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STATE HIGHWAY ADMINISTRATION

RESEARCH REPORT

ENHANCEMENT OF FREEWAY INCIDENT TRAFFIC MANAGEMENT AND RESULTING BENEFITS

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16. Abstract

To improve traffic conditions on major highways plagued by non-recurrent congestion, most highway agencies have invested their resources in two principal operational programs: incident response and clearance, and traffic impact management. However, even with the wide-spread implementation of such programs, effectively minimizing the traffic impact caused by multi-lane blocked incidents remains a critical and challenging issue for most highway agencies. This research developed a multi-criteria decision-support system for determining the necessity of detour operations during incident management from an overall socio-economic benefit perspective. The developed system enables responsible agencies to consider all associated critical factors with preferred weights, including the direct benefits and operational costs, safety and reliability, accessibility of detour, and acceptability by travelers. This research is part of our developed integrated incident managing system for SHA that has various essential functions, ranging from prediction of incident duration to estimation of operational benefits. This decision module, based on the AHP (***a***nalytical ***h***ierarchical ***p***rocess) methodology, features its computing efficiency and operational flexibility, allowing users to make necessary revisions if more data are available or more criteria need to be included.

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CHAPTER 1: Introduction

1.1 Research Background

Although the incident response program, named Coordinated Highways Action Response Team (CHART) by the Maryland State Highway Administration (SHA), has been well recognized as one of the most efficient incident response programs, the main priority of CHART operators in response to detected incidents has been managing traffic at the incident scene and assisting drivers. However, CHART still faces a challenge on how to effectively streamline the traffic management actions for the highway network impacted by a major accident that demands the activation of a Freeway Incident Traffic Management (FITM) plan.

Conceivably, efficiently detouring vehicles during the response operations of major accidents to minimize the formation of traffic queues is a complex task, as it necessitates good coordination among agencies responsible for various critical activities. These include estimating the incident duration and its impact boundaries, identifying the available detour routes, deciding where and what to display on dynamic message signs (DMS), and determining how to accommodate the detoured traffic with responsive signal settings. Hence, the FITM plan with the best planning and proper execution may yield substantial benefits to both the network drivers and the entire society. The impressive annual benefits reported by CHART, however, do not include the contribution from the activation of the detour traffic management plans during major incidents.

In view of the diminishing resources and the pressing needs to minimize the impacts of highway accidents, a priority task for SHA to further improve its operational effectiveness is to identify potential areas for CHART to enhance its traffic incident management, especially during implementation of the FITM plan. Such a task includes: (1) analyzing the spatial distribution and nature of recent incidents that required SHA to implement various FITM plans; (2) understanding the interrelationships between the duration of detected incidents, the congestion level, and SHA's decision to trigger a FITM plan; and (3) estimating the costs and benefits associated with some of SHA's past FITM operations. Due to the additional efforts and costs involved in detouring traffic, it is also imperative to have a reliable decision-support tool that can assist SHA engineers in making a reliable and timely decision in a real-time operational environment, based on the detected incident severity, estimated clearance duration, traffic conditions on available detour routes, estimated traffic flow speeds on the primary and detour routes with and without implementing the FITM plan, and the resulting costs and benefits for various traffic management strategies.

1.2 Research Objectives

This research, proposed in response to the need to enhance the efficiency of CHART's FITM operations, has two primary objectives: (1) understand the nature of those incidents that triggered implementation of the FITM plan over the past five years; and (2) develop a decision-support system that enables traffic engineers to determine whether any detour operation can be justified. The decision support system can also serve as an evaluation tool for responsible staff to review the past performance of FITM operations and reset some decision thresholds based on available resources and personnel constraints.

Note that the byproduct of such a decision-support tool includes the estimated travel times on the primary and detour routes and the resulting benefits such as reduction in delay, fuel consumption, and emissions. Thus, with some additional traffic information such as volumes on the primary and detour routes during the incident clearance period, CHART can further estimate the operational benefits produced from annual FITM operations and include these potentially large benefits in the annual CHART performance evaluation report.

1.3 Report Organization

Based on the above objectives, the research results are organized into six chapters. A brief description of each chapter follows:

Chapter 2 reviews the literature concerning incident response and management, dividing all related studies into three categories: incident response strategies, incident duration estimation, and detour decision-support systems. Section 2-2 presents those location coverage models for best allocating traffic response units and housing available resources for emergency service needs. Section 2-3 summarizes state-of-the-art studies on predicting the duration of a detected incident, the most critical information for estimating the resulting traffic impact boundaries. Section 2-4 briefly describes state of the practices on detour decision support systems, including their decision criteria, data needs, operational procedures, and potential areas for enhancement. Results of the literature review serve as the basis for developing a multi-criteria decision-support tool that allows responsible agencies to make a proper detour decision based on all associated factors.

Chapter 3 provides an in-depth review of SHA's FITM program and identifies potential areas for its further enhancement. Section 3-2 compares the criteria used by SHA's FITM program with those employed by other highway agencies, highlighting the need for an effective decision-support system for detour operations. Section 3-3 reports the analysis results of those incidents triggering the FITM operations (called FITM-incidents) between 2007 and 2010, including the distributions by location, incident nature, number of blocked lanes, truck involvement, clearance duration, different times of the day, and different days of the week.

Chapter 4 introduces a freeway corridor control model designed specifically to optimize the detour operations during non-recurrent congestion, including the optimal time-varying detour rate, the activation and terminating times for detour operations, and the signal time for each intersection in the detour route. Section 4-2 presents the formulations for modeling the complex interactions between freeway and surface traffic flows under various detour control strategies, considering the potential mutual lane blockage at critical intersections due to the detouring traffic. Section 4-3 discusses methods to specify the objective function of the optimal corridor model so that its implementation can minimize the total congestion and balance traffic conditions on the primary and detour routes. Section 4-4 details the solution algorithm designed specifically to enable traffic engineers to operate the developed model in real time during detour operations. Section 4-5 illustrates the case study results with the optimal corridor control model, highlighting the potential operational issues and the resulting benefits.

Chapter 5 presents a multi-criteria decision support system for detour operations, including estimation of incident duration, projection of the traffic queue length, comparison between the traffic flow speeds with and without detouring traffic, and a cost-benefit analysis for implementing the FITM plan. Section 5-2 illustrates the structure of an effective system for real-time incident response and traffic management system, focusing on the operational procedures and criteria to evaluate the need for detour operations. Section 5-3 analyzes the system application with incident scenarios calibrated from field data, including the sensitivity of various key parameters on the decision outcome, and prioritizing embedded criteria based on available resources and public concerns.

Chapter 6 first summarizes the research findings of this study, highlighting the deficiencies of existing practices in contending with non-recurrent congestion and potential areas for enhancement. This section is followed by recommendations for SHA to advance its FITM operations in view of the increasing frequency of incidents and diminishing resources.

CHAPTER 2: Literature Review

2.1 Introduction

This chapter summarizes the major studies concerning freeway incident traffic management over the past decades, focusing on critical issues, existing approaches, and potential research directions. This chapter presents and divides the review results into the following categories:

- Incident response strategies: focusing on how best to use the available resources in response to detected and potential incidents over the service area during a target time period;
- Incident duration estimation: highlighting the data issues and major stream of methodologies to reliably estimate the duration of a detected incident; and
- Detour decision support system: evaluating existing criteria, strategies, or procedures used to determine whether or not detour operations are necessary.

The remaining sections present a brief summary of existing studies related to each category in sequence.

2.2 Incident Response Strategies

A large body of traffic studies has pointed out the critical role of efficient response to the total delay incurred by incidents and concluded that an increase in incident response time may contribute to the likelihood of having secondary incidents (Bentham, 1986; Brodsky and Hakkert, 1983; Mueller et al., 1988). The study results by Sanchez-Mangas et al. (2009) shows that a reduction of 10 minutes in emergency response time could result in 33 percent less probability of incurring vehicle collision and fatalities. Most studies conclude that dispatching emergency services units and clearing the incident scenes in a timely manner are the key tasks for minimizing incident impact (Kepaptsoglou et al., 2011: Huang and Fan, 2011).

In improving the efficiency of emergency incident responses, both availability and accessibility of service units play essential roles. The availability of response units can differ depending on the relationship between emergency response resources and the likely distribution of incidents. Accessibility is usually measured in terms of transportation costs (e.g., travel time, travel distance, etc.) between dispatching sites and incident locations. For that reason, two vital questions arise in planning and managing emergency services: how many response units are needed, and where should they be allocated in response to the temporal and spatial distribution of incidents? The core methodology for dealing with this issue belongs to the category of facility location assignment.

The core issue of facility location problem was to locate a single warehouse from all candidate sites (Weber, 1929). Similar models have also been developed and applied in a variety of fields, including healthcare facilities, plants and warehouses, post offices, and landfills (Eiselt, 2007; Owen and Daskin, 1998).

Two main issues associated with facility location studies and the emergency incident response are: (1) allocating emergency service units for recurrent emergency events and (2) planning the location such as the response centers to house the resources for emergency services and incident management. Typically, the factors considered when designing the location and distribution of emergency service resources include the total assets, operational costs, incident demand coverage, and incident response timeliness. The next three sections summarize three categories of studies that can be used to optimize incident response efficiency: covering models, *P*-median models, and *P*-center models.

