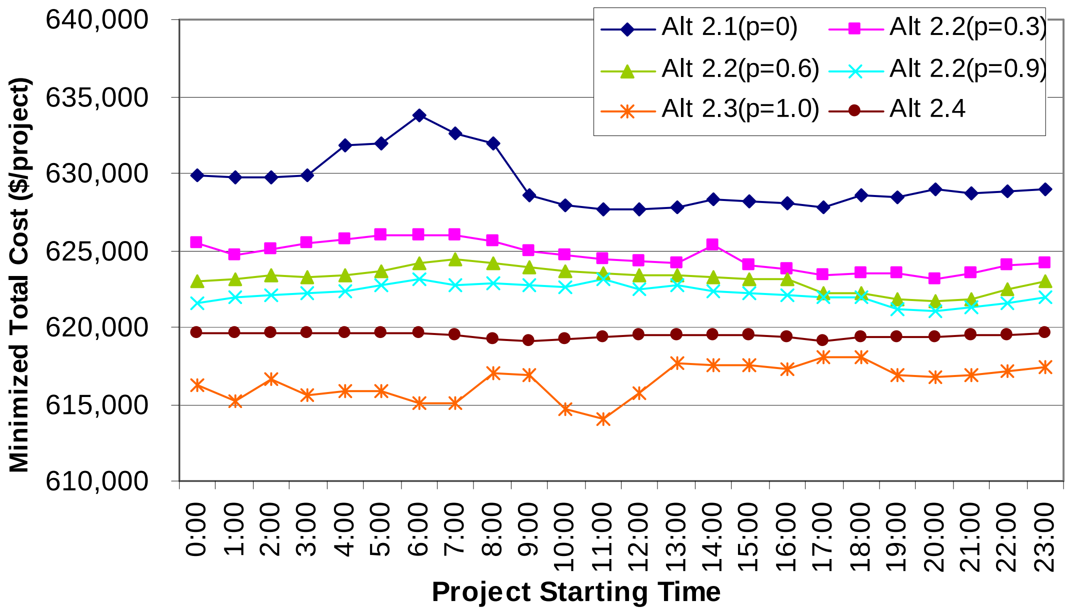


Figure 5.5 Minimized Total Cost vs. Project Starting Time (Two-lane Highway Work Zones)

Table 5.3 Optimized Results for Numerical Example, Project Starting Time: 11:00, v_d=$800/hr, Alternative 2.3

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	2.65	17.92	11.00	4.92	-	216,854
2	2.22	15.34	4.92	20.26	0.00	182,824
3	2.62	17.74	20.26	14.00	0.00	214,395
Total	7.50	51.00			0.00	614,073
Maintenance cost						603,000
Queuing delay cost						0
Moving delay cost						11,021
Idling cost						0
Accident Cost						52
Total cost						614,073
Total cost/project-km ($/lane-km)						81,876

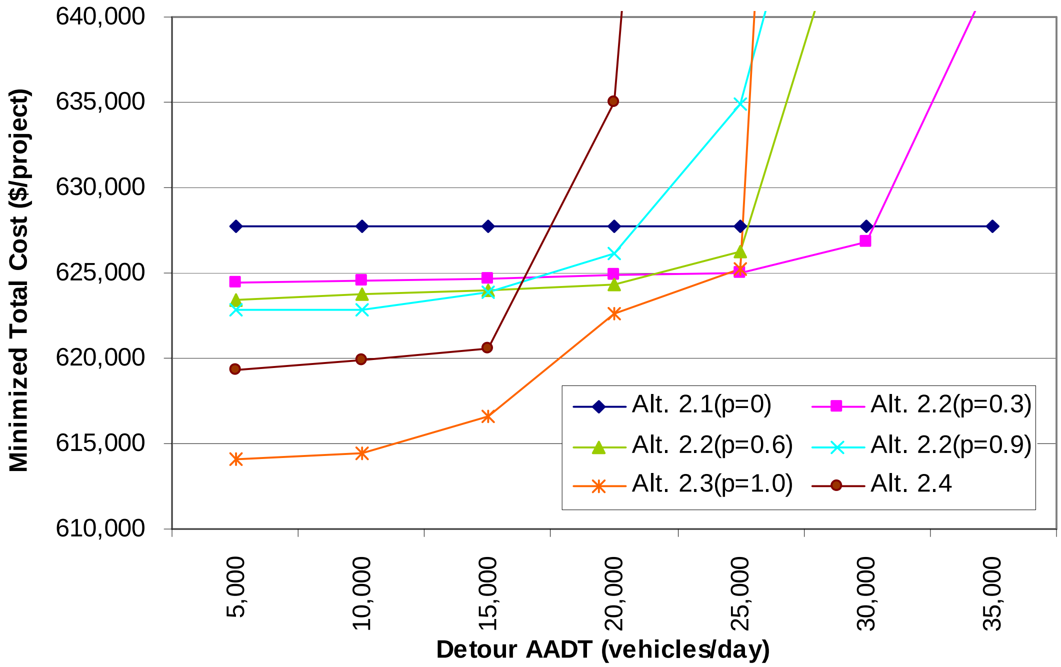


Figure 5.6 Minimized Total Cost vs. Detour AADT

Figure 5.6 shows that Alternatives 2.1, 2.2, and 2.3 are on the lowest cost envelope. The first of two thresholds with respect to Annual Average Daily Traffic (AADT) occurs at 25,000 vehicles per day, beyond which Alternative 2.2 (p=0.3) becomes preferable to Alternative 2.3; beyond 30,000 vehicles per day Alternative 2.1 (p=0) becomes preferable to Alternative 2.2 (p=0.3). A sharp increase in cost occurs for all alternatives except 2.1 since the detour queuing delays increase drastically when the diverted flow plus original detour flow exceed the detour capacity. Because there is no detour in Alternative 2.1, its minimized total cost is not sensitive to detour AADT. This threshold analysis is similar to Figure 3.10, which indicates that Alternatives 2.4, 2.3, 2.2, and 2.1 become preferable as detour length increases. This occurs because higher detour traffic (Figure 5.6) or longer detours (Figure 3.10) increase the time that diverted motorists need to return to the original main road. If the motorists must spend much more time on the detour, little or no diversion is desirable.

5.4 Numerical Examples - Four-Lane Highway Work Zone with a Detour

The effects of various parameters on work zone scheduling for four-lane highway and on the preferable alternatives are examined in this section. The baseline numerical values for each variable are the same as in Table 5.1 except AADT on main road and detour. A numerical example sequences unequal work zones for a 7.5-km maintenance project on a four-lane highway with a detour. Table 4.8 shows the hourly traffic distribution on the maintained road. The annual average daily traffic (AADT) on the main road is 35,000 vehicles, as shown in Figure 4.19 for Alternative 4.1. The annual average daily traffic on the detour is 10,000 vehicles per day and the hourly traffic distribution is shown in Table 5.4.

Figure 5.7 shows the minimized total cost and project starting time for Alternatives 4.1, 4.2 (p=0.3, 0.6, 0.9), 4.3 (p=1) and 4.4. The best project starting times are 22:00, 12:00, 16:00, and 10:00, respectively, for Alternatives 4.2 (p=0.3), 4.2 (p=0.6), 4.2 (p=0.9) and 4.3; and 21:00 for Alternatives 4.1 and 4.4. Based on the baseline values, the cost of Alternative 4.1 is minimized by starting the work at 21:00. Its minimized total cost is $612,908/project, with five work zones whose optimized lengths of 1.52, 1.35, 1.80, 0.91, and 1.90 km add up to 7.5 km, and whose idling time is 2.03 hours, as shown in Table 4.8.

Table 5.4 AADT and Hourly Traffic Distribution on Detour (Four-lane Highway)

Hour	Volume (Both Direction)	% of AADT	% of Direction1	% of Direction2	Q_3 (vph)	Q_4 (vph)
0	233	2.33%	0.48	0.52	112	121
1	233	2.33%	0.48	0.52	112	121
2	233	2.33%	0.45	0.55	105	128
3	233	2.33%	0.53	0.47	123	110
4	233	2.33%	0.53	0.47	123	110
5	233	2.33%	0.53	0.47	123	110
6	368	3.68%	0.57	0.43	210	158
7	600	6.00%	0.56	0.44	336	264
8	768	7.68%	0.56	0.44	430	338
9	668	6.68%	0.54	0.46	361	307
10	533	5.33%	0.51	0.49	272	261
11	433	4.33%	0.51	0.49	221	212
12	400	4.00%	0.50	0.50	200	200
13	368	3.68%	0.52	0.48	191	177
14	433	4.33%	0.51	0.49	221	212
15	568	5.68%	0.53	0.47	301	267
16	733	7.33%	0.49	0.51	359	374
17	563	5.63%	0.47	0.53	265	298
18	500	5.00%	0.47	0.53	235	265
19	468	4.68%	0.47	0.53	220	248
20	400	4.00%	0.46	0.54	184	216
21	333	3.33%	0.48	0.52	160	173
22	233	2.33%	0.48	0.52	112	121
23	233	2.33%	0.48	0.52	112	121
AADT	**10,000**	**100.00%**	**-**	**-**	**5,088**	**4,912**

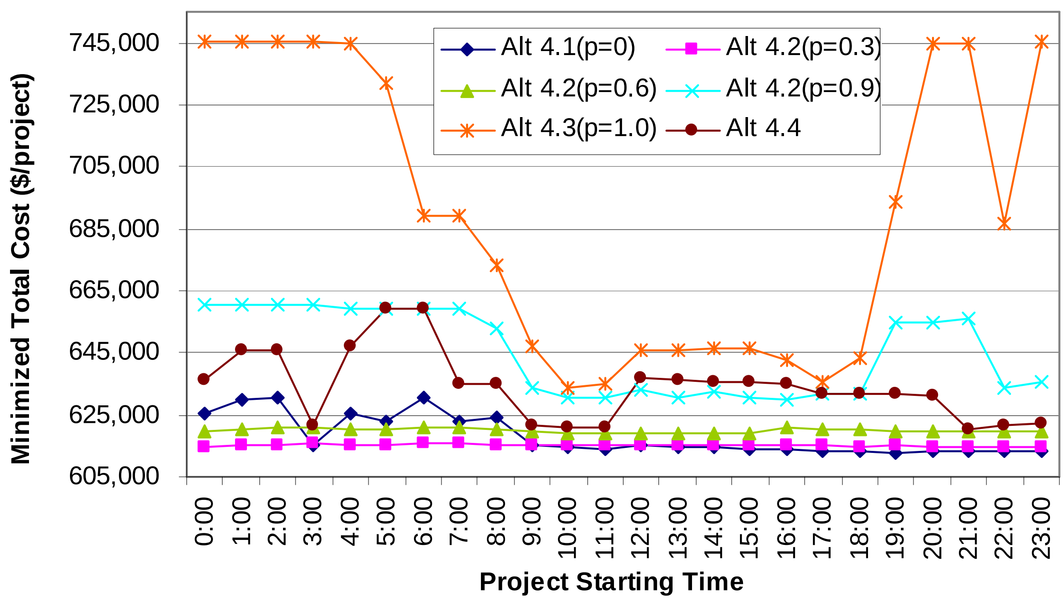
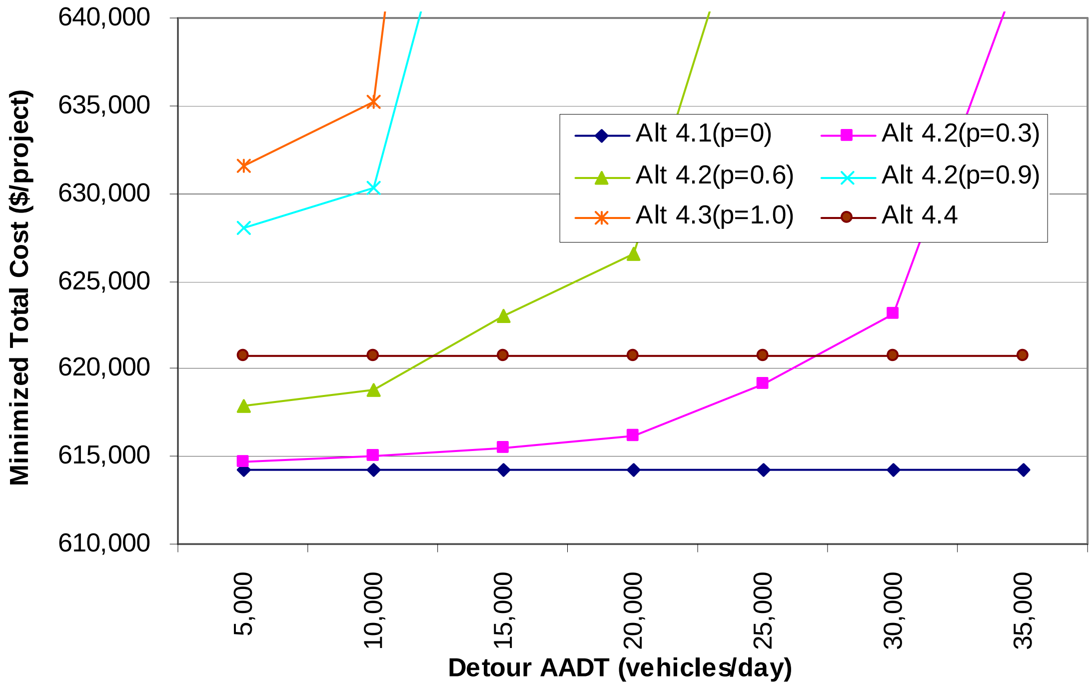


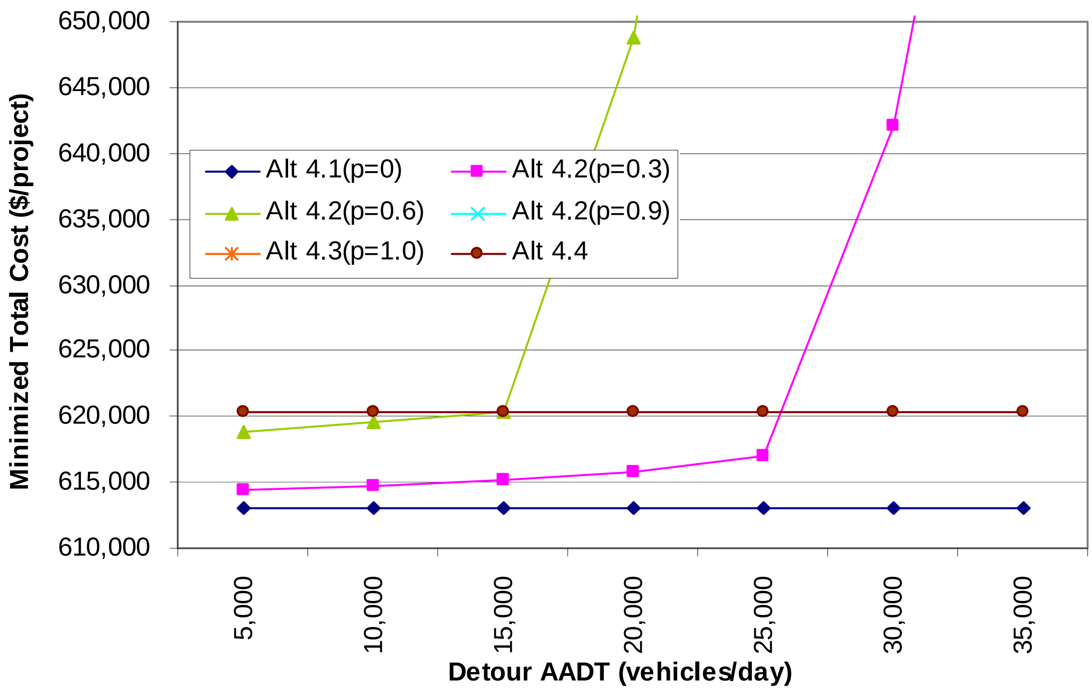
Figure 5.7 Minimized Total Cost vs. Project Starting Time (Four-lane Highway Work Zones)

Figure 5.8 shows the relation between the minimized total cost and the detour AADT for two project starting times: (a) 11:00 and (b) 21:00. In (a) at 11:00 the minimized total costs of all alternatives are much closer. In (b) Alternative 4.1 has the lowest minimized total cost. The minimized total costs of Alternatives 4.1 and 4.4 are not sensitive to detour AADT because these two alternatives do not consider any detour. There is no detour AADT threshold in Figures 5.8(a) and (b) because higher detour AADT and higher diverted flow increase the queuing delays on the detour quickly so that no threshold with Alternative 4.1 occurs.