2.2.1 Covering models

Covering models, the most widely used approach for allocating the sites of emergency service units, attempt to provide "coverage" to all demand points, which are considered covered only if a response unit is available to provide services to the demand points within a distance limit. The literature describes two major schools of methods: the location set covering problem (LSCP) and the maximal covering location problem (MCLP).

The LSCP is an earlier version of the emergency facility location model by Toregas et al. (1971); it seeks to minimize the required number of facility locations that cover all demand points. To overcome the deficiencies of the LSCP, several researchers (Church and ReVelle, 1974; White and Case, 1974; Schilling et al., 1979) developed the MCLP model. That model aims to maximize the coverage of demands subjected resource constraints and the minimal service standards so that it does not require covering all demand points. The MCLP and its variants have been broadly applied to various emergency service problems. Such a study, by Eaton et al. (1985), involved planning the location of emergency response vehicles in Texas. When implemented, this plan actually decreased the average emergency response time.

The covering methodology for locating emergency services has also been extended to consider the stochastic nature of emergency events. One approach that reflects the complexity and uncertainty of the response allocation issue uses chance-constrained models (Chapman and White, 1974) that can guarantee a certain level of service reliability. For instance, Daskin (1983) estimated the probability that at least one server is available to serve the request from any demand and formulated the maximum expected covering location problem (MEXCLP) to position *P* facilities in order to maximize the average of demand coverage. MEXCLP was enhanced later by ReVelle and Hogan (1986). Their proposed model, the probabilistic location set covering problem (PLSCP), uses an average server busy faction (q_i) and a service reliability factor (a) for demand points and then places the facilities to maximize the probability of service units being free to serve within a particular distance. MEXCLP and PLSCP have been further modified and improved for other EMS location problems by many researchers, and modeling details of their studies are available in the literature (ReVelle and Hogan, 1989a; Bianchi and Church,1988; Batta *et al.*,1989; Goldberg *et al.,* 1990; and Repede and Bernardo, 1994).

Another approach taken to tackle the stochastic properties of the emergency service location issue uses the scenario planning methodology to handle multiple possibilities of a random event by estimating possible parameters that may vary over different emergency scenarios. In practice, responsible agencies may evaluate each scenario individually and then aggregate all strategies to develop scenario-specific solutions based on mostly engineering judgments. For example, MCLP was extended by Schilling (1982) to incorporate scenarios, aiming to maximize the demand coverage over all considered scenarios. Schilling used individual scenarios to discover a range of good location decisions and then determined the final locations design common to all scenarios based on a compromise decision. Although such an approach is conceptually and computationally simple, it may not yield reliable results. Thus, Serra and Marianov (1999) developed a stochastic approach to represent the uncertainty of target parameters. Some other stochastic methods reported in the literature include stochastic programming (SP) and robust optimization (RO). In general, SP focuses on the expectation of performance measures so that it relies on the complete probability distribution of random parameters, and thus having less consideration for the risk (Birge and Louveaux, 1997). In contrast, RO places more emphasis on the worst-case scenario, which tends to yield more conservative results.

2.2.2 *P*-median models

Another key method for evaluating the effectiveness of strategies for allocating emergency service sites involves measuring the average (or total) distance between the facilities and their demand sites. In general, as the average/total distance decreases, the accessibility and effectiveness of facilities increase. Hakimi (1964) used this property in developing his model, introducing the *P*-median method to locate *P* facilities in order to minimize the average (or total) distance between facilities and demands. The original *P*-median model assumed that the demands at each node and the travel distances between nodes of the network are deterministic. ReVelle and Swain (1970) later modeled the *P*-median problem as a linear integer program and solved it with a branch-and-bound algorithm.

Along the same line of research, Carson and Batta (1990) developed a *P*-median model to produce the dynamic strategy that can best position ambulances to minimize the average response time for a campus emergency service. Berlin et al. (1976) studied two *P*-median models to locate hospitals and ambulances. Their first model mainly focused on patient needs and aimed to minimize the average distance between the hospitals and demand points, as well as the average response time by ambulances from their bases to the demand points. Their second model was designed to enhance the performance of the system by adding a new objective function to minimize the average distance from the ambulance bases to the hospitals. Mandell (1998) adopted priority dispatching in a *P*-median problem to optimize the locations of emergency units for an EMS (emergency medical service) system that consisted of advanced life support (ALS) units and basic life support (BLS) units.

The *P*-median model has also been extended to account for uncertainty in travel times and demand patterns. For instance, Mirchandani (1980) took into account situations where service was unavailable for a demand and solved the problem using a Markov process to create a system whose states were characterized by demand distribution, service and travel time, and service unit availability. Serra and Marianov (1999) introduced the concept of regret and minmax objectives in locating a fire station in Barcelona. Their model explicitly tackled the uncertainty in demand, travel time, and distance, using scenarios to integrate the variation of uncertain factors. Their model searched for a compromise solution by minimizing the maximum regret over the identified scenarios.

2.2.3 *P*-center models

While the *P*-median model pays attention to optimizing the overall system performance, the *P*-center model concentrates on minimizing the worst system performance, emphasizing the importance of service inequity rather than the average system performance. The *P*-center model assumes that a demand is served by the nearest facility, thus making full coverage for all demand points always possible by minimizing the maximum distance between any demand and its nearest facility. However, unlike the full coverage offered by covering models, which requires excessive resources, the *P*-center model achieves its aims with limited resources.

The first *P*-center model, posed by Sylvester (1857) more than a century ago, seeks to identify the center of a circle with the smallest radius that can cover all target destinations. Since then, this model has been extended to a wide range of facility location applications, including medical (e.g., EMS centers and hospitals) and public facilities. For example, Garfinkel et al. (1977) modeled their problem with integer programming and successfully solved it with a binary search technique and a combination of exact tests as well as heuristics. The formulations by ReVelle and Hogan (1989b) for their *P*-center problem sought to minimize the maximum distance for available EMS units with a specified reliability (α). They considered system congestion and derived the probability of a service unit being busy to constrain the service reliability for all demands.

The *P*-center models have also been extended to consider its stochastic aspect. For instance, Hochbaum and Pathria (1998) tried to minimize the maximum distance on the network over all time periods. Since the costs and the distances between locations differ in each time period, they used k fundamental networks to represent different time periods and then developed a polynomial-time approximation algorithm to solve for each problem. Another instance is the application for locating and dispatching three emergency rescue helicopters for EMS demands due to accidents related to skiing, hiking and climbing the north and south Alpine mountain during holiday seasons (Talwar, 2002). The problem was solved by using effective heuristics in order to minimize the worst response times.

In addition to the aforementioned studies, a wide range of applications with different formulations can be found in the literature (Handler, 1990; Brandeau et al., 1995; Daskin, 2000; and Current et al., 2001).

2.3 Incident Duration Estimation

Reliable estimation of incident duration has long been studied by researchers for several decades with various methodologies. At the earlier stage researchers used the descriptive statistics of the data from closed-circuit television (CCTV) logs (1964), police logs (1971), and the time lapse camera (1974) to estimate the incident duration distribution. As more advanced technologies for data collection emerged over the past decades, traffic researchers have developed more analytical methodologies. Most existing approaches found in the literature can be sorted into the following categories: (1) probabilistic distributions, (2) conditional probabilities, (3) regression models, (4) discrete choice or classification models, (5) decision or classification trees, and (6) time sequential models. The rest of this section discusses each approach in detail.

2.3.1 Probabilistic distributions

Probabilistic models, the first category of approaches for estimating incident durations that this study will review, are relatively straightforward. These models center on the idea of viewing an incident's duration as a random variable and attempting to find a probability density function (PDF) that can fit the data set. Golob et al. (1987) conducted their research using approximately 530 incidents involving trucks and found that they could model incident duration with a log-normal distribution. Their findings were later supported by Giuliano (1989), Garib et al. (1997), and Sullivan (1997) in their studies of freeway incident durations. Ozbay and Kachroo (1999) also found that the distribution of incident durations from their data set showed a shape very similar to the log-normal distribution, although a few statistical significance tests rejected their hypothesis. However, they realized that when the study data set was subdivided by incident type and severity, these subsets followed a normal distribution. This finding has important implications, since it supports the theory that incident duration is a random variable (Smith and Smith, 2002). Similarly, Jones et al. (1991) discovered that a log-logistic distribution could be used to describe their study data set from Seattle. Nam and Mannering (2000) found that their data set could be illustrated with the Weibull distribution. However, Smith and Smith (2002) could not find an appropriate probability distribution, including log-normal and Weibull distributions, to fit the incident clearance times for their study data.