Figure 5.9 shows the relation between the minimized total cost and the main road AADT when the project starting time is 11:00. There is one threshold and Alternatives 4.1 and 4.2 (p=0.3) successively define the lowest cost envelope. The threshold occurs at 37,000 vehicles per day, beyond which Alternative 4.2 (p=0.3) becomes preferable to Alternative 4.1. Alternatives 4.2 (p=0.9) and 4.3, which have higher diverted fraction, have sharp increases in minimized total costs due to sharp increases in detour queuing delay and never become preferable alternatives. In Chapter 3, based on steady traffic inflows and without considering detour queuing delay, Figure 3.18 indicates that a higher diverted fraction (Alternative 4.3) is preferable when Q_1 is lower than 800 vph. However, compared to Figure 3.18, alternatives with higher diverted fraction never become preferable because the detour queuing delay is considered here.



(a)



(b)

Figure 5.8 Minimized Total Cost vs. Detour AADT (a) Project Starting Time: 11:00 (b) Project Starting Time: 21:00

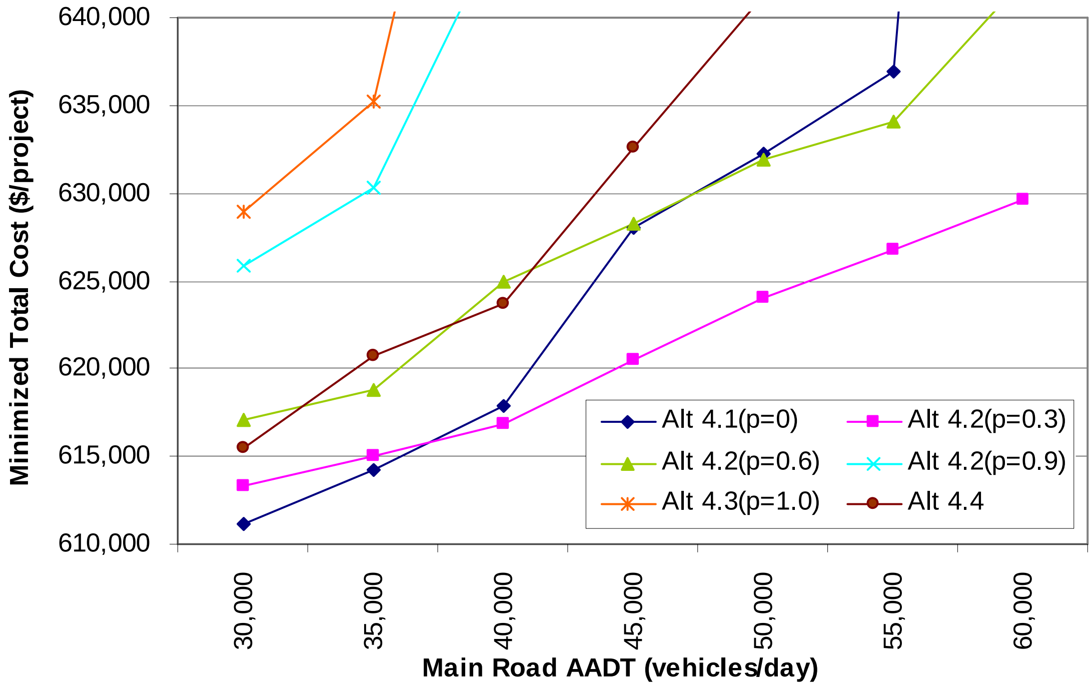


Figure 5.9 Minimized Total Cost vs. Main Road AADT (Project Starting Time: 11:00)

Tables 5.5(a) and (b) show the optimized solutions for Alternatives 4.1 and 4.2 (p=0.3). Alternative 4.2 (p=0.3) has lower minimized total cost than Alternative 4.1 as the main road AADT is 40,000 vehicles per day. Compared to Table 5.5(b), Table 5.5(a) indicates that, for a higher main road AADT and without a detour, the optimized number of zones increases to avoid the moving delay and the optimized idling time increases to avoid queuing delay along work zones. The optimized solution of Alternative 4.2 (p=0.3) in Table 5.5(b) shows fewer zones and no idling time decrease the maintenance cost and idling cost and the solution reaches the lowest minimized total cost for all alternatives. In such a case, the considerably lower agency cost, including maintenance cost and idling cost, for Alternative 4.2 (p=0.3) is the key factor in reaching the lowest minimized total cost, even if it has higher user cost than Alternative 4.1.

Table 5.5(a) Optimized Results for Numerical Example, Main Road AADT=40,000 veh/day, Project Starting Time: 11:00, Alternative 4.1

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.53	5.20	11.00	16.20	-	44,003
2	2.06	14.38	16.91	7.29	0.71	168,469
3	0.71	6.28	9.91	16.19	2.62	60,685
4	2.06	14.38	16.91	7.29	0.72	168,477
5	0.71	6.28	9.91	16.19	2.62	60,685
6	1.41	10.48	16.91	3.39	0.72	115,538
Total	7.50	57.00			7.39	617,857
Maintenance cost						606,000
Queuing delay cost						633
Moving delay cost						5,286
Idling cost						5,910
Accident Cost						28
Total cost						617,857
Total cost/project-km ($/lane-km)						82,381

Table 5.5(b) Optimized Results for Numerical Example, Main Road AADT=40,000 veh/day, Project Starting Time: 11:00, Alternative 4.2, *p*=0.3

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	2.80	18.82	11.00	5.82	-	229,912
2	2.11	14.68	5.82	20.50	0.00	174,841
3	2.58	17.50	20.50	14.00	0.00	212,120
Total	7.50	51.00			0.00	616,873
Maintenance cost						603,000
Queuing delay cost						52
Moving delay cost						13,756
Idling cost						0
Accident Cost						65
Total cost						616,873
Total cost/project-km ($/lane-km)						82,250

5.5 Numerical Examples – Mixed Alternatives

In Figure 5.3, the minimized total costs for different detour AADT can be obtained through threshold analysis. Here Figure 5.3 is modified by adding a curve which represents the minimized total costs for mixed alternatives. The modified result is shown in Figure 5.10.

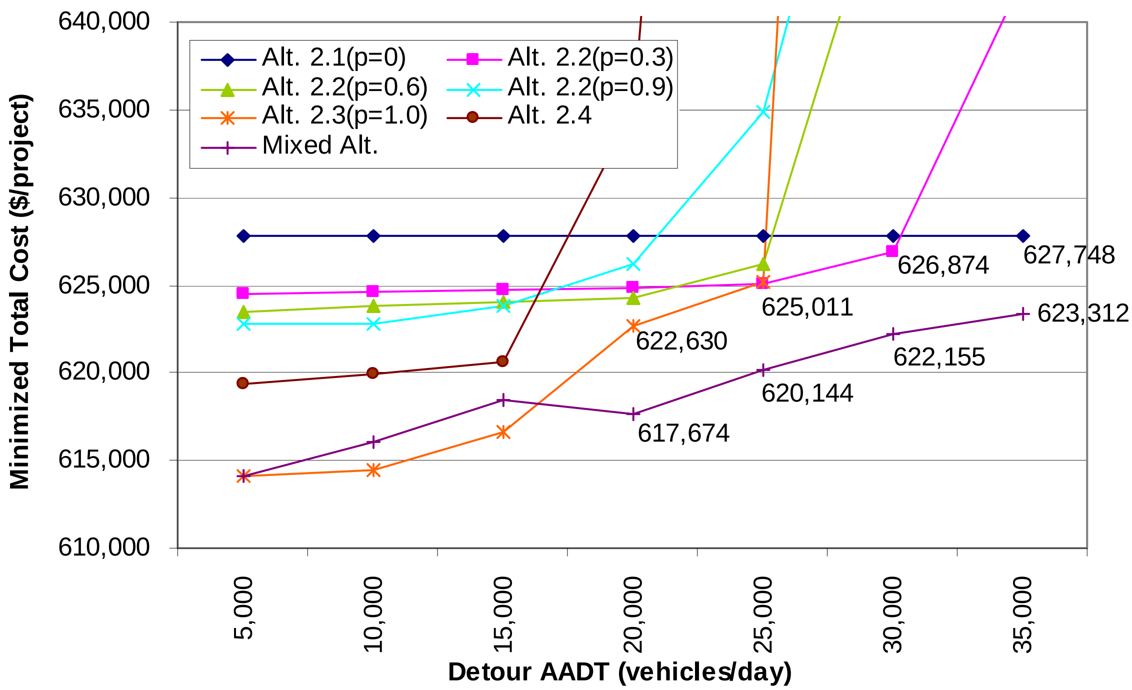


Figure 5.10 Minimized Total Cost vs. Detour AADT

Figure 5.10 shows that in most situations mixed alternatives can yield much lower minimized total costs than the envelope, on which Alternatives 2.1, 2.2, and 2.3 are included in Figure 5.3, when the detour AADT increases.

When the detour AADT is at its baseline value, 5,000 vehicles per day, Alternative 2.3 has the lowest minimized total cost at $614,073/project, and no further improvement is obtainable from mixed alternatives. Tables 5.6 (a) and (b) show that the optimized results for Alternative 2.3 and mixed alternatives at 11:00 has the same minimized total cost but the solutions are slightly different.

When the detour AADT increases to 20,000 vehicles per day, Alternative 2.3 has the lowest minimized total cost at $622,630/project in Figure 5.3. However, one lower minimized total cost, at $617,674/project, is found in Figure 5.10 by considering mixed alternatives. Tables 5.7 (a) and (b) show the optimized results for Alternative 2.3 and the mixed alternatives with an 11:00 start. Table 5.7(b) indicates that only 50% of flow in Direction 1 can be diverted to the detour during daytime, i.e. in zone 1; full diversion is applied at other zones when the detour AADT reaches 20,000 vehicles per day

Tables 5.8(a), (b), and (c) indicate the optimized results for mixed alternatives when the detour AADT reaches 25,000, 30,000, and 35,000 vehicles per day, respectively. The minimized total costs with mixed alternatives are also lower than the envelope with Alternative 2.2 (p=0.3) and Alternative 2.1. The differences of total costs are $4,867, $4,719, and $4,436, respectively. Tables 5.7 and 5.8 show that partial diversion or no diversion is applied during daytime, i.e., in zones 3 and 5 in Table 5.8(b); full diversion is applied during nighttime, i.e., in zones 2,4, and 6 in Table 5.8(b).

From Figure 5.10 and Tables 5.6 to 5.8, we can find that when detour AADT is low, e.g., 5,000 vehicles per day, the minimized total cost can be obtained by the uniform alternatives applied for an entire project. As detour AADT increases, mixed alternatives that integrate no diversion, partial diversion, or full diversion, in different zones can yield lower minimized total cost than uniform alternatives. Thus, an appropriate traffic management plan should be developed based on different traffic inflows.

Table 5.6(a) Optimized Results for Numerical Example, Detour AADT=5,000 veh/day, Project Starting Time: 11:00, Alternative 2.3

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	2.65	17.92	11.00	4.92	-	216,854
2	2.22	15.34	4.92	20.26	0.00	182,824
3	2.62	17.74	20.26	14.00	0.00	214,395
Total	7.50	51.00			0.00	614,073
Maintenance cost						603,000
Queuing delay cost						0
Moving delay cost						11,021
Idling cost						0
Accident Cost						52
Total cost						614,073
Total cost/project-km ($/lane-km)						81,876

Table 5.6(b) Optimized Results for Numerical Example, Detour AADT=5,000 veh/day, Project Starting Time: 11:00, Mixed Alternatives

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Prefered Zone Alt.	Prefered Diverted Fraction	Total Cost ($/zone)
1	2.65	17.91	11.00	4.91	-	23	1.00	216,704
2	2.23	15.36	4.91	20.27	0.00	23	1.00	183,125
3	2.62	17.73	20.27	14.00	0.00	23	1.00	214,244
Total	7.50	51.00			0.00			614,073
Maintenance cost								603,000
Queuing delay cost								0
Moving delay cost								11,021
Idling cost								0
Accident Cost								52
Total cost								614,073
Total cost/project-km ($/lane-km)								81,876

Table 5.7(a) Optimized Results for Numerical Example, Detour AADT=20,000 veh/day, Project Starting Time: 11:00, Alternative 2.3

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.50	5.00	11.00	16.00	-	42,043
2	0.86	7.16	16.79	23.95	0.79	71,743
3	1.11	8.66	23.95	8.61	0.00	91,133
4	0.89	7.34	8.61	15.95	0.00	74,024
5	0.86	7.16	16.79	23.95	0.84	71,785
6	1.11	8.66	23.95	8.61	0.00	91,135
7	0.89	7.34	8.61	15.95	0.00	74,023
8	0.47	4.82	16.79	21.61	0.84	40,278
9	0.81	6.86	21.61	4.47	0.00	66,466
Total	7.50	63.00			2.47	622,630
Maintenance cost						609,000
Queuing delay cost						180
Moving delay cost						11,421
Idling cost						1,974
Accident Cost						55
Total cost						622,630
Total cost/project-km ($/lane-km)						83,017

Table 5.7(b) Optimized Results for Numerical Example, Detour AADT=20,000 veh/day, Project Starting Time: 11:00, Mixed Alternatives