2.3.2 Conditional probabilities

Probability models for incident duration can be extended to apply a conditional probability methodology. The key idea of such models is to find the probability distribution of incident duration under certain given conditions — for example, the probability that the incident duration will run over 30 minutes, given that the incident has already lasted for ten minutes. It seems intuitively clear that the probability of an incident being removed within a given period of time would vary depending on how long the incident has already lasted — described as "duration dependence" by Nam and Mannering (2000) — and the incident's characteristics. One interesting approach using this concept is the hazard-based duration model. This model allows researchers to calculate incident duration with conditional probability models. Such models have been widely used in biometrics and industrial engineering to determine causality from duration data. Due to similarities with the nature of traffic incident durations, the theoretical concepts and models from these fields have recently been applied to transportation problems. Such approaches expand the focus from simply estimating and predicting an incident's duration to computing the likelihood that the incident will be cleared in the next short time period given its sustained duration.

One representative study using this methodology (Nam and Mannering, 2000) used a two-year data set from Washington State. The study showed that each incident time (i.e., detection/reporting, response, and clearance times) was significantly affected by numerous factors, and different distribution assumptions were recommended for different incident times. They also found that the estimated coefficients were unstable through the two-year data used in model development. Nam and Mannering concluded that this approach was more useful for determining which variables have greater influence on incident duration than for estimating or predicting the incident duration for given explanatory variables. Chung (2010) recently utilized a similar approach, the log-logistic accelerated failure time (AFT) metric model, to estimate/predict accident durations based on a two-year (2006 and 2007) accident data set from the Korean freeway system. The estimated duration model, based on year 2006 data, was validated with year 2007 data; the author concluded that the predictions of the developed model were reasonably acceptable. He also tested the temporal transferability of the proposed model and concluded that the estimated model parameters can be stable over time.

2.3.3 Regression models

Another simple methodology for predicting incident duration uses regression. These models usually include a number of binary variables as independent variables to indicate incident characteristics and a continuous or categorical variable as a dependent variable (i.e., incident duration). One of the best-known linear regression models for incident duration prediction was developed by Garib et al. (1997) using 277 samples from California. The researchers used various independent variables to represent incident characteristics (e.g., incident type, number of lanes affected by the incident, number of vehicles involved, and truck involvement) and weather conditions (rainy or dry). They also included all possible combinations of the independent variables to optimize the model. Here is the final incident duration model from their research:

$$Log(Duration) = 0.87 + 0.027x_1x_2 + 0.2x_5 - 0.17x_6 + 0.68x_7 - 0.24x_8$$

where Duration = incident duration (minutes)

x_1 = number of lanes affected by the incident

x_2 = number of vehicles involved in the incident

x_5 = truck involvement (dummy variable)

x_6 = morning or afternoon peak hour indicator (0: morning peak hour; 1: afternoon peak hour)

x_7 = natural logarithm of the police response time (minutes)

x_8 = weather condition indicator (0: no rain; 1: rain)

The logarithm form of incident durations indicates that the incident durations in this data set follow a log-normal distribution based on the Kolmogorov-Smirnov test. This result is similar to those from Golob et al. (1987) and Giuliano (1988). According to the authors, the police response time was the most significant factor affecting the incident duration, followed by weather conditions, peak hour, truck involvement, and the combined effect of the number of lanes and number of vehicles involved in the incident.

2.3.4 Discrete choice or classification models

While most studies in the literature have treated incident duration as a continuous variable, several researchers categorized the continuous variable of incident duration into discrete time intervals (e.g., 10 to 25 minutes) in order to apply discrete choice or classification approaches. For instance, Lin et al. (2004) developed a system that integrates a discrete choice model and a rule-based model to predict incident duration. They adopted the ordered probit models to first predict incident durations in a time interval format; then they developed and applied a rule-based supplemental model to enhance the accuracy of prediction results. Boyles et al. (2007) also redefined their original incident duration data into an interval format in developing their naïve Bayesian classifier (NBC), based on incident data from the Georgia Department of Transportation. They argued that the NBC has the following distinct advantages: (1) flexibility in accommodating changeable amounts of information (incomplete information or information received at different points in time), (2) increased robustness to outliers than standard techniques like linear regression, (3) computational simplicity, (4) easy adaptability as the number of samples for calibration grows, and (5) relative ease in interpreting research results.

2.3.5 Decision or classification trees

Another approach frequently appearing in the incident duration literature, the decision or classification tree model, has proven quite useful for discovering patterns in a given data set without considering the fundamental probabilistic distribution (Smith and Smith, 2001). This property is very helpful, since most incident duration data sets do not fit well to any commonly used distribution. Smith and Smith (2001) also pointed out that the pattern-recognition model has been used recently to develop incident duration models. One representative model, developed by Ozbay and Kachroo (1999) for the Northern Virginia region, began with a model to predict the clearance time using linear regression based on a large sample size. Unfortunately, the completed analysis with an unsatisfactory result ($R^2 \approx 0.35$) showed that their incident clearance time data followed neither a log-normal nor a log-logistic distribution. As an alternative method, they explored a decision tree model and finally generated relation patterns (see Figure 2-1) for use in predicting clearance times.

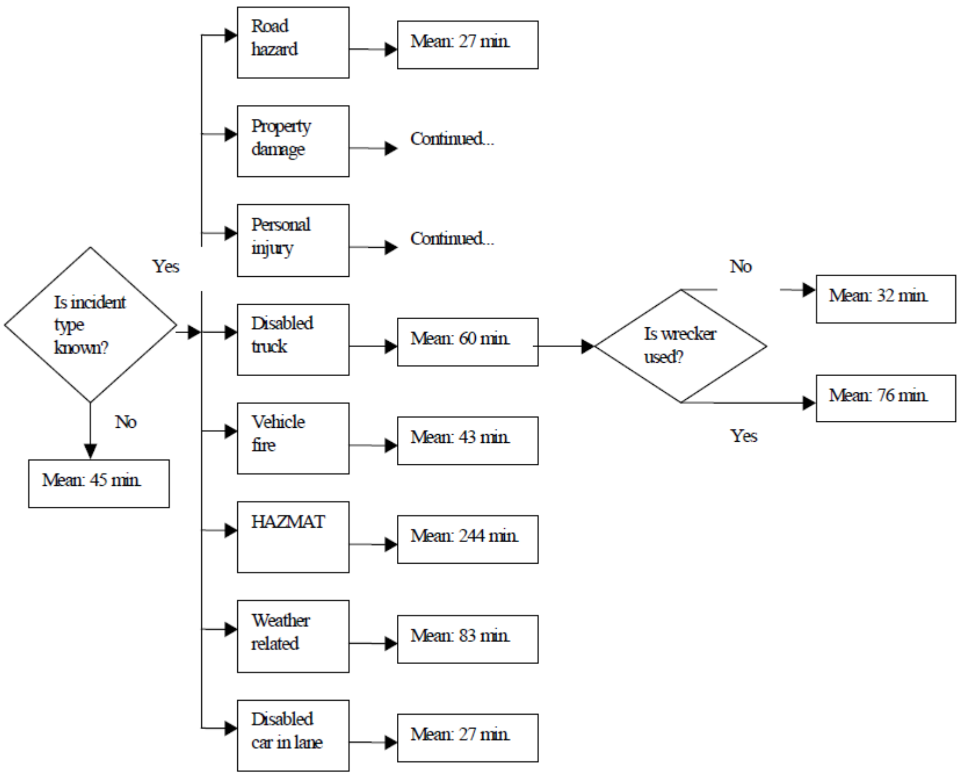


Figure 2-1: A Part of the complete decision tree to predict clearance Time by Ozbay and Kachroo (1999)

Note that an incident tree comprises a series of decision variables. This is another advantage of the tree-type methodologies — their self-explanatory nature, which is rooted in the tree-structure. Users can easily understand the output by following the branches related to the conditions of variables. For instance, the tree uses an incident type as the first variable to decide if the detected incident type is known or not. Once it is classified as an unknown type, then the tree immediately provides an estimate of 45 minutes for the average clearance time. Otherwise, it moves to the next level to determine the type of incident.

Smith and Smith (2001), inspired by the study of Ozbay and Kachroo, tried to develop a similar classification tree. They concluded that such a tree, developed on the basis of a reliable and sufficient database, performs well — even though theirs yielded unsatisfactory results due to poor data quality. To enhance its adaptability to incomplete information in real-time prediction, Yang et al. (2008) developed a Bayesian decision tree, which can predict incident duration with missing or inconsistent information. They inserted Bayesian nodes following every decision node to ask whether the required information is available. If the information is available, no further calculation will occur for that node. Otherwise, the model uses Bayesian theory to compute the value of the node. Then, the computed Bayesian node value is used to estimate the time interval class to which the detected incident belongs. Their model reportedly outperformed the traditional classification tree model developed on the same data set — the Bayesian decision tree and classification tree yielded 74 and 46 percent prediction accuracies, respectively.

2.3.6 Time sequential models

Khattak et al. (1995) realized that the full set of variables for incident forecasts would be available at the moment the incident was cleared. Although prediction models based on this total set of variables would be more accurate and reliable, they are less practical for use in real-time incident management operations — precisely because a full set of variables would only become available after clearing the incident. Thus, they introduced a time sequential model that focuses on predicting real-time incident duration under partial information. Their model considers ten distinct stages of incident duration, based on the availability of information. Each stage estimates different ranges of incident duration with a separate truncated regression model. As the model moves to the next stage, it includes progressively more variables to explain the stage's duration. Despite its originality and reasonability, this model was not tested or validated due to the lack of field data. The authors also mentioned that the purpose of their study was to introduce and demonstrate the time sequential model rather than to prove its performance in traffic operations.