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Prefered Zone Alt.	Prefered Diverted Fraction	Total Cost ($/zone)
1	0.65	5.93	11.00	16.93	-	22	0.50	54,965
2	2.34	16.01	16.93	8.94	0.00	23	1.00	190,984
3	2.10	14.57	8.94	23.51	0.00	23	1.00	173,720
4	2.41	16.49	23.51	16.00	0.00	23	1.00	198,005
Total	7.50	53.00			0.00			617,674
Maintenance cost								604,000
Queuing delay cost								2,113
Moving delay cost								11,497
Idling cost								0
Accident Cost								64
Total cost								617,674
Total cost/project-km ($/lane-km)								82,357

Table 5.8(a) Optimized Results for Numerical Example, Detour AADT=25,000 veh/day, Project Starting Time: 11:00, Mixed Alternatives

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Prefered Zone Alt.	Prefered Diverted Fraction	Total Cost ($/zone)
1	0.70	6.18	11.00	17.18	-	22	0.30	58,602
2	1.66	11.94	17.18	5.13	0.00	23	1.00	135,464
3	1.31	9.84	5.13	14.97	0.00	23	1.00	108,139
4	3.35	22.07	16.98	15.05	2.00	23	1.00	275,796
5	0.49	4.95	15.05	20.00	0.00	21	0.00	42,143
Total	7.50	55.00			2.00			620,144
Maintenance cost								605,000
Queuing delay cost								2,737
Moving delay cost								10,741
Idling cost								1,602
Accident Cost								64
Total cost								620,144
Total cost/project-km ($/lane-km)								82,686

Table 5.8(b) Optimized Results for Numerical Example, Detour AADT=30,000 veh/day, Project Starting Time: 11:00, Mixed Alternatives

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Prefered Zone Alt.	Prefered Diverted Fraction	Total Cost ($/zone)
1	0.50	4.99	11.00	15.99	-	21	0.00	41,729
2	2.16	14.95	17.01	7.97	1.02	23	1.00	177,289
3	0.83	6.97	9.03	15.99	1.06	22	0.30	70,235
4	2.31	15.85	17.01	8.86	1.01	23	1.00	189,905
5	0.85	7.09	8.86	15.94	0.00	22	0.30	71,171
6	0.86	7.15	17.00	0.16	1.06	23	1.00	71,826
Total	7.50	55.00			2.00			620,144
Maintenance cost								606,000
Queuing delay cost								3,803
Moving delay cost								8,966
Idling cost								3,326
Accident Cost								60
Total cost								622,155
Total cost/project-km ($/lane-km)								82,954

Table 5.8(c) Optimized Results for Numerical Example, Detour AADT=35,000 veh/day, Project Starting Time: 11:00, Mixed Alternatives

Zone No.	Optimized length (km)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Prefered Zone Alt.	Prefered Diverted Fraction	Total Cost ($/zone)
1	0.50	4.99	11.00	15.99	-	21	0.00	41,741
2	2.01	14.05	17.94	7.99	1.94	23	1.00	165,763
3	0.67	6.01	9.97	15.98	1.98	21	0.00	57,439
4	2.03	14.17	17.92	8.09	1.94	23	1.00	167,357
5	0.66	5.95	8.09	14.03	0.00	22	0.40	55,469
6	0.32	3.91	14.03	17.95	0.00	21	0.00	27,516
7	1.32	9.91	17.95	3.86	0.00	23	1.00	108,026
Total	7.50	59.00			5.86			623,313
Maintenance cost								607,000
Queuing delay cost								3,827
Moving delay cost								7,741
Idling cost								4,690
Accident Cost								55
Total cost								623,313
Total cost/project-km ($/lane-km)								83,108

Chapter VI Work Zone Optimization with Multiple Detour Paths

In Chapter 5 the SAMASD algorithm for selecting alternatives for various zones in a maintenance project was developed to optimize work zone scheduling and diverted fractions while considering a single detour. In Chapter 6, work zone optimization models for a road network with multiple detour paths and the SAMAMD (Simulated Annealing for Mixed Alternatives with Multiple Detour paths) algorithm are developed. For analyzing traffic diversion through multiple detour paths in a road network, the SAUAMD (Simulated Annealing algorithm for Uniform Alternatives with Multiple Detour paths) and the SAMAMD algorithms are used to optimize work zone lengths and schedule the resurfacing work. Simulation analyses based on CORSIM are used not only to estimate delay cost, but also to evaluate the effectiveness of optimization models. In a case study, a comparison of the results from optimization and simulation models indicates that they are consistent. The optimization models do significantly reduce total cost, including user cost and maintenance cost.

6.1 Types of Multiple Detour Paths

In a road network with multiple detour paths, the diverted flow from Direction 1 can be assigned to more than one detour. Figure 6.1 shows several types of multiple detour paths. A prototype of a road network with multiple detour paths is shown in Figure 6.1(a). Four diverted fractions, *p, q, r, k,* occur in this network. The flow pQ_1 is diverted toward segments A → C → F while the flow qQ_1 is diverted along segments A → G → H→ B. The remaining flow $(1-p-q)Q_1$ goes through work zone toward segment AE. Then the diverted flow pQ_1 is separated into two flows: pkQ_1 along F → D → B and $p(1-k)Q_1$

along F $\rightarrow$ E $\rightarrow$ B. The diverted flow $(1-p-q)Q_1$ is also separated into two flows: $r(1-p-q)Q_1$ along E $\rightarrow$ F $\rightarrow$ D $\rightarrow$ B and $(1-r)(1-p-q)Q_1$ along E $\rightarrow$ B. The flow volumes on each segment are shown in Figure 6.1(a).

However, some road networks may be simpler than Figure 6.1(a). Figures 6.1(b) to (f) show five special cases simplified from Figure 6.1(a). These five network configurations are as follows:

1. Figure 6.1(b): Maintained Segment: AB, Diverted Fraction: p, q, No segment EF, k=0, r=0.

   Two separate detours are available for maintenance on segment AB. The flow pQ_1 is diverted along segments A $\rightarrow$ C $\rightarrow$ D $\rightarrow$ B while the flow qQ_1 is diverted along segments A $\rightarrow$ G $\rightarrow$ H$\rightarrow$ B.

2. Figure 6.1(c): Maintained Segment: AE, Diverted Fraction: p, k, No segments A $\rightarrow$ G $\rightarrow$ H $\rightarrow$ B, q=0, r=0.

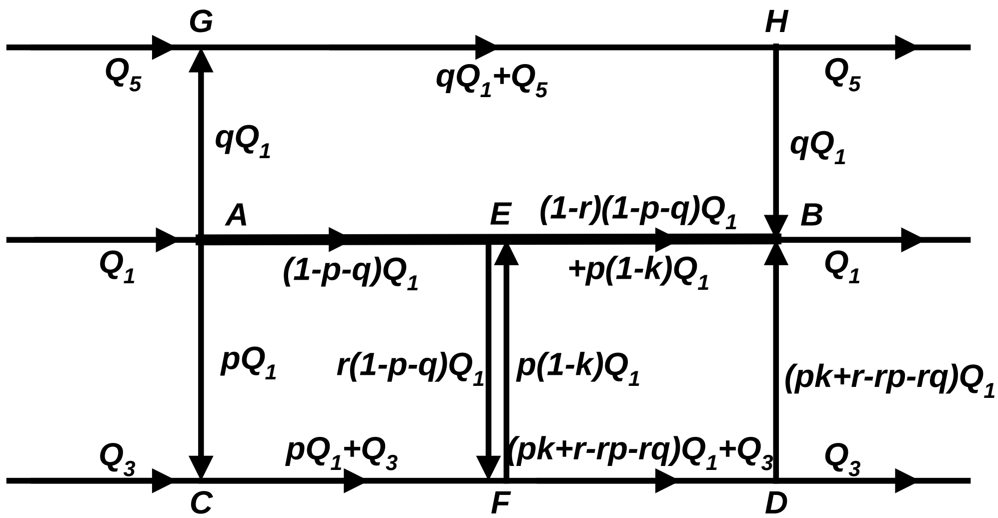
   The diverted flow pQ_1 can be considered as two separate flows: pkQ_1 along segments A $\rightarrow$ C $\rightarrow$ D $\rightarrow$ B and $p(1-k)Q_1$ along segments A $\rightarrow$ C $\rightarrow$ F $\rightarrow$ E.

3. Figure 6.1(d): Maintained Segment: EB, Diverted Fraction: p, r, No segments A $\rightarrow$ G $\rightarrow$ H $\rightarrow$ B, q=0, k=0.

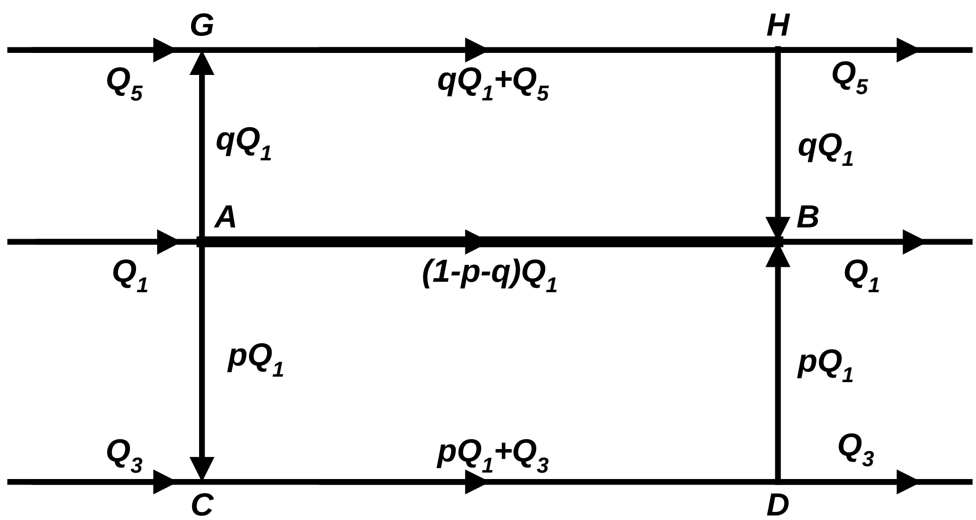
   The flow pQ_1 is diverted along segments A $\rightarrow$ C $\rightarrow$ D $\rightarrow$ B while the flow $r(1-p)Q_1$ is diverted along segments E $\rightarrow$ F $\rightarrow$ D $\rightarrow$ B.

4. Figure 6.1(e): Maintained Segment: AE, Diverted Fraction: p, q, k, r=0.

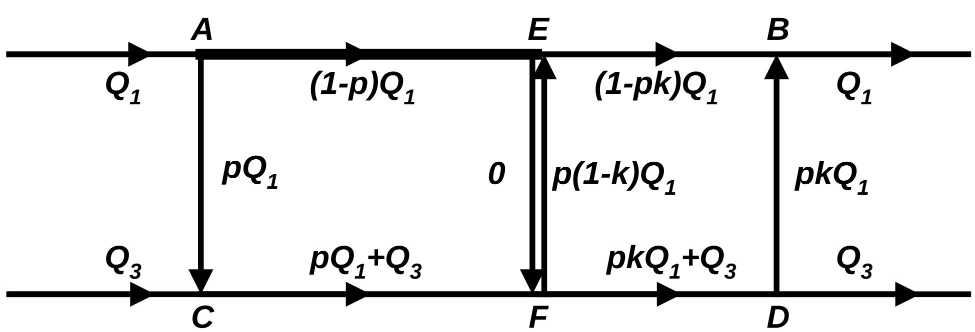
   The maintained segment is AE. No vehicles passing through AE will choose a longer trip along E $\rightarrow$ F $\rightarrow$ D $\rightarrow$ B instead of E $\rightarrow$ B.



(a) Maintained Segment: AB, Diverted Fraction: *p*, *q*, *r*, *k*

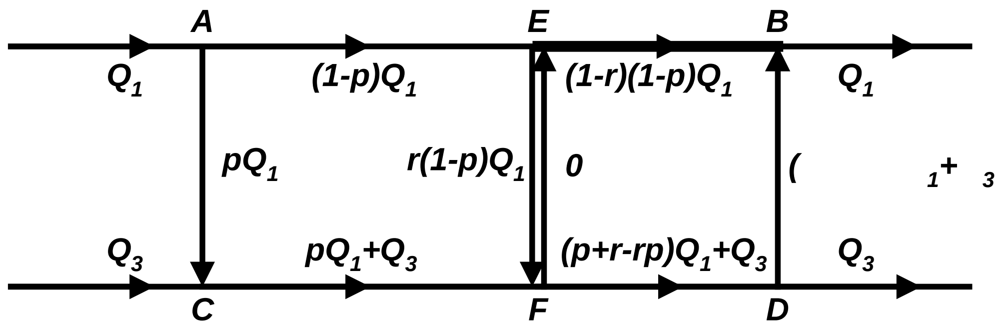


(b) Maintained Segment: AB, Diverted Fraction: *p*, *q*

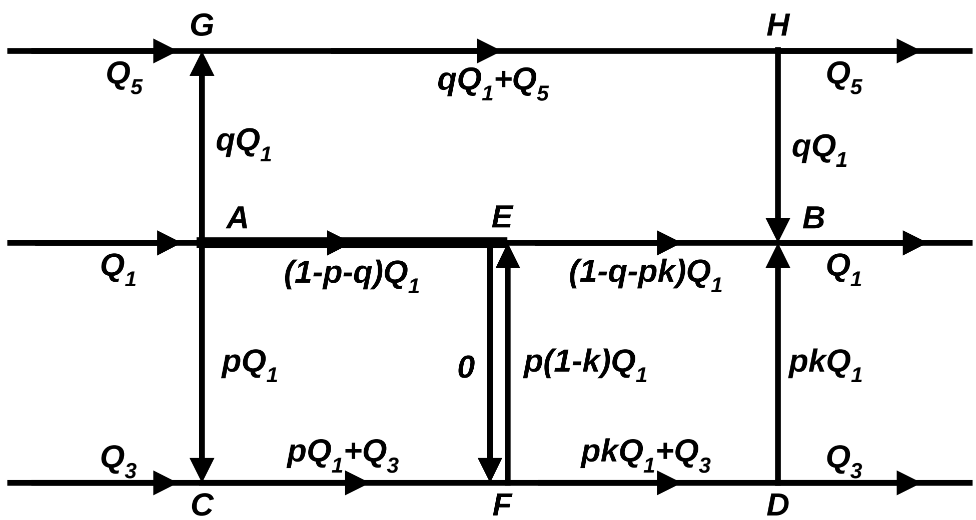


(c *p*, *k*

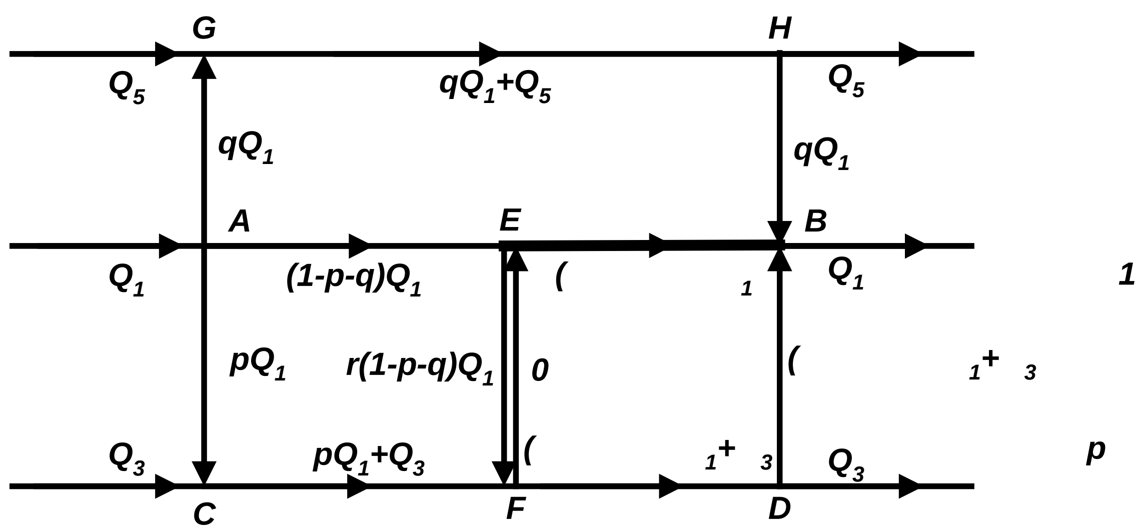
F i g



(d) Maintained Segment: EB, Diverted Fraction: *p*, *r*



(e) Maintained Segment: AE, Diverted Fraction: *p*, *q*, *k*



(f) Maintained Segment: EB, Diverted Fraction: *p*, *q*, *r*

Figure 6.1 Types of Multiple Detour Paths for Work Zones (continued)

5. Figure 6.1(f): Maintained Segment: EB, Diverted Fraction: p, q, r, k=0.

   The maintained segment is EB. No vehicles diverted to detour A $\rightarrow$ C $\rightarrow$ F will return to work zones along EB.