Since then, their approach has been extended and enhanced by several researchers. For instance, Wei and Lee (2007) proposed an adaptive procedure which includes two artificial-neural-network-based models for sequentially forecasting an incident's duration. The first model, the so-called *Model A*, was designed to predict the duration of the detected incident at its notification, at which point *Model B* takes over and updates the duration at multiple periods until clearance of the incident. The performances of these models were evaluated with mean absolute percentage error (MAPE) for predicted incident durations at every forecast time period; most results were less than 40 percent, implying that the proposed models are capable of yielding reasonable forecasts. Later, they improved their model by adding a procedure to select a best-performing subset of features using genetic algorithms (GAs) (Lee and Wei, 2010). They found that reducing the dimensionality of input features can decrease the cost of acquiring data and increase the interpretability and comprehensibility of model outputs. Furthermore, they claimed that data simplification can eliminate irrelevant data which can mislead the learning process and impair development of the final model. In fact, they reported that the MAPE for forecasted incident duration at each time period dropped, mostly falling below 29 percent after they applied their proposed feature selection method.

Qi and Teng (2008) also developed a time sequential procedure which divides the incident management process into multiple stages, depending on the availability of specific information. They developed a hazard-based duration regression model for each stage, with different variables representing different information available. The remaining incident duration, then, could be predicted online using the truncated median of incident duration and the estimated parameters and coefficients. Their study concluded that prediction accuracy increased as more information was integrated into the developed models.

Although various techniques produce acceptable results, most research findings are not directly applicable to other locations. Because each model was developed using different incident data sources and descriptive variables, and each yielded somewhat different results. Therefore, any target application requires either a recalibration of existing models in the literature to adjust the parameters to the new data source or the development of a new model which can outperform any existing models with the available data set.

2.4 Detour Decision Support System

Contending with non-recurrent congestion has long been a priority task for most highway agencies. A recent study (Lindley, 1987) pointed out that non-recurrent traffic congestion due to incidents, highway construction zones, and special events has contributed up to 60 percent of the total freeway corridor delay in the United States. Under most incident scenarios, if proper diversion plans can be implemented in time, motorists can circumvent the congested segments and best use the available corridor capacity. To contend with this vital operational issue, transportation professionals have proposed a variety of advanced diversion control and route guidance strategies (Pavlis and Papageorgiou, 1999; Morin, 1995; Papageorgiou, 1990; Messmer and Papageorgiou, 1995; Wu and Chang, 1999; Liu et al., 2011) to optimally balance the volumes between the freeway and the arterial systems. Certainly, those strategies have made invaluable contributions to improving incident management in freeway corridors.

Nevertheless, before implementing any detour strategy, traffic operators must justify its necessity based on various factors, since such operations usually demand substantial amount of resources and manpower. In this regard, very limited information is available in the literature to assist decision makers in assessing the benefits and costs of implementing detour operations, although numerous traffic safety and operation manuals (e.g., Delaware DOT, 2011; State Police NJ, 2010; Univ. of Kentucky, 2009; FHWA, 2009; Wisconsin DOT, 2008) have addressed the need of properly diverting traffic flows during major incidents or emergencies.

One source offering guidance for detour plan development, the Alternate Route Handbook (2006), provides comprehensive and general guidelines for how to plan and execute the detour operations involving various stakeholder agencies. According to this document, the key factors to consider include incident duration, number of lanes blocked, observed traffic condition, time of day, and day of the week. The capacity of the proposed alternative route and its background traffic are also critical factors.

Table 2-1 summarizes the criteria used in several states to decide whether or not to execute the predeveloped alternate route plan. Notice that the Florida DOT District IV operates its detour plan when two or more lanes are closed for at least two hours. On the other hand, most states require an incident duration of longer than 30 minutes or the complete closure of the roadway to implement detour plans. The Manual on Uniform Traffic Control Devices (MUTCD) (2009) states that major and intermediate incidents lasting more than 30 minutes usually require traffic diversion or detouring for road users, due to partial or full roadway closures, while traffic diversion may not be necessary for minor incidents usually cleared within 30 minutes. In reviewing the literature, it becomes evident that a reliable tool for traffic control operators to decide when and how to implement detour operations has yet to be developed.

Table 2-1: Criteria for deciding the implementation of detour plans in various states

AGENCY	CRITERIA
North Carolina DOT – main office	• Complete closure of the highway in either direction is anticipated for 15 minutes or longer.
North Carolina DOT – Charlotte regional office	• No action or discussion occurs until 15 minutes after the incident. After 15 minutes, an alternate route plan is deployed only if the highway is completely closed (all lanes closed, including the shoulder) and closure is expected to last at least an additional 15 minutes (30 minutes total).
North Carolina DOT – Charlotte regional office	• No action or discussion occurs until 15 minutes after the incident. After 15 minutes, an alternate route plan is deployed only if the highway is completely closed (all lanes closed, including the shoulder) and closure is expected to last at least an additional 15 minutes (30 minutes total).
New Jersey DOT	• Level 1: Lane closures on a state highway that are expected to have a prolonged duration and impact on traffic. • Level 2: Complete closure of a highway that is anticipated to last more than 90 minutes.
Oregon DOT	• Incident with two or more lanes blocked, or • Incident with one lane blocked and expected to last more than 20 minutes.
New York State DOT Region 1	• Implemented only when the highway is completely closed. • Will not be implemented if at least one lane(or even the shoulder) is open.
Florida DOT District IV	• Two or more lanes blocked for at least two hours.
ARTIMIS (Ohio/Kentucky)	• This plan has a detailed table with four different levels, based on some present criteria, such as: - During the morning and afternoon peak hours, an advisory alternate route is deployed in the event of a two-lane closure for more than two hours or a closure of more than two lanes for less than 30 minutes. - Mandatory alternate routes are deployed during the peak hours when more than two lanes are closed for at least 30 minutes.
Ada County, Idaho	• This plan specifies different levels of severity, including: - Levels C and D require implementation of a diversion route. - Level C is an incident taking 30 to 120 minutes from detection to full recovery of the traffic flow. - Level D is an incident taking over two hours from its detection to full recovery (including full freeway closure in one or both directions).
Wisconsin DOT (Blue Route)	• Incident causes delays that will exceed 30 minutes.

Source: Alternate Route Handbook (2006)

In view of the benefits and limitations in the existing studies and the additional functional requirements for real-world system applications, this study aims to achieve the following goals so as to mitigate incident impacts on freeways:

- Provide reliable guidelines and tools to help responsible agencies when they design, determine, and operate traffic management plan/program under non-recurrent congestion.
- Deliver a solid, integrated freeway traffic management system which can be utilized as a prototype and/or applied in real-time traffic operations

CHAPTER 3: Review of the FITM Program and Operations

3.1 Introduction of the FITM Program by SHA

To improve traffic conditions on major highways plagued by non-recurrent congestion, most highway agencies have invested their resources in two principal operational programs: incident response and clearance, and traffic impact management. The former includes effective detection of incidents, efficient response, and well-coordinated clearance operations; the latter focuses on minimizing incident impacts via dissemination of traffic information and implementation of necessary control strategies such as ramp closures or detour operations. However, even with the wide-spread implementation of such programs, effectively minimizing the traffic impact caused by multi-lane blocked incidents remains an especially critical and challenging issue for most highway agencies.

The FITM program developed by SHA aims to provide the guidelines for detour operations during the clearance of major incidents. FITM is a set of location-specific routing plans to detour vehicles from an incident-impacted highway segment to a pre-selected neighboring parallel arterial and then guide traffic back to the same route. For all major highway corridors covered by the CHART program, the FITM program offers a detailed operational manual for each roadway link when detour operations in response to a severe incident are justified. Figure 3-1 illustrates an example of the detour plan provided by the FITM operations manual for the roadway link between exits 7 and 9 of I-495 on the Capital Beltway.

It is noticeable that the map-based routing plan for an identified incident location shows clearly where to exit and return to the primary route, the number of intersections on the detour route, and some key geometric or control features on those detour links that may affect the operational efficiency. The operational manual also provides a detailed link-based navigation in a table format and the emergency contact phone numbers for detouring travelers (see Table 3-1). Since the detour traffic certainly will cause a volume surge on the detour route and demand the intersection signals to accommodate with a responsive timing plan, the FITM operational manual also lists the affected signals and their phases that need to be adjusted during the period of incident response and traffic management.

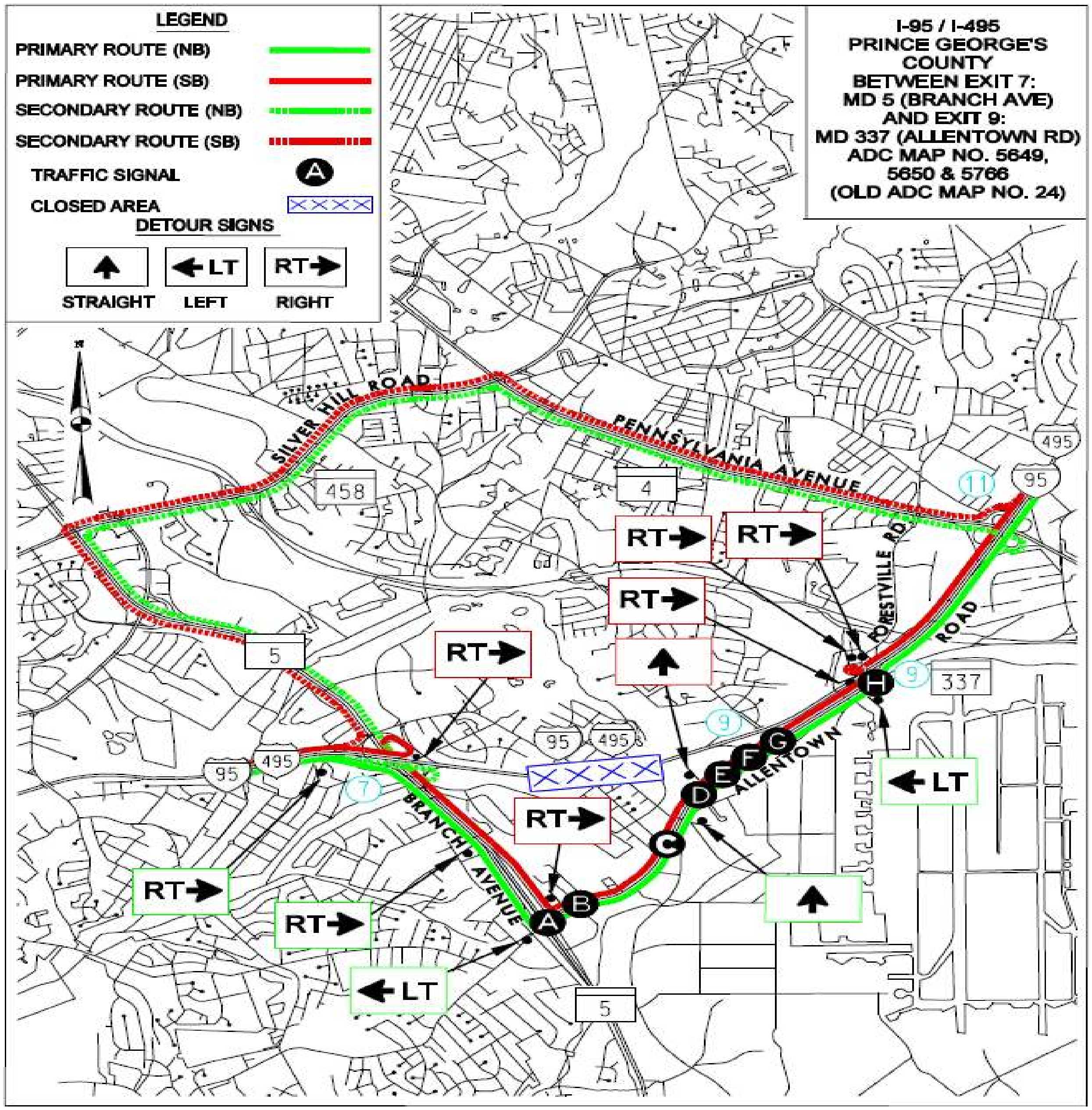


Figure 3-1: An example of the detour plan on I-495 provided by the FITM manual

Table 3-1: An example of the detour plan on I-495 provided by the FITM manual

I-95/I-495 - PRINCE GEORGE'S COUNTY (UPPER MARLBORO SHOP) DISTRICT 3
INCIDENT BETWEEN EXIT 7: MD 5 (BRANCH AVENUE) AND EXIT 9: MD 337 (ALLENTOWN ROAD)

ALTERNATIVE ROUTES

NORTHBOUND FROM I-95/ I-495
PRIMARY DIVERSION – 8 SIGNALS (A-H)
MERGE RIGHT onto EXIT 7: MD 5 (Branch Avenue)
MERGE RIGHT to MD 337 (Allentown Road)
TURN LEFT onto MD 337 (Allentown Road)
TURN LEFT at Forestville Road onto I-95/I-495 Northbound

SOUTHBOUND FROM I-95/ I-495
PRIMARY DIVERSION – 8 SIGNALS (H-A)
MERGE RIGHT onto EXIT 9: MD 337 (Allentown Road)
TURN RIGHT onto Forestville Road
TURN RIGHT onto Allentown Road
TURN RIGHT onto MD 5 (Branch Avenue)
MERGE RIGHT onto I-95/I-495 Southbound

EMERGENCY CONTACT NUMBERS	
MD State Police Barrack Forestville (L)	(301) 568-8101
Statewide Operations Center	(410) 582-5650
Prince George's County Fire	(301) 333-4000
Prince George's County Police	(301) 883-5200
Prince George's County TRIP Center Dispatch	(301) 324-2710
Department of Environment and Hazardous Materials	(410) 974-3351
Traffic Operations Center 3	(301) 345-7130

SUGGESTED DYNAMIC MESSAGE SIGNS (DMS)
3314, 3315, 3319, 3317, 3320, 3322, 3323, 3327, 3331, 3332, 4401, 4406, 5501, 7703, 7704, 7706, 7707
SUGGESTED HIGHWAY ADVISORY RADIO (HAR)
393, 395, 398, 498, 798

AFFECTED TRAFFIC SIGNALS									
Signal ID	Intersection	Affected Direction				Maintained by			Signal Operation*
		NB	SB	EB	WB	SHA	CO	MNCPLTY	
A	MD 337 at MD 5 Ramps			X	X	X			C
B	MD 337 at Allentown Mall Entrance			X	X	X			C
C	MD 337 at Auth Road			X	X	X			C
D	MD 337 at Andrews Air Force Base Westgate			X	X	X			C
E	MD 337 at Andrews Manor			X	X	X			C
F	MD 337 at I-95 NB Off-ramp			X	X	X			C
G	MD 337 at Suitland Road/ Westover Drive			X	X	X			C
H	MD 337 at Forestville Road		X	X		X			C

* - I – Isolated, T – Time Based, C – Coordinated System

Note that the FITM program by SHA currently covers only the following major routes:

- Interstate highways: I-70, I-81, I-83, I-97, I-270, I-695, I-495, and I-95;
- State routes: MD100, MD295, MD404; and
- US routes: US13, US50, US113, US301.

For a detected incident, SHA traffic engineers are instructed to use the following criteria to determine whether or not an FITM plan should be implemented:

- The estimated incident duration exceeds one hour; or
- The incident nature and its clearance cause a complete lane blockage.

However, the FITM operational manual does not address a method to dynamically adjust signal times in response to the time-varying detouring traffic flows, nor does the current program discuss such critical operational issues as:

- Where are the locations to place portable DMS to guide the detouring travelers?
- What messages should be displayed to inform the approaching roadway users?
- How should the initial estimate of the clearance duration be provided for a detected incident and its impact boundaries?
- How should the incident management progress be updated for travelers within the incident impact area?
- How should the most effective traffic control and management plan be selected for the same incident location but at different times of a day such as peak versus off-peak periods?
- Should the detour operations be sustained when the clearance operation has recovered the roadway from a complete blockage to the partial lane-open status?
- What percentage of traffic should be detoured during different stages of the clearance operations (e.g., one lane or two-lane open) to avoid excessive congestion on the detour route?
- When will be the optimal time to terminate the detour operations, considering both the approaching traffic volume on the primary and the detour routes?

Aside from dealing with the above issues, the current FITM program does not include some essential operational steps to coordinate with local agencies responsible for managing traffic and controlling signals on the detour route.

3.2 Criteria for Activating Detour Operations

Among all the critical issues to be addressed by the FITM program, the most difficult one is when to activate the detour operations, because its effective implementation needs not only sufficient equipment for information dissemination and control management, but also vast human resources to ensure driver compliance. Hence, all those highway agencies with an active incident response program have adopted their own criteria to activate detour operations.

The last chapter on literature review has reported those criteria used by nine highway agencies. Notably, all such criteria vary widely, though mainly based on the incident duration and lane blockage level. Nevertheless, they can be classified into two categories;

Category-1: Implement the *mandatory detour operation* only if

- All lane closure and the incident duration are expected to last more than 15 minutes (e.g., North Carolina state main center), 30 minutes (North Carolina), or 90 minutes (e.g., New Jersey);
- Either All-lane closure or the incident duration are expected to last more than 60 minutes (e.g. Maryland);
- Roadway closure for two lanes or more (Oregon) and for more than two hours (Florida, Idaho); and
- Roadway closure of more than two lanes for more than 30 minutes during the peak hours (Ohio and Kentucky);

Category-2*:* Activate the *advisory detour operation* only if

- One-lane closure for more than 20 minutes (Oregon);
- Two-lane closure for more than two hours or more than two-lane closure for less than 30 minutes (Ohio and Kentucky); and
- Delays caused by the detected incident exceed 30 minutes.

In general, those states that experienced more heavy recurrent congestion are less willing to implement detour operations during the duration of incident clearance. For example, New York will not activate its traffic diversion program as long as one or the shoulder lane remains open, and SHA would activate its FITM operations only if the incident duration exceeded one hour.