6.2 Optimization Models for Work Zones with Multiple Detour Paths

The model formulation for a road network with multiple detour paths as shown in Figure 6.1(a) is developed in this section. One multiple detour case study will be analyzed and a simulation model is developed to evaluate the work zone optimization models. The road network analyzed in this case study includes IS-95 and US1, from MD32, MD 175, to MD-100. This network is consistent with Figure 6.1(c). The model formulation for Figure 6.1(c) can be derived by setting q=0 and r=0 in the model derived below for Figure 6.1(a).

6.2.1 Extension of Optimization Model for Multiple-lane Highway

Although the optimization models in Section 4.1.2 (Alternative 4.1) and Section 5.1.3 (Alternatives 4.2, 4.3, and 4.4) are developed for four-lane highway work zones, the models can be extended to analyze work zones on multiple-lane highways with six and eight lanes. Some parameters in Figure 4.3, namely c_0, the maximum discharge rate without the work zone, and c_w, the maximum discharge rate along the work zone, are redefined here. c_0 is replaced with $n_l c_l$, where n_l is the number of lanes in Direction 1 and c_l is the maximum discharge rate (without a work zone) for one lane; c_w is replaced with $n_{rl} c_{wrl}$, where n_{rl} is the number of the remaining lanes along a work zone and c_{wrl} is the maximum lane discharge rate along a work zone. c_l (maximum discharge rate without a work zone for one lane), c_{wrl} (maximum lane discharge rate along a work zone), V_w (average work zone speed), and V_a (average approaching speed) will be given higher values for the IS-95 freeway case in this chapter instead of the baseline values in Tables 3.1 and 3.5 for four-lane rural highway work zones.

6.2.2 Model Formulation

According to the definitions of four alternatives for four-lane highway work zones in Section 3.2, we can define Alternative 8.1 as having no detour and one lane closed for Q_1^{ij} traffic ($p=0$) for eight-lane highway work zones, Alternative 8.2 as having $(1-p-q)Q_1^{ij}$ traffic through the detour and one lane closed, and Alternative 8.3 as having all Q_1^{ij} traffic through the detour and allowing work on all lanes in Direction 1 ($p+q=1$). Because Alternatives 8.1 and 8.3 are special cases of Alternative 8.2 with $p+q$=0 and 1, only the model of Alternative 8.2 with multiple detour paths needs to be derived below.

1. Queuing Delay Cost

Figure 6.1(a) shows that the fraction $p+q$ of the flow Q_1^{ij} in Direction 1 is diverted to the alternate routes. The user queuing delay cost of the remaining flow $(1-p-q)Q_1^{ij}$ in Direction 1 for work zone i, $C^{82}_{q(1-p-q),\,i}$, is the area C in Figure 4.3 multiplied by v but with $(1-p-q)Q_1^{ij}$ substituted for Q_1^{ij}, with $n_l c_l$ substituted for c_0, and with $n_{rl}c_{wrl}$ substituted for c_w, where n_l is 4 (4 lanes in Direction 1) in a eight-lane highway and n_{rl} is 3 when one lane is closed. The user queuing delay cost of the flow $(1-p-q)Q_1^{ij}$ is:

$$C^{82}_{q(1-p-q),i} = (\,area\ of\ C\,)v \tag{6.1}$$

The diverted flow pQ_1 can be considered as two separate flows: pkQ_1^{ij} along A $\rightarrow$ C $\rightarrow$ D $\rightarrow$ B and $p(1-k)Q_1^{ij}$ along A $\rightarrow$ C $\rightarrow$ F $\rightarrow$ E. The detour queuing delay costs for pkQ_1^{ij} and $p(1-k)Q_1^{ij}$ are considered together. The possible detour queuing delay cost of the diverted flow pQ_1^{ij} and Q_3^{ij} in Direction 3 along CD for zone *i*, denoted $C_{qdCD,i}^{82}$, is the area C in Figure 5.1 multiplied by *v*.

$$C_{qdCD,i}^{82} = (\text{area of } C)v \tag{6.2}$$

The queuing delay costs for pkQ_1^{ij} and $p(1-k)Q_1^{ij}$ due to intersection signal or stop delay along detour are considered separately. The user delay cost of the diverted flow pkQ_1^{ij} from Direction 1 along the detour A $\rightarrow$ C $\rightarrow$ D $\rightarrow$ B due to intersection signal or stop delay, denoted as $C_{int,pk,i}^{82}$, is:

$$C_{int,pk,i}^{82} = \sum_{j}^{n} pkQ_1^{ij} D_i N_{int,CD} \frac{t_{int}}{3600} v \tag{6.3}$$

where $N_{int,CD}$ is the number of intersections along CD.

The user delay cost of the diverted flow $p(1-k)Q_1^{ij}$ from Direction 1 along the detour A $\rightarrow$ C $\rightarrow$ F $\rightarrow$ E due to intersection signal or stop delay, denoted as $C_{int,p(1-k),i}^{82}$, is:

$$C_{int,p(1-k),i}^{82} = \sum_{j}^{n} p(1-k)Q_1^{ij} D_i N_{int,CF} \frac{t_{int}}{3600} v \tag{6.4}$$

where $N_{int,CF}$ is the number of intersections along CF.

The remaining flow $(1\text{-}p\text{-}q)Q_1$ can be considered as two separate flows: $r(1-p-q)Q_1^{ij}$ along E $\rightarrow$ F $\rightarrow$ D $\rightarrow$ B (on the detour) and $(1-r)(1-p-q)Q_1^{ij}$ along E $\rightarrow$ B (on the main road). The detour queuing delay costs for $r(1-p-q)Q_1^{ij}$ are considered here. The possible detour queuing delay cost of the diverted flow $r(1-p-q)Q_1^{ij}$ and $pkQ_1^{ij}+Q_3^{ij}$ in Direction 3 along FD for zone *i*, denoted $C_{qdFD,i}^{82}$, is the area C in Figure 5.1 multiplied by *v* but with $r(1-p-q)Q_1^{ij}$ substituted for pQ_1^{ij} and with $pkQ_1^{ij}+Q_3^{ij}$ substituted for Q_3^{ij}.

$$C_{qdFD,i}^{82} = (\,area\ of\ C\,)v \tag{6.5}$$

The queuing delay cost of the diverted flow $r(1-p-q)Q_1^{ij}$ from Direction 1 along the detour E $\rightarrow$ F $\rightarrow$ D $\rightarrow$ B due to intersection signal or stop delay, denoted as $C_{int,r(1-p-q),i}^{82}$, is:

$$C_{int,r(1-p-q),i}^{82} = \sum_{j}^{n} r(1-p-q)Q_1^{ij} D_i N_{int,FD} \frac{t_{int}}{3600} v \tag{6.6}$$

where $N_{int,FD}$ is the number of intersections along FD.

The other diverted flow qQ_1 may also yield possible detour queuing delay along A $\rightarrow$ G $\rightarrow$ H $\rightarrow$ B. The possible detour queuing delay cost of the diverted flow qQ_1^{ij} and Q_5^{ij} in Direction 5 along GH for zone *i*, denoted $C_{qdGH,i}^{82}$, is the area C in Figure 5.1 multiplied by *v* but with qQ_1^{ij} substituted for pQ_1^{ij} and with Q_5^{ij} substituted for Q_3^{ij}. (Direction 5 is defined as the direction along GH and the original flow along GH is Q_5^{ij}.)

$$C_{qdGH,i}^{82} = (\,area\ of\ C\,)v \tag{6.7}$$

The combined queuing delay cost for the maintained road AE and the detours, C_{qi}^{82}, can be derived as:

$$C_{qi}^{82} = C_{q(1-p-q),i}^{82} + C_{qdCD,i}^{82} + C_{qdFD,i}^{82} + C_{qdGH,i}^{82} + C_{int,pk,i}^{82} + C_{int,p(1-k),i}^{82} + + C_{int,r(1-p-q),i}^{82} \tag{6.8}$$

2. Moving Delay Cost

The moving delay cost of the traffic flows $(1-p-q)Q_1^{ij}$ in work zone *i*, denoted $C^{82}_{v(1-p-q),i}$, is:

$$C^{82}_{v(1-p-q),i} = (\frac{L_i}{V_w} - \frac{L_i}{V_a})(1-p-q)Q_1^{ij} D_{ij} v \qquad when\ (1-p-q)Q_1^{ij} \le n_{rl} c_{wrl} \quad (6.9a)$$

$$C^{82}_{v(1-p-q),i} = (\frac{L_i}{V_w} - \frac{L_i}{V_a}) n_{rl} c_{wrl} D_{ij} v \qquad when\ (1-p-q)Q_1^{ij} > n_{rl} c_{wrl} \quad (6.9b)$$

The moving delay costs for pkQ_1^{ij} and $p(1-k)Q_1^{ij}$ along the detour are considered separately. The moving delay cost of the diverted flow pkQ_1^{ij} from Direction 1, denoted as $C^{82}_{vpk,i}$, is:

$$C^{82}_{vpk,i} = \sum_j^n pkQ_1^{ij} D_i [\frac{L_{AC}+L_{DB}}{V_0} + \frac{L_{CF}}{V^{*3}_{d,CF}} + \frac{L_{FD}}{V^{*3}_{d,FD}} - \frac{L_{AB}}{V_0}] v \quad (6.10)$$

where L_{AC}, L_{DB}, and L_{CD} are the lengths of segments AC, DB, and CD along the detour and L_{AB} is the length of AB along the main road. $V^{*3}_{d,CF}$ is the detour speed affected by pQ_1^{ij} along CF. $V^{*3}_{d,FD}$ is the detour speed affected by $(pk + r - rp - rq)Q_1^{ij}$ along FD.

The moving delay cost of the diverted flow $p(1-k)Q_1^{ij}$ from Direction 1, denoted as $C^{82}_{vp(1-k),i}$, is:

$$C^{82}_{vp(1-k),i} = \sum_j^n p(1-k)Q_1^{ij} D_i [\frac{L_{AC}+L_{FE}}{V_0} + \frac{L_{CF}}{V^{*3}_{d,CF}} - \frac{L_{AE}}{V_0}] v \quad (6.11)$$

where L_{FE} and L_{CF} are the lengths of segments FE and CF along the detour and L_{AE} is the length of AE along the main road.

The moving delay cost of the diverted flow $r(1-p-q)Q_1^{ij}$ from Direction 1, denoted as $C^{82}_{vr(1-p-q),i}$, is:

$$C^{82}_{vr(1-p-q),i} = \sum_j^n r(1-p-q)Q_1^{ij} D_i [\frac{L_{EF} + L_{DB}}{V_0} + \frac{L_{FD}}{V^{*3}_{d,FD}} - \frac{L_{EB}}{V_0}] v \tag{6.12}$$

where L_{EF} is the length of segment EF along the detour and L_{EB} is the length of EB along the main road.

The moving delay cost of the diverted flow qQ_1^{ij} from Direction 1, denoted as $C^{82}_{vq,i}$, is:

$$C^{82}_{vq,i} = \sum_j^n qQ_1^{ij} D_i [\frac{L_{AG} + L_{HB}}{V_0} + \frac{L_{GH}}{V^{*5}_{d,GH}} - \frac{L_{AB}}{V_0}] v \tag{6.13}$$

where L_{AG} and L_{HB} are the lengths of segments AG and HB along the detour. $V^{*5}_{d,GH}$ is the detour speed affected by qQ_1^{ij} along GH in Direction 5.