3.3 A FITM Decision and Evaluation Framework

Despite the discrepancies among various criteria for detour operations, most highway agencies share the following two common principles in contending with non-recurrent congestion:

- Detour operations demand well-designed plans in advance and need vast personnel and resources to implement effectively and in a timely manner; and
- Care should be exercised to ensure that drivers complying with the incident management instructions will not suffer excessive and undue delays on the suggested alternate route.

The challenge for highway agencies in contending with this issue is that during the partial-lane closure scenario drivers may encounter unexpectedly long travel times and congestion on the alternative route if proper messages, route guidance, and signal accommodation have not been placed in time to guide the traffic flow. Otherwise, very often those staying on the primary route during the partial-lane closure period may actually experience a shorter travel time, and drivers will consequently lose confidence in traffic information or instructions displayed via DMS or any means of advanced transportation information systems (ATIS). This is one reason that most highway agencies are reluctant to divert traffic unless the need and benefits are too obvious to ignore.

Conceivably, a successful operation of any detour plan during a major incident needs full cooperation between motorists and the responsible agencies. While the latter should have a well-prepared plan and sufficient resources to accomplish the objective of minimizing the total societal cost during the incident clearance period, the former should be willing to follow the instructions to reduce their delays incurred by the accident and clearance operations. Unfortunately, developing such mutual confidence and cooperation between motorists and traffic agencies at the desirable level is a difficult task and much remains to be done in incident management practice. The following section presents a framework for an integrated FITM system in response to such a need.

Undoubtedly, a reliable and efficient FITM system needs to first win the trust or confidence of motorists who can be convinced that any message or instruction from responsible highway agencies during the incident-plagued period can lead them to experience less travel cost. Such an effective FITM, however, must have the following essential components to accomplish its mission (see Figure 3-2):

- *Incident detection module* that can take advantage of various sources of available information (e.g., detector, cellular calls, probe vehicles);
- *Incident duration module* that can accurately estimate the required clearance and traffic recovery durations, and the time-varying traffic impact boundaries; and
- *Traffic evaluation and optimization module* to ensure that a candidate detour plan indeed can minimize the total congestion and societal cost at the network level, based on the estimated incident impacts, distribution of network traffic volumes, recurrent congestion level at the detour routes, required operational costs, and the resulting benefits.

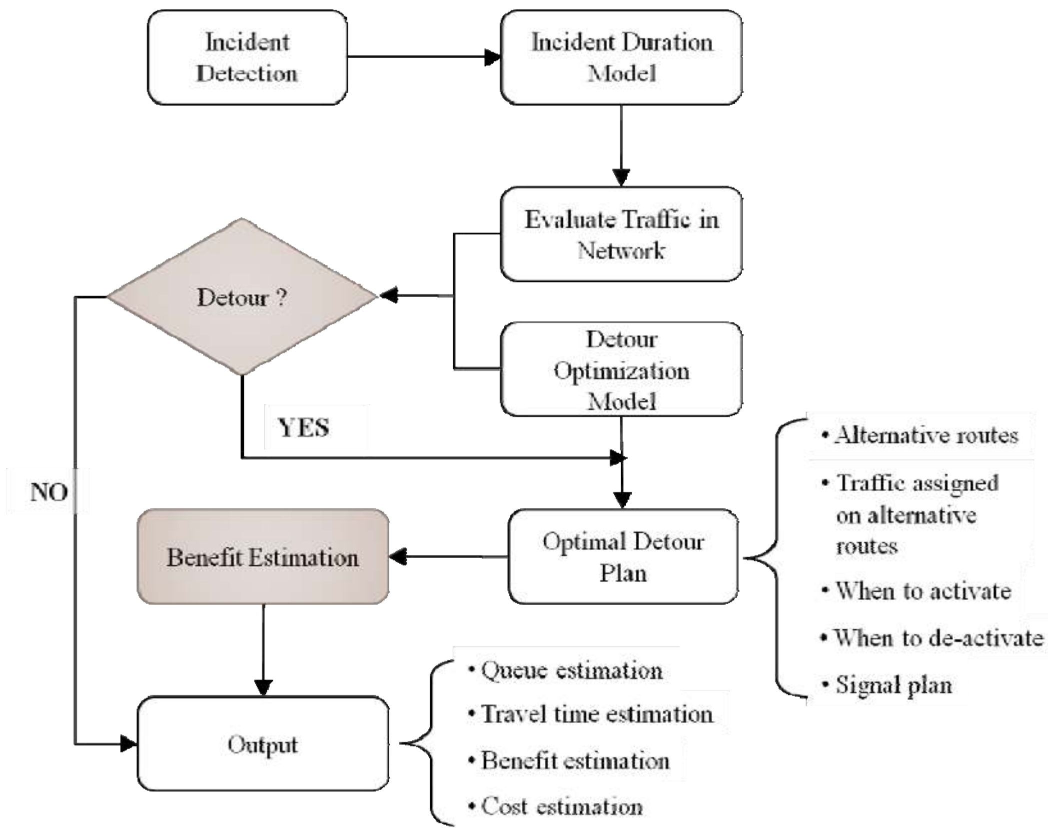


Figure 3-2: Flowchart for the FITM decision and evaluation system

Note that the third module for traffic evaluation and optimization should be sufficiently efficient in producing the cost/benefit estimate for responsible agencies to make the decision in real time. If the suggestion of implementing detour operations is adopted, this system should also be able to provide the following essential information:

- The starting and deactivating times for the detour operations;
- The set of alternate routes and their roadway segments to be in the FITM period;
- The optimal detour percentage that may vary with the progress of clearance operations and approaching volumes at both the primary and detour routes; and
- The set of revised signal timing plans for those intersections on the detour route to accommodate the surge in different traffic directional flows.

Upon full recovery of the traffic conditions, this vital module should be able to produce the total direct (e.g., reduction in delays, fuel consumption, and emissions) and indirect benefits associated with the operation for the responsible traffic agency to justify its costs and future resource needs. If the incident impacts for the detected network traffic conditions are not significant enough to trigger the FITM operations, then this vital module should yield the estimated time-varying queue length and travel time information during the incident clearance period to en-route motorists via available ATIS.

3.4 Analysis of FITM-incidents

This section presents those incidents that triggered the FITM operations by CHART between 2007 and 2010. For convenience of illustration, those major incidents will be referred as "FITM-incidents," hereafter in the remaining presentation.

Figure 3-3(a) shows the distribution of FITM-incidents among freeways, and Figure 3-3(b) illustrate those in the "Others" category, mostly major arterials covered by CHART operations. It is noticeable that 49 out of a total of 78 FITM operations between 2007 and 2010 took place in local highways, reflecting the increasing demand of effectively contending with non-recurrent congestion even for non-freeway traffic corridors.

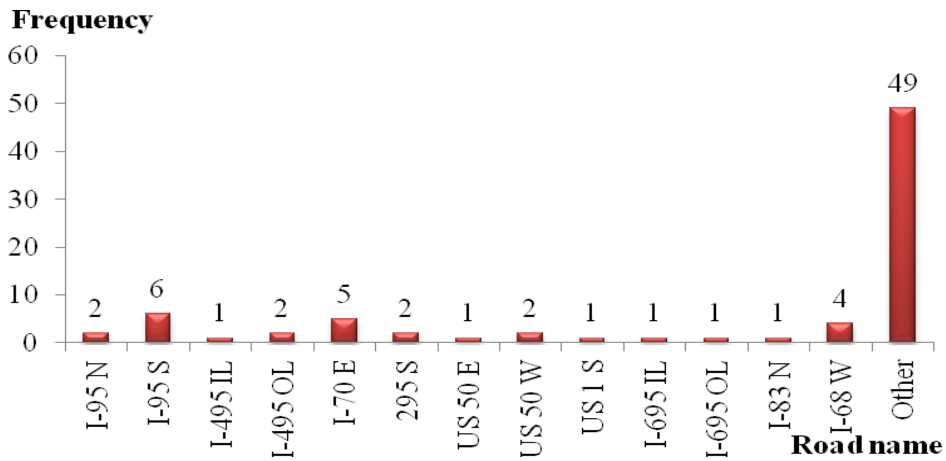


Figure 3-3(a): Distribution of FITM-incidents by highway

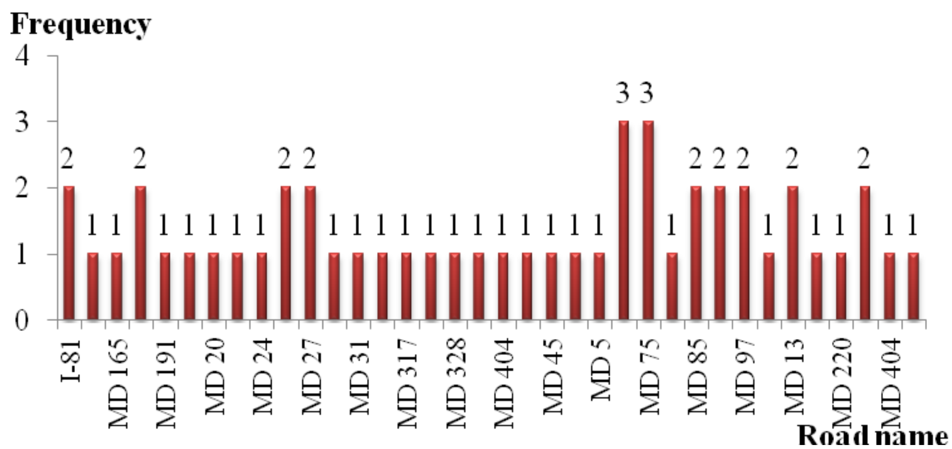


Figure 3-3(b): Distribution of FITM-incidents classified in the category of OTHER

Further analysis of FITM incidents with respect to the truck involvement is shown in Figure 3-4, where about 49 percent of those operations involved one or more trucks. Among those 37 percent of incidents involving one truck, about 22 percent of their durations were longer than 4 hours. This seems to indicate that any major incident involving trucks has about 60 percent (i.e., 22 percent out of 37 percent) of the probability to block the traffic for more than four hours.

In contrast, among 51 percent of FITM incidents incurred solely by passenger car, only 13 percent of those had the duration of more than four hours, about 25 percent (13 out of 51 percent) of the probability for incidents in this category. This clearly evidenced the common belief that incidents involving trucks generally require more effort and time for emergency response teams to return traffic to normal conditions. .

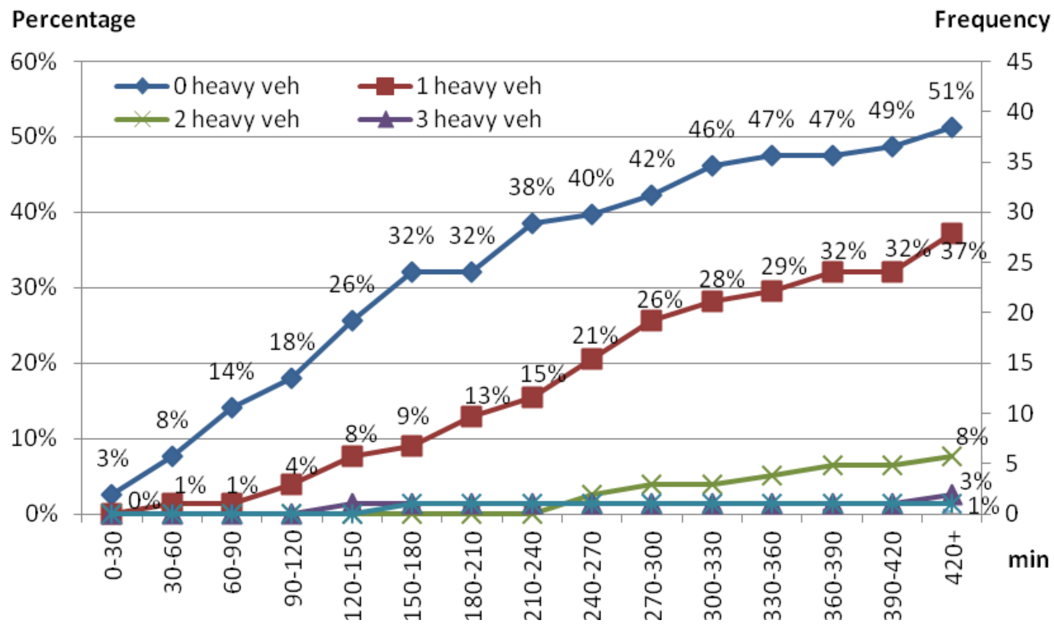


Figure 3-4: Distribution of FITM-incidents by truck involvement

Figure 3-5 presents the distribution of FITM-incidents by weekend and weekday, where only about 12 percent of those incurred during weekends. However, it should be noted that some FITM operations implemented during weekends seem to take more than three hours (i.e., 4 percent out of 12 percent incidents), which may be related to the limited available resources during non-working days.

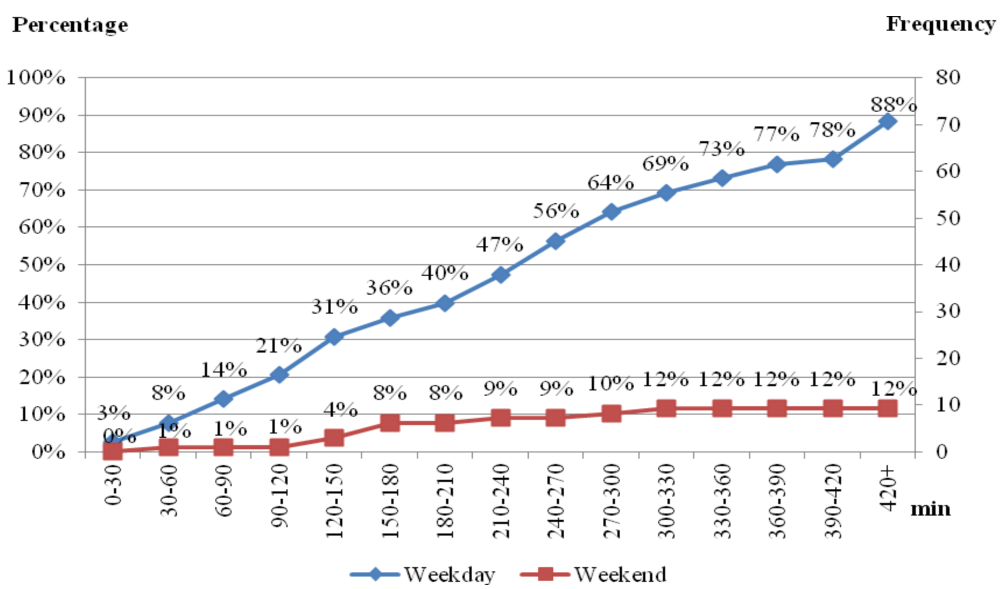


Figure 3-5: Distribution of FITM-incidents by weekend and weekday

Table 3-2 classifies those FITM-incidents by the number of blocked lanes, where about 90 percent resulted in more than one lane blockage. Among those, the type of incidents causing 4-lane blockage exhibits the highest frequency of 26 during the period of 2007-2010. The distribution of average incident durations by blocked lanes, however, does not show any distinct pattern, and mostly last between three to four hours except those blocking only one lane.

Table 3-3 presents the distribution of FITM-incidents during peak and off-peak periods from 2007-2010, where most such incidents occurred during the off-peak periods, It is worth noting that CHART's incident management team activated a total of 59 FITM operations (47 in off-peak periods) in 2008, a significantly higher frequency than other years. The distribution of those 59 FITM-incidents in 2008 among major highways can be viewed from Table 3-4 which displays all FITM-incidents between the analysis period by road name and by year. Except I-95S and I-70E exhibited a relatively high frequency in 2008, the distribution of FITM-incidents among all other highways appears to be quite random in nature.

The distribution of all FITM-incidents by nature, based on the definitions by SHA, is summarized in Table 3-5. As expected, collision was the obvious contributing factor and it compounded with injury had the highest frequency of 26 in 2008, followed by the frequency of 13 due to "collision and fatality" in the same year. The distribution of incident durations, however, does not reveal any obvious pattern.

Table 3-2: Distribution of FITM-incidents by lane blockage type

No. lane closed	Frequency(cumulative percentage)	Average Incident Duration (SD*) Unit: minute
0	8 (10%)	261.60 (148.26)
1	1 (12%)	23.02 (N/A)
2	9 (12%)	258.48 (171.39)
3	5 (23%)	202.87 (69.85)
4	26 (63%)	225.39 (137.45)
5	7 (72%)	286.31 (115.58)
6	11(86%)	216.85 (99.25)
7	2 (88%)	152.44 (133.42)
8	6 (96%)	162.48 (92.66)
9	1 (97%)	159.82 (N/A)
10	2 (100%)	253.87 (94.87)

* Standard deviation

Table 3-3: Distribution of FITM-incidents during peak and off-peak hours

Year	Time of Day	Frequency	Average Incident Duration (SD*) Unit: minute
2007	Off-peak	1	218.92 (N/A)
	Peak	1	23.02 (N/A)
2008	Off-peak	47	252.76 (125.86)
	Peak	12	192.97 (140.87)
2009	Off-peak	5	161.72 (82.00)
	Peak	2	118.10 (84.85)
2010	Off-peak	5	196.58 (123.76)
	Peak	5	237.68 (142.18)