The moving delay cost of the original flow on the detour along CD, Q_3^{ij}, as affected by the pQ_1^{ij} and $r(1-p-q)Q_1^{ij}$, denoted as $C^{82}_{v3,i}$, is:

$$C^{82}_{v3,i} = \sum_j^n Q_3^{ij} D_i (\frac{L_{CF}}{V^{*3}_{d,CF}} + \frac{L_{FD}}{V^{*3}_{d,FD}} - \frac{L_{CD}}{V_{d0}}) v \tag{6.14}$$

The moving delay cost of the original flow on the detour along GH, Q_5^{ij}, as affected by the qQ_1^{ij}, denoted as $C^{82}_{v5,i}$, is:

$$C^{82}_{v5,i} = \sum_j^n Q_5^{ij} D_i (\frac{L_{GH}}{V^{*5}_{d,GH}} - \frac{L_{GH}}{V_{d0}}) v \tag{6.15}$$

The combined moving delay cost for the maintained road AE and the detour C^{42}_{vi} can be derived as:

$$C_{vi}^{82} = C_{v(1-p),i}^{82} + C_{vpk,i}^{82} + C_{vp(1-k),i}^{82} + C_{vr(1-p-q),i}^{82} + C_{vq,i}^{82} + C_{v3,i}^{82} + C_{v5,i}^{82} \tag{6.16}$$

3. Idling Cost

The idling cost for zone i C_{Ii}^{82} is:

$$C_{Ii}^{82} = v_d \Delta t_i \tag{6.17}$$

4. Accident Cost

The accident cost for zone i, C_{ai}^{82}, is formulated as:

$$C_{ai}^{82} = \frac{(C_{qi}^{82} + C_{vi}^{82})}{v} \frac{n_a v_a}{10^8} \tag{6.18}$$

5. Maintenance Cost

The maintenance cost for zone i, C_{mi}^{82}, is $z_1 + z_2 L_i$. Then the total cost for zone i, C_{ti}^{82}, is:

$$C_{ti}^{82} = (z_1 + z_2 L_i) + C_{qi}^{82} + C_{vi}^{82} + v_d \Delta t_i + \frac{(C_{qi}^{82} + C_{vi}^{82})}{v} \frac{n_a v_a}{10^8} \tag{6.19}$$

6. Total Cost

The total cost for resurfacing road length L_T by scheduling m work zones, C_{PT}^{82}, is expressed as:

$$\begin{aligned} C_{PT}^{82} &= \sum_i^m C_{ti}^{82} \\ &= \sum_i^m (z_1 + z_2 L_i) + \sum_i^m C_{qi}^{82} + \sum_i^m C_{vi}^{82} + \sum_i^m v_d \Delta t_i + \sum_i^m \frac{(C_{qi}^{82} + C_{vi}^{82})}{v} \frac{n_a v_a}{10^8} \end{aligned} \tag{6.20}$$

The total cost in Eq.(6.20) will be minimized with the Simulated Annealing algorithms, including SAUAMD and SAMAMD. The SAUAMD (Simulated Annealing algorithm for Uniform Alternatives with Multiple Detour paths) follows the same procedures as SAUASD but its cost function is replaced by Eq.(6.20) for multiple detour paths. SAMAMD is derived below.

6.2.3 Simulated Annealing Algorithm for Mixed Alternatives with Multiple Detour Paths - SAMAMD

The optimization with SAUAMD and the threshold analysis for selecting alternatives in this case study will be presented in Section 6.4. Moreover, in order to further reduce total cost by considering mixed alternatives with different configurations in successive zones, an improved search method, SAMAMD (Simulated Annealing algorithm for Mixed Alternatives with Multiple Detour paths), is developed here for selecting alternatives in successive zones, where the diverted fractions, p and k, for each zone are optimized. The concept of this search method for multiple detour paths is similar to the SAMASD method shown in Section 5.2 but the new diverted fraction k along the additional detour is added and optimized. This search method can be obtained by modifying Figures 5.3 and 5.4.

The SAMAMD algorithm is as follows:

1. Add new variables k_i and $k_{opt,i}$ in Step 0 in Section 4.2.2 (the variables A_i, p_i, $A_{opt,i}$, $p_{opt,i}$ have been added in Section 5.2.2), where

   k_i: diverted fraction of pQ_1 along F $\rightarrow$ D $\rightarrow$ B for zone i, $k_i = 0 - 1$, $i=1, \ldots, m$;

   $k_{opt,i}$: final optimal diverted fraction of pQ_1 along F $\rightarrow$ D $\rightarrow$ B for zone i, $k_{opt,i} = 0 - 1$, $i=1, \ldots, m$.

The notation used for eight-lane highway alternatives is applied here. “81” represents Alternative 8.1. For other multiple-lane highway work zones, Alternatives 8.1, 8.2, and 8.3 can be replaced.

Set the initial $A_{opt,i}$, =81, $p_{opt,i}$ =0, $k_{opt,i}$ =0, i=*1*, …., m, for all zones.

2. Modify Figure 5.4. Test possible A_i, p_i, and k_i combinations and calculate the total cost for the current combination. If the total cost for the current combination is lower than for the previous combination, update $A_{opt,i}$, $p_{opt,i}$, and $k_{opt,i}$; otherwise, keep the previous solution and mixed alternatives. This procedure terminates when all possible A_i, p_i, and k_i combinations are tested. Figure 6.2 shows the flow chart for determining alternatives and diverted fractions in SAMAMD.

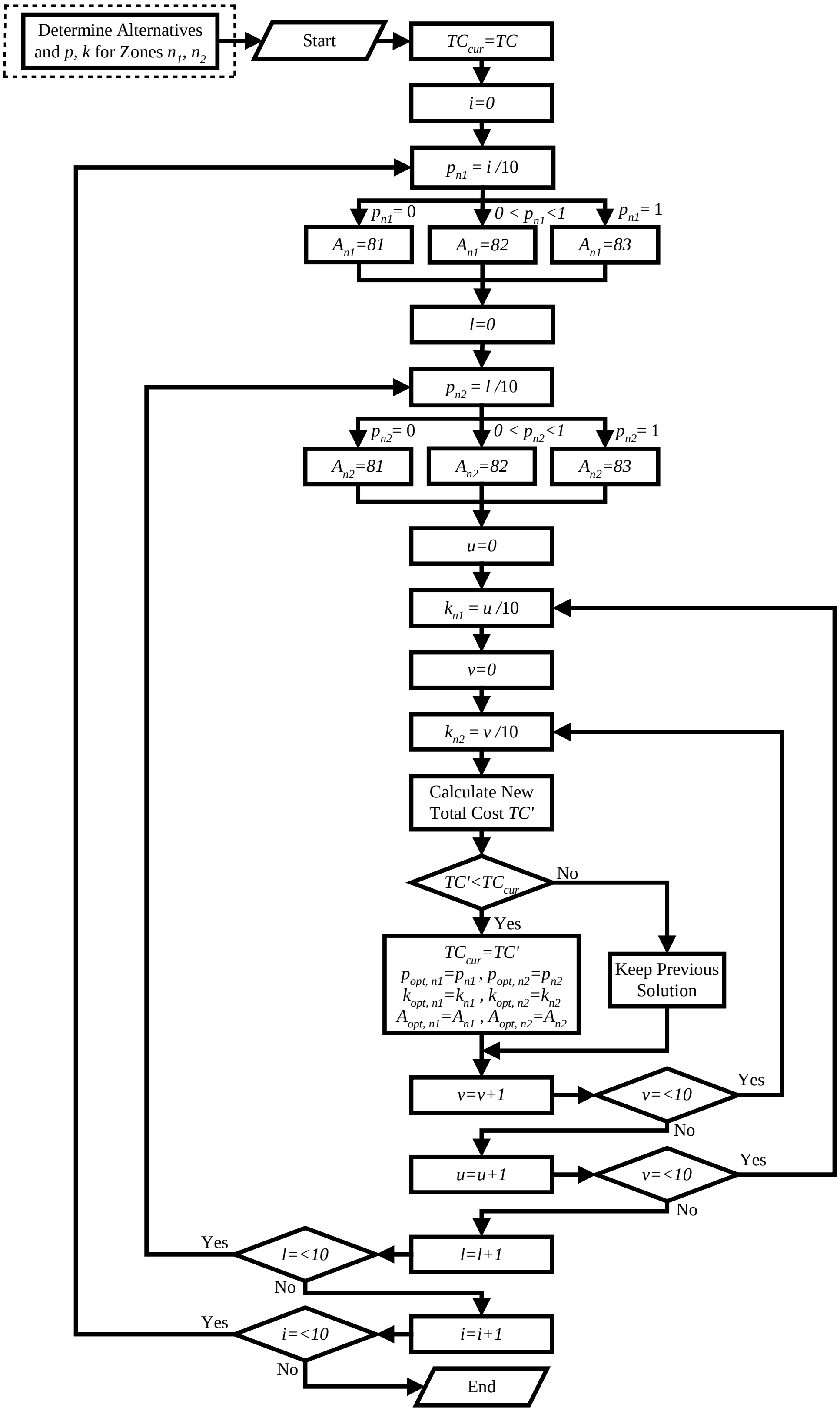


Figure 6.2 Determining Alternatives and Diverted Fractions in SAMAMD

6.3 Development of Simulation Model

6.3.1 Simulation Model for Work Zone

In Chapters 4 and 5, user costs are determined with heuristic algorithms, SAUASD and SAMASD. However, the user costs for a road network with multiple detour paths can also be obtained from simulation. Improved analytical models incorporating equilibrium assignment and queuing relations as well as a detailed simulation model using CORSIM are developed. Both of these analyze diversion of flows that vary over time through multiple paths in a highway network. The simulation model which analyzes users delay is developed to evaluate the optimized results obtained with the analytical models developed in this study and to evaluate the effectiveness of these analytical models. CORSIM (Corridor Simulator) is a microscopic simulation model developed by Federal Highway Administration (FHWA) which can be adapted to simulate traffic operations around a work zone. This can be done by assuming that a lane closure for a work zone results in the same impact on highway capacity as a lane blockage caused by an incident.

Note that the optimization models in Chapters 4 and 5 are based on a macroscopic model in which speed is derived from the relations among flow, speed, and density. CORSIM, a microscopic simulation model, is based on a car following theory, from which speeds are derived. A simulation model such as CORSIM provides a very comprehensive and detailed method for estimating the delays resulting from a work zone in a complex road network. Therefore, it will be used to estimate such user delay costs. The user costs will be obtained separately from analytic and simulation models and then compared.

6.3.2 Evaluation of Optimization Models by Simulation

The simulation model using CORSIM presented in this chapter is used not only for estimating user costs but also for evaluating the analytical models developed in this study. The current work zone policy and the optimized results will be simulated and compared. The current policy is the current work zone schedule used by highway agencies. It can be obtained from local highway agencies. The optimized results can be obtained with the SAMAMD algorithm which is developed in Section 6.2.3. Figure 6.3 shows how the effectiveness of optimization models is evaluated based on the CORSIM simulation model. The procedures are as follows:

1. Read input data of optimization model from original TRF file, which is the simulation input file of CORSIM (no work zone parameters are set in the original TRF file). Highway geometric characteristics, such as main road length, detour length, and traffic data, such as hourly traffic volumes and turn movement percentages, can be obtained from a TRF file. The input data for work zone optimization models can also be read from a TRF file.
2. Generate Optimized Solution: Run the optimization model. The optimized solution, including optimized work zone lengths, zone starting times, zone ending times, and diverted fractions can be obtained. Write these values into the original TRF file and save that as another TRF file. A new TRF file with optimized work zone schedule is generated.
3. Run Simulation based on Optimized Solution: Use this new TRF file to run CORSIM and obtain the simulation results for the previously optimized work zone schedule.

4. Specify Current Work Zone Policy and Simulate it: Write the current work zone policy into the original TRF file. A new TRF file with the current policy is then generated. Use this TRF file to run a CORSIM and obtain the simulated output for the current policy.
5. Evaluation of Optimization Model: First, compare the total cost of the current policy, estimated with the objective function developed in Eq.(6.14), and the total cost minimized by SAMAMD. Check if the minimized total cost is lower than the current total cost. If yes, this indicates that the optimized solution is effective and that current policy can be improved. Otherwise, the solution obtained with SAMAMD is not really optimized.

   Second, compare the simulation outputs based on the optimized solution and the current policy. Check if the simulated user cost of the optimized solution is lower than the simulated user cost for the current policy. If yes, this indicates that the simulation results are consistent with the optimization results. The optimization model does reduce total cost, including user cost and maintenance cost. Otherwise, the solution obtained with SAMAMD is not better than the current policy.

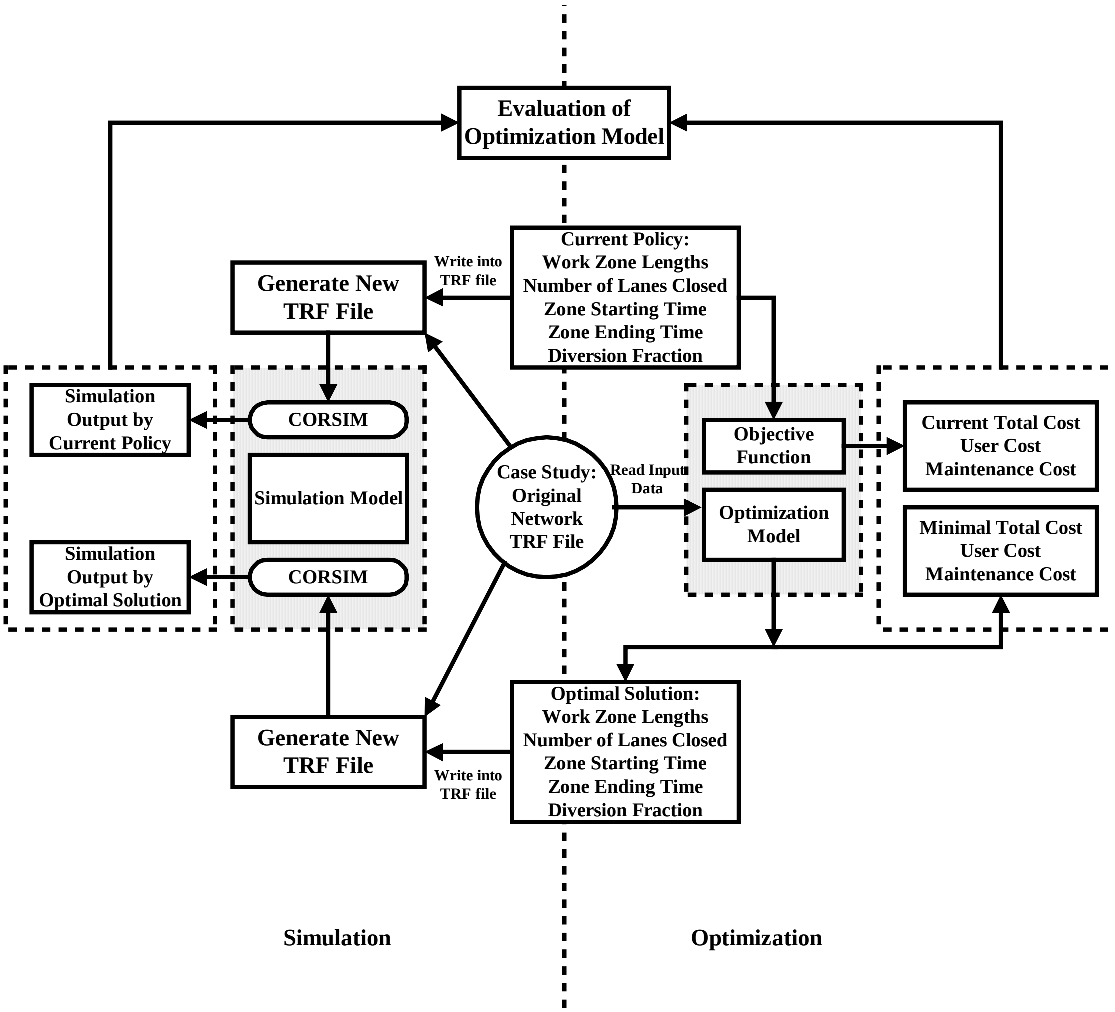


Figure 6.3 Evaluation of Work Zone Optimization Model by Simulation

6.4 Case Study

The road network analyzed in this case study includes IS-95 and US1, from MD32, MD 175, to MD-100. This case study will sequence and schedule unequal work zones based on the current policy and the optimized results for a 2.965-mile one lane maintenance project on IS-95 Northbound, from MD32 to MD175. The Maryland State Highway Administration shows that the current lane-closure policies for highway maintenance in Maryland (Chen, 2003) are 9:00 a.m. – 3:00 p.m. and 7:00 p.m. – 5:00 a.m. for single-lane closure; 10:00 p.m. – 5:00 a.m. for double-lane closure; and 12:00 a.m. – 5:00 a.m. for three-lane closure. Single-lane closure policy is applied for a one-lane maintenance project here.

6.4.1 Optimization Results

The optimization by SAUAMD and SAMAMD for IS-95 case study is presented in this section. The numerical values for each variable in this section were obtained from the Maryland State Highway Administration and shown in Table 6.1. Table 6.2 shows the AADT and hourly traffic distributions on the maintained road and the detour. The annual average daily traffic (AADT) in IS-95 Northbound is 94,438 vehicles. The annual average daily traffic on the detour, US-1 Northbound, is 26,377 vehicles per day. Two possible detour paths in Figure 6.1(c), pkQ_1 along A → C → D → B and $p(1-k)Q_1$ along A → C → F → E, are along IS-95 North → MD32 East → US-1 North → MD100 West → IS-95 North and along IS-95 North → MD32 East → US-1 North → MD175 West →IS-95 North, respectively.

Table 6.1 Inputs for Case Study for IS-95 Eight-Lane Freeway Work Zones

Variable	Description	Values
$AADT$	Annual average daily traffic on main Road	94,438
	Annual average daily traffic on detour	26,377
c_{d3}	Maximum discharge rate along detour CD	3,600 vph
c_l	Maximum discharge rate without work zone for one lane	2000 vph
c_{rwl}	Maximum discharge rate along work zone for one lane	1600 vph
L_T	Project road length = length of AE	2.965 miles
L_{AE}	Length of AE along detour	2.965 miles
L_{AB}	Length of AB along detour	4.933 miles
L_{AC}	Length of AC along detour	1.908 miles
L_{CF}	Length of CF along detour	2.453 miles
L_{CD}	Length of CD along detour	4.462 miles
L_{FE}	Length of FE along detour	0.602 miles
L_{DB}	Length of DB along detour	1.133 miles
$N_{int,CF}$	Number of intersections along detour CF	3
$N_{int,CD}$	Number of intersections along detour CD	3
n_a	Number of accidents per 100 million vehicle hour	40 acc/100mvh
t_{int}	Average waiting time per intersection	30 sec
V_a	Average approaching speed along AB	65 mile/hr
V	Average work zone speed	35 mile/hr
v	Value of user time	12 \$/veh·hr
v_a	Average accident cost	142,000 \$/accident
v_d	Average Cost of Idling Time	800 \$/hr
z_1	Fixed setup cost	1,300 \$/zone
z_2	Average maintenance cost per lane·mile	33,000 \$/lane·mile
z_3	Fixed setup time	2 hr/zone
z_4	Average maintenance time per lane·mile	9.6 hr/lane·mile

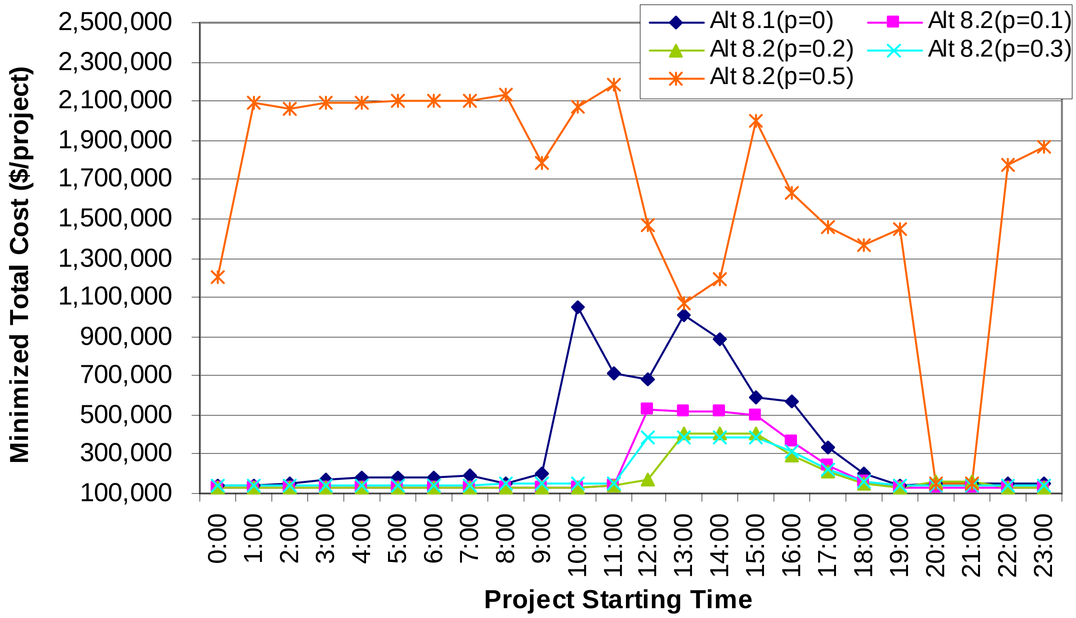
Figure 6.4 shows the minimized total cost and project starting time for Alternatives 8.1, 8.2 (p=0.1, 0.2, 0.3, and 0.5) when k = 0. Figures 6.4(a) and (b) are the same figures but different minimized total cost scale are shown. Figure 6.4(b) shows that Alternative 8.2 (p=0.1) has lowest minimized total cost among all alternatives. When the diverted fraction p reaches 0.5, the minimized total costs become very high because the

diverted flow plus the detour flow exceed the detour capacity and very high queuing delays occur. Much higher diverted flows for Alternatives 8.2 (p=0.6 – 0.9) and 8.3 (p=1.0) lead to much higher detour queuing delay than p=0.5 and these curves exceed the scale of $2,500,000 so that they are not shown in Figure 6.4. The best project times for Alternatives 8.1 (p=0), 8.2 (p=0.1), 8.2 (p=0.2), 8.2(p=0.3), and 8.2(p=0.5) are 19:00, 20:00, 23:00, 20:00, and 20:00, respectively. Based on the numerical values in Table 6.1, Alternative 8.2 (p=0.1) reaches the minimized total cost at 20:00 among all alternatives. Its minimized total cost is $126,731/project, with four work zones whose optimized lengths of 0.940, 0.550, 0.932, and 0.543 miles add up to 2.965 miles, and whose idling time is 4.76 hours, as shown in Table 6.3.

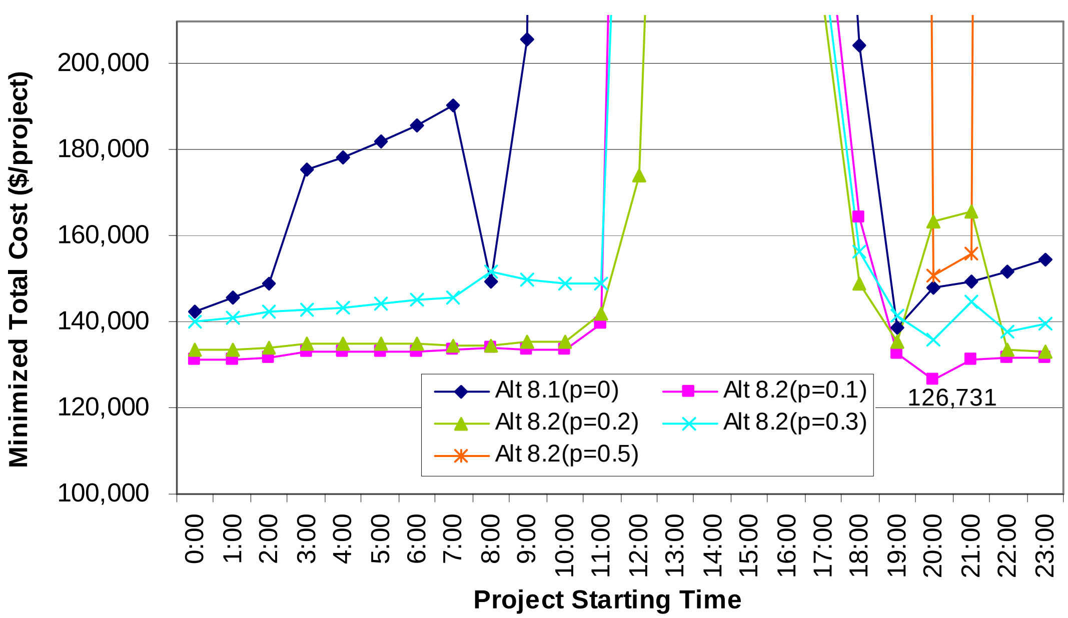
The optimized results in Figure 6.4 and Table 6.3 are based on k=0, which means there is no diverted flow from pQ_1 along the detour F $\rightarrow$ D $\rightarrow$ B. Figure 6.5 shows the optimized results when k increases from 0 to 1 (p=0.1 and project starting time=20:00). It indicates that a lower minimized total cost, $125,714/project, can be found when k=0.2, compared to $126,731/project when k=0. The optimized solution is shown in Table 6.4. Compared to Table 6.3, the user cost in Table 6.4 decreases by 4.6% and the maintenance cost is unchanged. The results show that flows appropriately diverted into multiple detour paths can yield lower total costs.

Table 6.2 AADT and Hourly Traffic Distributions on Main Road (IS-95) and Detour (US-1)

Hour	**Q_1 (vph) (IS-95 Northbound)**	**% of AADT**	**Q_3 (vph) (US-1 Northbound)**	**% of AADT**
0	1,362	1.44%	149	0.56%
1	1,149	1.22%	94	0.36%
2	897	0.95%	49	0.19%
3	917	0.97%	56	0.21%
4	1,160	1.23%	90	0.34%
5	2,098	2.22%	320	1.21%
6	3,670	3.89%	1,017	3.86%
7	5,143	5.45%	1,862	7.06%
8	5,388	5.71%	2,038	7.73%
9	4,283	4.54%	1,170	4.44%
10	4,403	4.66%	892	3.38%
11	4,904	5.19%	964	3.65%
12	4,982	5.28%	1,012	3.84%
13	5,095	5.40%	1,131	4.29%
14	5,519	5.84%	1,403	5.32%
15	7,167	7.59%	2,178	8.26%
16	7,336	7.77%	2,774	10.52%
17	7,214	7.64%	3,025	11.47%
18	7,089	7.51%	2,311	8.76%
19	5,172	5.48%	1,451	5.50%
20	3,307	3.50%	890	3.37%
21	2,406	2.55%	693	2.63%
22	2,158	2.29%	508	1.93%
23	1,619	1.71%	300	1.14%
AADT (One Direction)	**94,438**	**100.00%**	**26,377**	**100.00%**



(a)



(b)

Figure 6.4 Minimized Total Cost vs. Project Starting Time (IS-95, Eight-lane Freeway Work Zones, *k*=0) (a) Minimized Total Cost Scale: 100,000 – 2,500,000 (b) Minimized Total Cost Scale: 100,000 – 210,000

Table 6.3 Optimized Results for Case Study, Project Starting Time: 20:00, Alternative 8.2 (*p*=0.1, *k*=0)

Zone No.	Optimal length (miles)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.940	11.02	20.00	7.02	-	36,370
2	0.550	7.28	7.02	14.30	0.00	25,401
3	0.933	10.95	19.06	6.01	4.76	40,209
4	0.543	7.21	6.01	13.22	0.00	24,752
Total	2.965	36.46			4.76	126,731
Maintenance cost						103,045
Queuing delay cost						714
Moving delay cost						19,072
Idling cost						3,807
Accident Cost						94
Total cost						126,731
Total cost/project-mile ($/lane-mile)						42,742

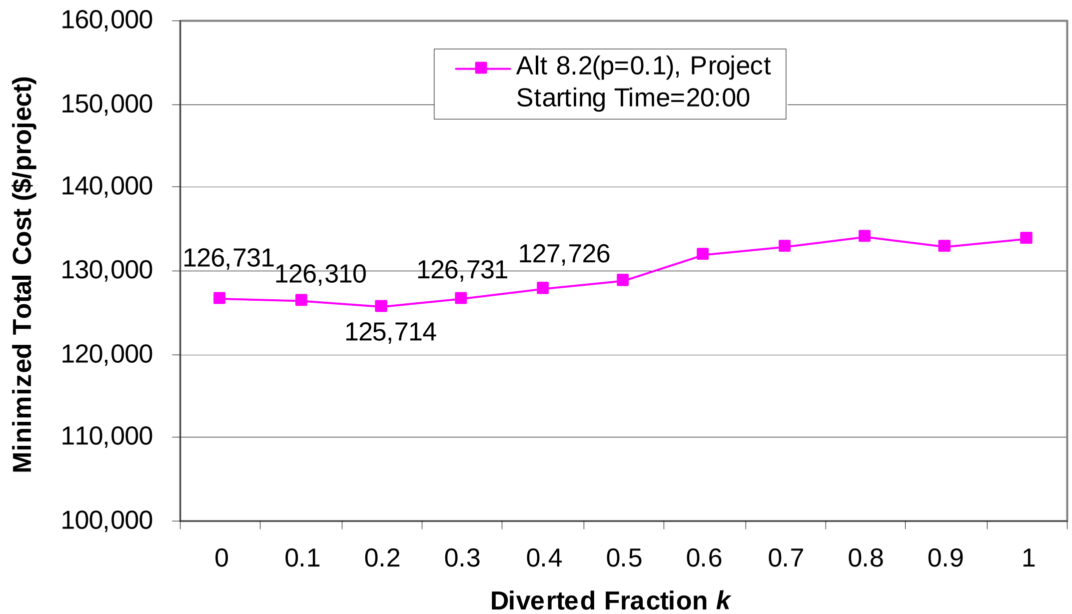


Figure 6.5 Minimized Total Cost vs. Diverted Fraction *k* (*p*=0.1, Project Starting Time: 20:00, IS-95, Eight-lane Freeway Work Zones)

Table 6.4 Optimized Results for Case Study, Project Starting Time: 20:00, Alternative 8.2 (*p*=0.1, *k*=0.2)

Zone No.	Optimized length (miles)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.932	10.95	20.00	6.95	-	35,868
2	0.562	7.40	6.95	14.35	0.00	25,661
3	0.935	10.98	19.03	6.01	4.69	40,081
4	0.535	7.14	6.01	13.15	0.00	24,104
Total	2.965	36.46			4.69	125,714
Maintenance cost						103,045
Queuing delay cost						746
Moving delay cost						18,085
Idling cost						3,749
Accident Cost						89
Total cost						125,714
Total cost/project-mile ($/lane-mile)						42,399

The optimized results in Table 6.4 are based on uniform zone alternatives and obtained with SAUAMD. If SAMAMD is applied, new optimized results are found, which yield lower total cost than in Table 6.4. Table 6.5 shows the optimized results obtained with SAMAMD. The results are almost the same as the solution shown in Table 6.4 but there is no diversion in the first zone and *k*=0 for all four zones. This indicates that diverting traffic to a longer alternate path is not necessary if mixed alternatives are considered. Thus, two different traffic management plans, namely uniform alternatives and mixed alternatives, result in different work zone optimization results. Such different management strategies should be carefully considered in project scheduling.

Table 6.5 Optimized Results for Case Study, Project Starting Time: 20:00, Mixed Alternatives

Zone No.	Optimized length (mile)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Prefered Zone Alt.	Prefered Diverted Fraction *p*	Prefered Diverted Fraction *k*	Total Cost ($/zone)
1	0.932	10.95	20.00	6.95	-	81	0.00	0.00	35,109
2	0.562	7.40	6.95	14.35	0.00	83	0.10	0.00	24,998
3	0.935	10.98	19.03	6.01	4.69	83	0.10	0.00	39,709
4	0.535	7.14	6.01	13.15	0.00	83	0.10	0.00	23,492
Total	2.965	36.46			4.69				123,308
Maintenance cost									103,045
Queuing delay cost									746
Moving delay cost									15,690
Idling cost									3,749
Accident Cost									78
Total cost									123,308
Total cost/project-mile ($/lane-mile)									41,588

6.4.2 Current Policy

Two current maintenance schedules for the 2.965-mile project are shown in Tables 6.6 and 6.7 in term of this single-lane closure policy. Table 6.6 shows the first policy whose project starting time is 9:00 a.m. The first zone ends at 3:00 p.m. Using the relation between zone length and duration, $D_i=z_3+z_4L_i$, the zone length can be obtained. Each zone is scheduled step by step until total zone lengths add up to 2.965 miles. The total cost computed with Eq.(6.20) is $242,153/project. The second policy shown in Table 6.7 starts at 19:00 p.m. The total cost is $199,994/project.

Table 6.6 Current Work Zone Policy for Case Study (*p*=0, *k*=0), Project Starting Time: 9:00

Zone No.	Zone length (miles)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.417	6.00	9.00	15.00	-	49,056
2	0.833	10.00	19.00	5.00	4.00	37,420
3	0.417	6.00	9.00	15.00	4.00	52,256
4	0.833	10.00	19.00	5.00	4.00	37,420
5	0.465	6.46	9.00	15.46	4.00	66,001
Total	2.965	38.46			16.00	242,153
Maintenance cost						104,345
Queuing delay cost						113,311
Moving delay cost						11,108
Idling cost						12,800
Accident Cost						589
Total cost						242,153
Total cost/project-mile ($/lane-mile)						81,671

Table 6.7 Current Work Zone Policy for Case Study (*p*=0, *k*=0), Project Starting Time: 19:00

Zone No.	Zone length (miles)	Duration (hr)	Starting time (0-23.99)	Ending time (0~23.99)	Idling time (hr)	Total Cost ($/zone)
1	0.833	10.00	19.00	5.00	-	34,220
2	0.417	6.00	9.00	15.00	4.00	52,256
3	0.833	10.00	19.00	5.00	4.00	37,420
4	0.417	6.00	9.00	15.00	4.00	52,256
5	0.465	6.46	19.00	1.46	4.00	23,842
Total	2.965	38.46			16.00	199,994
Maintenance cost						104,345
Queuing delay cost						72,376
Moving delay cost						10,084
Idling cost						12,800
Accident Cost						390
Total cost						199,994
Total cost/project-mile ($/lane-mile)						67,452

Compared to the current total costs in Tables 6.6 and 6.7, the optimized results obtained with SAMAMD in Table 6.5 can reduce total cost significantly. If the current project starting time is 9:00 a.m., the optimization model can reduce agency cost by 8.8%, user cost by 86.8%, and total cost by 49.1%; if the current project starting time is

19:00 a.m., the optimization model can reduce agency cost by 8.8%, user cost by 80.1%, and total cost by 38.3%. The comparison between the total costs of the current policy and optimized results obtained with SAMAMD is shown in Table 6.8. This comparison confirms that the SAMAMD algorithm developed in this study can very significantly reduce the agency cost, user cost, and total cost.

Table 6.8 Comparison Between Total Costs of Current Policy and Optimized Solution

	Current Policy	Optimized Solution	Reduction	% Reduced
Project Starting Time	9:00 a.m.	20:00 p.m.	-	-
Diversion	No Diversion	10% Diversion	-	-
Number of Zones	5	4	-	-
Work Duration (hr)	38.46	36.46	2	5.2%
Idling Time (hr)	16	4.69	11.31	70.7%
Total Duration (hr)	54.46	41.15	13.31	24.4%
Agency Cost ($/project)	117,145	106,794	10,351	8.8%
User Cost ($/project)	125,008	16,514	108,494	86.8%
Total Cost ($/project)	242,153	123,308	118,845	49.1%
Project Starting Time	19:00 a.m.	20:00 p.m.	-	-
Diversion	No Diversion	10% Diversion	-	-
Number of Zones	5	4	-	-
Work Duration (hr)	38.46	36.46	2	5.2%
Idling Time (hr)	16	4.69	11.31	70.7%
Total Duration (hr)	54.46	41.15	13.31	24.4%
Agency Cost ($/project)	117,145	106,794	10,351	8.8%
User Cost ($/project)	82,849	16,514	66,335	80.1%
Total Cost ($/project)	199,994	123,308	76,686	38.3%

* Agency cost includes maintenance cost and idling cost and user cost includes queuing delay cost, moving delay cost, and accident cost.

6.4.3 Simulation Results

When the parameters for incidents (work zones) are set, we found that CORSIM only allows users to specify the onset time of an incident (in seconds) at up to 9,999 seconds. (Time is measured from the start of the simulation in CORSIM). This indicates that two zones cannot be successive in a TRF file if the first zone duration exceeds 9,999 seconds. Due to this limitation of CORSIM, the work zone activities for a 2.965-mile project cannot be simulated by a TRF file. (All zone durations shown in Tables 6.5, 6.6, and 6.7 exceed 9,999 seconds.) Therefore each zone in Tables 6.5 to 6.7 has its TRF files. The TRF file for each zone is modified by adding work zone length and location (for both current policy and optimized solution) and changing turn-movement percentages (for optimized solution). The original TRF file without a work zone is necessary because the net delay due to a work zone is the difference between the simulated delay with and without that work zone. The simulation results for this IS-95 case study, including the current policies (starting at 19:00) and optimized solution with mixed alternatives, are shown in Table 6.9.

Table 6.9(a) Simulation (Simplified Network[1]) and Optimization Results of Current Policies and Optimized Solution

Zone	Work Zone Duration	Simulation Duration	Simulation Delay with Work Zone (veh-hr)	Simulation Delay without Work Zone (veh-hr)	Net Delay due to Work Zone (veh-hr)	Delay by Optimization Model [2] (veh-hr)
Current Policy (Project Starting Time: 19:00)						
1	19.00-5.00	19.00-9.00	243	158	85	-
2	9.00-15.00	9.00-19.00	7,326	5,239	2,087	-
3	19.00-5.00	19.00-9.00	941	158	784	-
4	9.00-15.00	9.00-19.00	8,643	5,239	3,404	-
5	19.00-1.46	19.00-9.00	998	158	841	-
Total	-	-			7,201	6,904
Optimized Solution (Project Starting Time: 20:00)						
1	20.00-6.95	19.00-9.00	164	158	6	-
2	6.95-14.35	6.00-19.00	5,828	5,428	401	-
3	19.03-6.01	19.00-9.00	430	158	272	-
4	6.01-13.15	6.00-19.00	6,202	5,428	774	-
Total	-	-			1,453	1,376

1. Simplified Network is developed in this study and has the same configuration as the Figure 6.1(c) The traffic volumes and link lengths are applied from Tables 6.1 and 6.2.
2. Delays by Optimization Model are obtained from the user costs in Table 6.8 divided by the value of user time *v* (baseline = $12/hr).

Table 6.9(b) Simulation (Complete Network[1]) and Optimization Results of Current Policies and Optimized Solution

Zone	Work Zone Duration	Simulation Duration	Simulation Delay with Work Zone	Simulation Delay without Work Zone	Net Delay due to Work Zone
Current Policy (Project Starting Time: 19:00)					
1	19.00-5.00	19.00-9.00	7,188	4,906	2,283
2	9.00-15.00	9.00-19.00	27,204	14,898	12,306
3	19.00-5.00	19.00-9.00	6,282	4,906	1,376
4	9.00-15.00	9.00-19.00	26,688	14,898	11,790
5	19.00-1.46	19.00-9.00	5,046	4,906	140
Total	**-**	**-**	**-**	**-**	**27,895**
Optimized Solution (Project Starting Time: 20:00)					
1	20.00-6.95	19.00-9.00	5,066	4,906	160
2	6.95-14.35	6.00-19.00	40,727	23,943	16,784
3	19.03-6.01	19.00-9.00	5,207	4,906	301
4	6.01-13.15	6.00-19.00	25,832	23,943	1,889
Total	**-**	**-**	**-**	**-**	**19,135**

1. Complete Network is provided by the Maryland State Highway Administration. The traffic volumes and link lengths are applied from Tables 6.1 and 6.2.

Table 6.10 Comparison of the Results of Optimization Model and Simulation Model

	Current Policy	Optimized Solution	Delay (or Cost) Reduction	% Reduction
Project Starting Time	19:00	20:00	-	-
Optimization Model				
Agency Cost ($/project)	117,145	106,794	10,351	8%
Delay by Analytical Model (veh-hr)	6,904	1,376	5,528	80%
Simulation Model				
Delay by Simulation (veh-hr) (Simplified Network)	7,201	1,453	5,748	80%
Delay by Simulation (veh-hr) (Complete Network)	27,895	19,135	8,760	31%

The overall net simulated work zone delay of the optimized results decreases by 80% (simplified network) and 31% (complete network) compared to the current policy starting at 19:00. A comparison of the results of optimization and simulation models indicates that they are consistent, as shown in Table 6.10. The optimization models do significantly reduce total cost, including user cost and maintenance cost, compared to the total cost of the current policy in Maryland.

Chapter VII Conclusions and Recommendations

7.1 Summary

Work zone optimization problems have been solved with analytical methods for steady traffic inflows and heuristic Simulated Annealing algorithms for time-dependent inflows and multiple detour paths. In Chapter 3, four alternatives for two-lane highway work zones and four alternatives for four-lane highway work zones are developed and optimized analytically. The objective of work zone optimization is to minimize the total cost, including agency cost and user cost by optimizing work zone lengths for each alternative and finding optimal diversion fraction. Guidelines for selecting the best alternative for different characteristics of traffic flows, road and maintenance processes are developed by deriving thresholds among alternatives. In Chapter 4, the models for two-lane highway and four-lane highway work zones for time-dependent inflows are developed. Two optimization methods, Powell's and Simulated Annealing, are adapted for this problem and compared. In numerical tests, the Simulated Annealing algorithm yields better solutions using less computer time than Powell's Method. The reliability of Simulated Annealing algorithm is also assessed.

In Chapter 5, optimization models are developed for four work zone alternatives on two-lane highways and four alternatives on four-lane highways, all with time-dependent inflows. The SAUASD (Simulated Annealing for Uniform Alternatives with a Single Detour) algorithm is developed for alternative selection. Moreover, the SAMASD (Simulated Annealing for Mixed Alternatives with a Single Detour) algorithm is developed to search through mixed alternatives and to optimize diverted fractions in order to find lower total cost than for uniform alternatives.

In Chapter 6, work zone optimization models for a road network with multiple detour paths and the SAMAMD (Simulated Annealing for Mixed Alternatives with Multiple Detour paths) algorithms are developed. Both analytical and simulation models are developed to estimate delay cost and total cost. For analyzing traffic diversion through multiple detour paths in a road network, a Simulated Annealing algorithm combined with an assignment method is used to optimize work zone lengths and schedule the resurfacing work. Simulation analyses based on CORSIM are not only used to estimate delay cost, but also to evaluate the effectiveness of optimization models. In the IS-95 case study, a comparison of the results from optimization and simulation models indicates that they are consistent.

7.2 Conclusions

The conclusions from the numerical results, threshold analysis, and case study may be summarized as follows.

7.2.1 Work Zone Optimization for Steady Traffic Inflows

Two-lane highway work zones

1. When optimized in Section 3.6.1, Alternative 2.1 has higher user costs and shorter zones while Alternative 2.4 has lower user costs and longer zones than other alternatives in the baseline conditions.
2. Based on the threshold analysis presented in Section 3.6.2, Alternative 2.4 is preferred alternative in the baseline conditions. As detour length increases beyond its threshold (10 km), Alternatives 2.1, 2.2, and 2.3 eventually become preferable.

3. Considering an optimized diverted fraction among Alternatives 2.1, 2.2, and 2.3 in Section 3.6.3, full diversion is preferable if the detour is short; partial or no division becomes preferable as detour length increases.

Four-lane highway work zones

1. Section 3.7.2 shows that traffic flows affect the rankings of alternatives.
2. In the threshold analysis, Alternative 4.3 is preferred when Q_1 does not exceed the first flow threshold, 800 vph. As Q_1 increases beyond its threshold, Alternatives 4.1 or 4.2 becomes preferable.
3. Considering the optimized diverted fractions among Alternatives 4.1, 4.2, and 4.3 under the baseline conditions in Section 3.7.3, full diversion (p=1) is preferable if Q_1 is lower than 800 vph; no diversion (p=0) is preferable if Q_1 is between 800 vph and the work zone capacity of 1200 vph; for higher Q_1, the total cost is minimized if any vehicles beyond 1200 vph from Q_1 are detoured (the detour length is the baseline value 6 km).

7.2.2 Work Zone Optimization for Time-Dependent Inflows

Two-lane highway work zones

1. Considering 24 hourly project starting times over a day in Section 4.3, the total cost comparison demonstrates that the SAUA (Simulated Annealing with Uniform Alternatives) algorithm yields better results (18 of 24) in less time than Powell's Method.

2. The optimized work zone lengths and schedules are sensitive to input parameters such as the average cost of idling time, work zone setup cost and its duration.
3. Maintenance plans with or without pauses can be optimized with the proposed methods, preferably with the SAUA.
4. In Section 4.5, to test the reliability of the SAUA, 50 replications of the cost minimization are performed with 50 different groups of random numbers. Given the small relative variance of the 50 replications of minimized total costs, we are quite unlikely to find a value much below the mean. Thus, the statistical analysis and numerical examples indicate that Simulated Annealing is very likely to find solutions that are very close in value to the global optimum.

Four-lane highway work zones

1. Considering 24 hourly project starting times over a day in Section 4.4, the total cost comparison also demonstrates that the SAUA algorithm yields better results (17 of 24) in less time than Powell's Method.
2. The optimized work zone lengths and schedules are not sensitive to the average cost of idling time v_d because queuing delay in the baseline condition will be cumulative during peak periods and even increasing the average cost of idling time v_d can not compensate the high queuing delay costs for four-lane highway work zone so that the pauses are mandatory during peak periods.

7.2.3 Work Zone Optimization with a Detour

1. Based on the threshold analysis in Section 5.3, Alternative 2.3 is preferred in the baseline condition. As detour AADT increases beyond its threshold, 25,000 vehicles per day, Alternatives 2.2 (p=0.3) and 2.1 become preferable.
2. In Section 5.4 for four-lane highway work zones no detour AADT threshold is found in this study because higher detour AADT and higher diverted flow increase the queuing delay on detour quickly so that no threshold with Alternative 4.1 occurs.
3. In Chapter 3, without considering detour capacity, alternatives with high diverted fraction may be preferable for four-lane highway work zones. However, those alternatives never become preferable when considering detour queuing delay and time-dependent inflows in Chapter 5, due to faster increases in detour queuing delay.
4. From the numerical example for mixed alternatives in Section 5.5, partial diversion or no diversion is appropriate during daytime; full diversion is usually preferable during nighttime due to faster return to the main road than during daytime.
5. When detour AADT is higher, mixed alternatives that combine no diversion, partial diversion, or full diversion, can yield lower minimized total cost than uniform alternatives. An appropriate traffic management plan should be developed based on different traffic demands.

7.2.4 Work Zone Optimization for Multiple Detour Paths

1. Appropriately diverted flows into multiple detour paths can yield lower total costs than single detours.
2. For the IS-95 case study in Section 6.4.2, the optimized solution can reduce total cost very significantly below the current policy. If the current project starting time is 9:00 a.m., the SAMAMD can reduce agency cost by 8.8%, user cost by 86.8%, and total cost by 49.1%. If the current project starting time is 19:00 a.m., the SAMAMD can reduce agency cost by 8.8%, user cost by 80.1%, and total cost by 38.3%.
3. The overall net simulated work zone delay of the optimized results decreases by 80% (simplified network) and 31% (complete network) compared to the current policy starting at 19:00. A comparison of the results from optimization and simulation models indicates that they are consistent. The optimization models do significantly reduce total cost, including user cost and maintenance cost compared to the total cost of the current work zone policy in Maryland.

7.3 Recommendations for Future Research

Although this study has developed satisfactory methods for optimizing work zone scheduling problem, possible extensions of the analysis and models developed in this study are desirable and suggested as follows:

1. Speeded-up Maintenance Work

In this study, average maintenance cost z_2 and average maintenance time z_4 are constants. However, highway agencies may be able to speed up maintenance work by accepting higher cost, i.e. for more equipment and crews, to reduce the maintenance duration. Models considering the relations between maintenance cost and duration are desirable.

2. Two-lane Highway Models for Demand That Exceeds Capacity

Optimization models for two-lane highway work zone in this study are suitable when hourly demands in both directions do not exceed work zone capacity. Future extensions of the present work might consider work zone optimization for two-lane highways where two-way demand may temporarily exceed one-lane capacity during some periods.

3. Comparison of System Optimization and User Equilibrium

The models in this study are based on system optimization, which minimizes the total costs, including highway agency cost and user cost; however, in a multiple detour network, we may expect "user equilibrium" assignment to reflect user decision, based on available information, regardless of the pre-planned traffic control decision. Therefore, comparisons of "system optimization" and "user equilibrium" results should be made in future research.

4. Safety Effects of Different Alternatives

Safety cost is included in user cost for this study and it is derived based on queuing delay and moving costs. However, work zone configurations for different alternatives might have different safety influences. Further research on safety cost estimation and safety improvements is desirable.

5. Work Zone Cost and Duration Parameters

We assume the fixed setup cost and its duration are the same for all alternatives in this study. However, work zone configurations for different alternatives may vary and these cost and duration parameters should be surveyed for different alternatives. Further research on applying different cost and duration parameters for various alternatives is desirable.

6. Transfer Cost for Mixed Alternatives

The SAMASD algorithm in Section 5.2.2 and the SAMAMD algorithm in Section 6.2.3 for mixed alternatives are assumed that there is no transfer cost from one zone alternative to the other zone alternative. However, there may well be transfer costs when zone alternatives are changed. Future research for searching different alternatives for each zone within a project may consider transfer cost as well as different cost and duration parameters.

7. Development of Simulation-based Methods for Optimizing Flows through Complex Networks

Simulation in this study is applied to evaluate the effectiveness of work zone optimization models. However, simulation might also be used to evaluate the objective functions of the work zone optimization models in optimizing flows (or diverted fractions) through complex networks, as well as work zone scheduling. However, such optimization through simulation may impose severe computation burdens. Further research on work zone optimization through simulation is desirable.

8. Consideration of Work Zone Constraints

The optimization model in this study could be further developed to consider some highway agencies's constraints, e.g. on queue length, number of lane closed at various times, and maximum diverted fractions.

9. Time-Dependent Diversion Fraction

In the current models the diversion fractions stay constant while one zone is resurfaced. However, diversion fractions which vary with time-dependent inflows may be considered for dynamic traffic control. Further research on time-depedent diversion fractions is desirable.

Appendix A Variable List

The following variables are used in this dissertation (with units in parentheses):

A_i = Alternative for zone i, A_i= $j1$, $j2$, $j3$,and $j4$, $i=1, \ldots, m$, $j=2, 4, 6, 8$;

$A_{opt,i}$ = final optimized Alternative for zone i, $A_{opt,i}$=$j1$, $j2$, $j3$,and $j4$, $i=1, \ldots, m$, $j=2, 4, 6, 8$;

C_a^{2i} = accident cost per lane-kilometer for Alternative 2.i, $i=1,2,3,4$ (\$/lane·km);

C_a^{4i} = accident cost per lane-kilometer for Alternative 4.i, $i=1,2,3,4$ (\$/lane·km);

C_{ai}^{2j} = accident cost per lane-zone for work zone i for Alternative 2.j, $j=1,2,3,4$ (\$/lane-zone);

C_{ai}^{4j} = accident cost per lane-zone for work zone i for Alternative 4.j, $j=1,2,3,4$ (\$/lane-zone);

$C_{I\,I}$ = idle cost for zone i (\$/zone);

c_l = maximum lane discharge rate without a work zone for multiple- highways (vph/lane);

C_M = maintenance cost per lane-kilometer (\$/lane·km);

C_{mi} = maintenance cost per lane-zone for work zone i (\$/lane·zone);

c_o = maximum discharge rate without work zone for four-lane two-way highways (vph); baseline = 2,600 vph;

C_{PT} = total cost per lane for a maintenance project (\$/project);

C_{PT}^{2i} = total cost per lane for a maintenance project for Alternative 2.i, $i=1,2,3,4$ (\$/project);

C_{PT}^{4i} = total cost per lane for a maintenance project for Alternative 4.i, $i=1,2,3,4$ (\$/project);

C_q^{2i} = queuing delay cost per lane-kilometer for Alternative 2.*i*, *i=1,2,3,4*

C_q^{4i} = queuing delay cost per lane-kilometer for Alternative 4.*i*, *i=1,2,3,4*

C_{qi}^{2j} = queuing delay cost per lane-zone for work zone *i* for Alternative 2.*j*, *j=1,2,3,4* ($/lane-zone);

C_{qi}^{4j} = queuing delay cost per lane-zone for work zone *i* for Alternative 4.*j*, *j=1,2,3,4* ($/lane-zone);

C_S = supplier cost;

C_T = total cost per lane-kilometer ($/lane·km);

C_T^{*2i} = minimized total cost of Alternative 2.*i* for optimized work zone length L^{*2i}, *i=1,2,3,4* ($/lane·km);

C_T^{*4i} = minimized total cost of Alternative 4.*i* for optimized work zone length L^{*4i}, *i=1,2,3,4* ($/lane·km);

C_{ti} = total cost per lane-zone for work zone *i* ($/lane·zone);

$\square$ = total cost per lane-zone for work zone *i* for Alternative 2.*j*, *j=1,2,3,4* ($/lane-zone);

$\square$ = total cost per lane-zone for work zone *i* for Alternative 4.*j*, *j=1,2,3,4* ($/lane-zone);

C_U = user delay cost per lane-kilometer ($/lane·km);

$\square$ = user delay cost per lane-kilometer for Alternative 2.*i*, *i=1,2,3,4* ($/lane·km);

$\square$ = user delay cost per lane-kilometer for Alternative 4.*i*, *i=1,2,3,4* ($/lane·km);

C_{ui} = user delay cost per lane-zone for work zone *i* ($/lane·zone);

$\square$ = moving delay cost per lane-kilometer for Alternative 2.*i*, *i=1,2,3,4*

$\square$ = moving delay cost per lane-kilometer for Alternative 4.*i*, *i=1,2,3,4*

$\square$ = moving delay cost per lane-zone for work zone i for Alternative 2.j, $j=1,2,3,4$ (\$/lane-zone);

$\square$ = moving delay cost per lane-zone for work zone i for Alternative 4.j, $j=1,2,3,4$ (\$/lane-zone);

c_w = maximum discharge rate along work zone (vph); baseline = 1,200 vph;

c_{wrl} = maximum lane discharge rate along a work zone for multiple- highways (vph/lane);

D = total maintenance duration for work zone length L (h);

D_i = maintenance duration for work zone i (h);

D_{ij} = period j of maintenance duration for work zone i, j=1,2, .., n (h);

$D_{s,l}$ = duration of Stage l (hr);

ΔD = duration unit for increasing or decreasing a unit length, $\Delta D=\Delta L*z_4$;

ΔD_r = duration difference between new $t_{e,\,i}$ and old $t_{s,\,i+1}$ when new $t_{e,\,i}$ exceeds old $t_{s,\,i+1}$;

d = average maintenance time (hr/lane·km);

H = average headway through work zone area (s); baseline = 3 s;

J_{max} = number of iterations for reducing temperature from T_0 to T_f;

Kj = jam density along AB and detour (veh/lane·km); baseline = 200 veh/lane·km;

K_{max} = maximum number of iterations for temperature T_j to equilibrium;

k_i = diverted fraction of pQ_1 along F $\rightarrow$ D $\rightarrow$ B for zone i, k_i = 0 - 1, $i=1$, …., m;

$k_{opt,i}$ = final optimal diverted fraction of pQ_1 along F $\rightarrow$ D $\rightarrow$ B for zone i, $k_{opt,i}$ = 0 - 1, $i=1$, …., m;

L = work zone length (km);

L^{*2i} = optimized work zone length of Alternative 2.i, $i=1,2,3,4$ (km);

L^{*4i} = optimized work zone length of Alternative 4.i, i=1,2,3,4 (km);

L_1 = distance from A to work zone start point (km);

L_2 = distance from work zone end point to B (km);

L_{assign} = deleted last zone length divided by $m-1$, which is averagely assigned to the previous $m-1$ zones;

L_{avg} = average zone length in current solution;

L_d = $L_{d1}+L_{d2}+L_{d3}$, detour length (km);

L_{d1} = length of first detour segment (km); baseline = 0.5 km;

L_{d2} = length of second detour segment (km); baseline = 5 km;

L_{d3} = length of third detour segment (km); baseline = 0.5 km;

L_{min} = minimum zone length in current solution;

L_T = road length of a maintained project (km);

L_t = $L+L_1+L_2$; length from A to B (km); baseline = 5 km;

ΔL = length unit for increasing or decreasing, baseline=0.01km;

ΔL_r = length difference between length unit and the remaining length of the deleted zone;

m = number of work zones of a maintained project;

N = number of cycles per maintained kilometer (cycles/kilometer);

N_i = number of cycles for work zone i (cycles/zone);

N_{ij} = number of cycles per varying traffic flow period D_{ij} (cycles);

N_{limit} = maximum number of iterations for temperature T_j in which the total cost is successfully reduced to equilibrium;

N_{succ} = cumulative number of iterations for temperature T_j in which the total cost is successfully reduced to equilibrium;

$N_{r,succ}$ = cumulative number of iterations for temperature T_j in which the total cost is successfully reduced for generating neighboring solution repeatedly using the same random numbers;

n_a = number of accidents per 100 million vehicle hour (acc/100mvh); baseline = 40 acc/100mvh

n_l = number of lanes in Direction 1;

n_{rl} = number of the remaining lanes along a work zone;

p = diverted fraction of flow in Direction 1 to alternative route;

p_i = diverted fraction for zone i, $p_i = 0 - 1$, $i=1, \ldots, m$;

$p_{opt,i}$ = final optimized diverted fraction for zone i, $p_{opt,i} = 0 - 1$, $i=1, \ldots, m$.

Q_i = hourly flow rate in Direction i (vph);

= traffic flow of Direction 1 during the period j for work zone i (vph);

= traffic flow of Direction 2 during the period j for work zone i (vph);

Ds_r = duration of Stage r for time-dependent inflows (h);

T_{sr} = starting time of Stage r for time-dependent inflows;

T_{er} = ending time of Stage r for time-dependent inflows;

T_0 = initial temperature in SA algorithm;

T_f = final temperature in SA algorithm;

t_i = discharge phase for servicing the traffic flow in Direction i (second);

= discharge phase for servicing the traffic flow in Direction 1 (second);

= discharge phase for servicing the traffic flow in Direction 2 (second);

Δt_i = idle time between zone i and zone $i-1$;

$t_{s,i}$ = starting time for work zone i;

$t_{e,i}$ = ending time for work zone i;

V = average work zone speed for two-lane highways (km/hr); baseline = 50 km/hr;

V_f = free flow speed along AB and detour (km/h); baseline = 80 km/hr;

V_d = detour speed affected by Q_1 (km/hr);

$\square$ = detour speed affected by pQ_1 for Alternative 2.2 or 4.2 (km/hr);

$\square$ = detour speed affected by Q_1 for Alternative 2.3 or 4.3 (km/hr);

V_{d0} = original speed on L_{d2} unaffected by Q_1 (km/hr);

V_0 = free flow speed on original road without work zone (km/hr);

V_w = average work zone speed for four-lane highways (km/hr); baseline = 50 km/hr;

v = value of user time (\$/veh·hr); baseline = 12 \$/veh·hr;

v_a = average accident cost (\$/accident); baseline = 142,000 \$/accident;

v_d = average cost of idling time per hour for crews and equipments (\$/hr); baseline=800 \$/hr;

Y = total delay per cycle in both directions (veh·hr);

Y_i = delay per cycle in Direction i (veh·hr);

z_1 = fixed setup cost (\$/zone); baseline = 1,000 \$/zone for all alternatives;

z_2 = average maintenance cost per additional lane·kilometer (\$/lane·km); baseline = 80,000 \$/lane·km;

z_3 = fixed setup time (hr/zone); baseline = 2 hr/zone for all alternatives

z_4 = average maintenance time per lane·kilometer (hr/lane·km); baseline = 6

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