Table 3-4: Distribution of FITM-incidents by road between 2007-2010

Year	Road	Frequency	Average Incident Duration(SD*) Unit: minute
2007	I-495 OL	2	120.97 (138.52)
2008	I-95 N	2	163.06 (4.58)
	I-95 S	6	259.73 (119.90)
	I-495 IL	1	46.77 (N/A)
	I-70 E	4	227.45 (149.81)
	MD 295 S	2	356.31 (71.29)
	US 50 E	1	268.63 (N/A)
	US 50 W	2	104.69 (100.07)
	I-695 IL	1	374.70 (N/A)
	I-695 OL	1	384.17 (N/A)
	I-83 N	1	252.37 (N/A)
	I-68 W	3	243.06 (88.90)
	I-81	2	323.93 (292.61)
	MD 10	1	282.48 (N/A)
	MD 165	1	78.47 (N/A)
	MD 175	1	320.17 (N/A)
	MD 191	1	283.20 (N/A)
	MD 2	1	87.83 (N/A)
	MD 20	1	268.12 (N/A)
	MD 213	1	575.03 (N/A)
	MD 24	1	144.55 (N/A)
	MD 26	2	274.27 (66.02)
	MD 27	1	201.18 (N/A)
	MD 298	1	229.75 (N/A)
	MD 31	1	219.18 (N/A)
	MD 317	1	124.22 (N/A)
	MD 322	1	301.77 (N/A)
	MD 404	1	123.62 (N/A)
	MD 450	1	142.43 (N/A)
	MD 5	1	302.67 (N/A)
	MD 7	3	134.18 (90.64)
	MD 75	3	400.31 (66.02)
	MD 85	1	41.77 (N/A)
	MD 91	2	150.65 (132.98)
	MD 97	1	454.13 (N/A)
	US 13	1	152.63 (N/A)
	US 15	1	440.72 (N/A)
	US 220	1	114.65 (N/A)
	US 301	1	246.78 (N/A)

Year	Road	Frequency	Average Incident Duration(SD*) Unit: minute
2009	I-70 E	1	126.83 (N/A)
	MD 328	1	141.73 (N/A)
	MD 4	1	58.10 (N/A)
	MD 41	1	161.63 (N/A)
	MD 77	1	80.18 (N/A)
	US 13	1	178.10 (N/A)
	US 301	1	298.22 (N/A)
2010	US 1S	1	282.50 (N/A)
	US 1S	1	282.50 (N/A)
	MD 175	1	124.40 (N/A)
	MD 27	1	19.80 (N/A)
	MD 313	1	216.30 (N/A)
	MD 450	1	137.15 (N/A)
	MD 85	1	331.43 (N/A)
	MD 97	1	196.63 (N/A)
	US 113	1	474.12 (N/A)
	US 40	1	256.12 (N/A)

Table 3-5: Distribution of FITM-incidents during 2007-2010 by nature

Year	Incident Nature	Frequency (cumulative percentage)	Average Incident Duration (SD*) Unit: minute)
2007	Collision and Property Damage	1 (1%)	23.02 (N/A)
	Collision and Injury	1 (3%)	218.92 (N/A)
2008	Collision and Fatality	13 (19%)	201.68 (72.65)
	Collision, Personal Injury and Fatality	8 (29%)	204.76 (88.36)
	Collision and Property Damage	8 (40%)	275.22 (151.19)
	Collision, Property Damage and Injury	3 (44%)	256.03 (186.74)
	Collision and Injury	26 (77%)	257.58 (152.95)
	Others	1 (78%)	268.63 (N/A)
2009	Collision and Fatality	4 (83%)	152.08 (22.46)
	Collision and Property Damage	1 (85%)	298.22 (N/A)
	Collision and Injury	1 (86%)	58.10 (N/A)
	Others	1 (87%)	80.18 (N/A)
2010	Collision and Fatality	1 (88%)	216.30 (N/A)
	Collision, Personal Injury and Fatality	4 (94%)	250.06 (159.92)
	Collision and Injury	5 (100%)	190.95 (124.37)

3.5 Closure

This chapter first reviewed SHA's FITM program, and then indicated potential areas for future enhancement. In view of the increasing demand by the general public to minimizing non-recurrent congestion, this chapter further presents a decision framework for SHA to develop an integrated real-time incident management system, based on the existing FITM program and various operational modules developed by the academic community over the past several years. Analyses of FITM-incident characteristics and distributions by various factors also constitute part of this chapter, highlighting the need to efficiently respond to those incidents and effectively manage traffic during the operational period.

CHAPTER 4: A Corridor Control Model for Optimizing Detour Operations

4.1 Introduction

This chapter presents an integrated model and its solution algorithm for freeway corridor control during incident management. It offers an effective tool to better capture the temporal and spatial interactions of traffic over a corridor network, including the freeway segments, arterials, and ramps. The corridor control model serves as the primary tool for the detour decision-support system described in the next chapter. Traffic engineers can apply such a model to determine when and how to implement detour operations in response to a detected incident if there are sufficient benefits.

The remainder of this chapter is organized as follows. The next section details the key model assumptions and the formulations for modeling network traffic flows, including three primary modules for traffic dynamics in the arterial, freeway, and on-off ramps. Section 4-3 illustrates the optimization model for integrated corridor control. Section 4-4 introduces the solution framework for computing the optimal detour rates and associated control parameters during incident management. Section 4-5 summarizes key research findings and potential applications.

4.2 Formulations of Network Traffic Flow Dynamics

Figure 4-1 illustrates a typical freeway corridor experiencing an incident, including the upstream on-ramps and downstream off-ramps from the incident location, and the connecting parallel arterial. To ensure that the complex relations between the proposed formulations are understandable and also realistically reflect the real-world operational constraints, the corridor control model uses the following assumptions:

- Traffic is diverted to the arterial through the off-ramp just upstream of the incident section and will be guided back to the freeway. The compliance rate of drivers is assumed to be obtainable from on-line surveillance systems deployed in the control area;
- Normal traffic patterns are assumed to be stable and not impacted by the detour traffic, or the impact can be estimated; and

- All intersections in the arterial are assumed to have a common cycle and phase sequence.

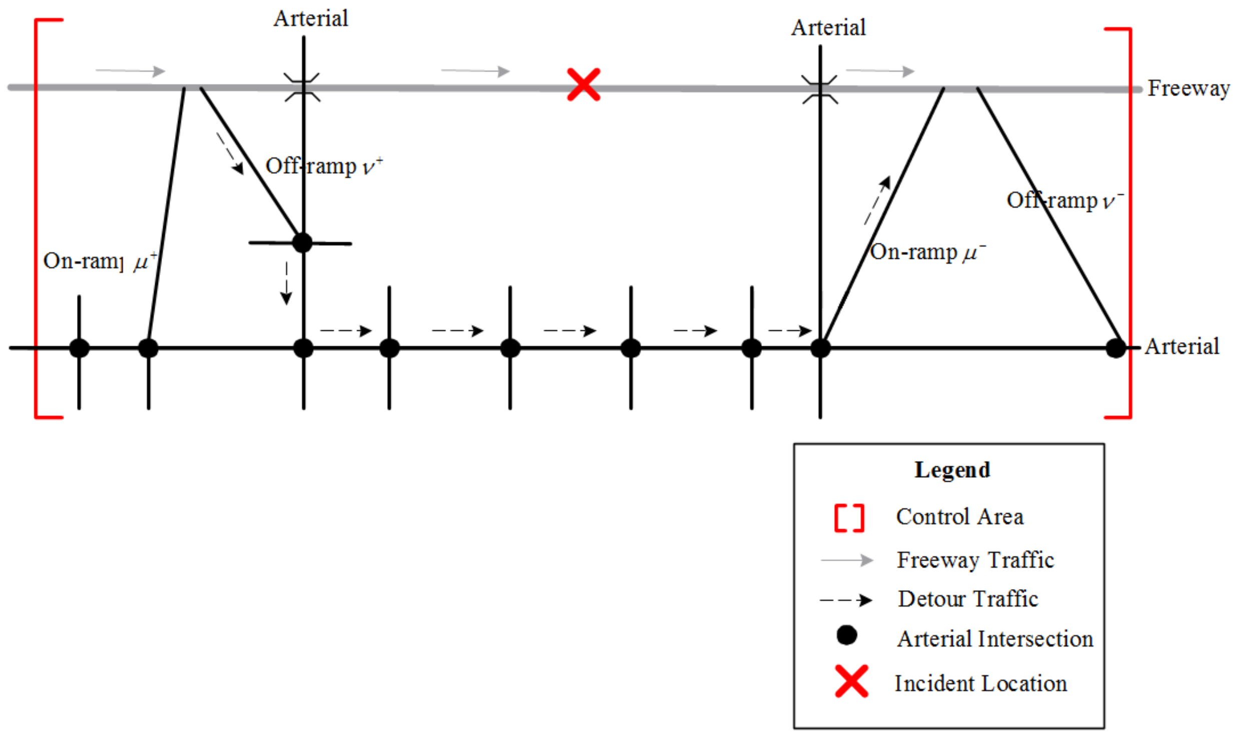


Figure 4-1: Graphical illustration of the corridor control network

Arterial Flow Dynamics

The module for modeling arterial traffic flow dynamics has six components: demand entries, upstream arrivals, joining the end of queue, merging into lane groups, departing process, and flow conservation. Figure 4-2 shows an example of a typical traffic evolution process on the internal arterial link. The proposed model has the following key features:

- Tracking the evolution of detour traffic along the arterial and its impact on each movement;
- Capturing the evolution of physical queues with respect to the signal status, arrivals, and departures;
- Modeling the merging and splitting of vehicle movements at intersections; and
- Capturing local bottlenecks such as overflows and blockages caused by dramatic changes in demand levels and patterns due to diversion operations.

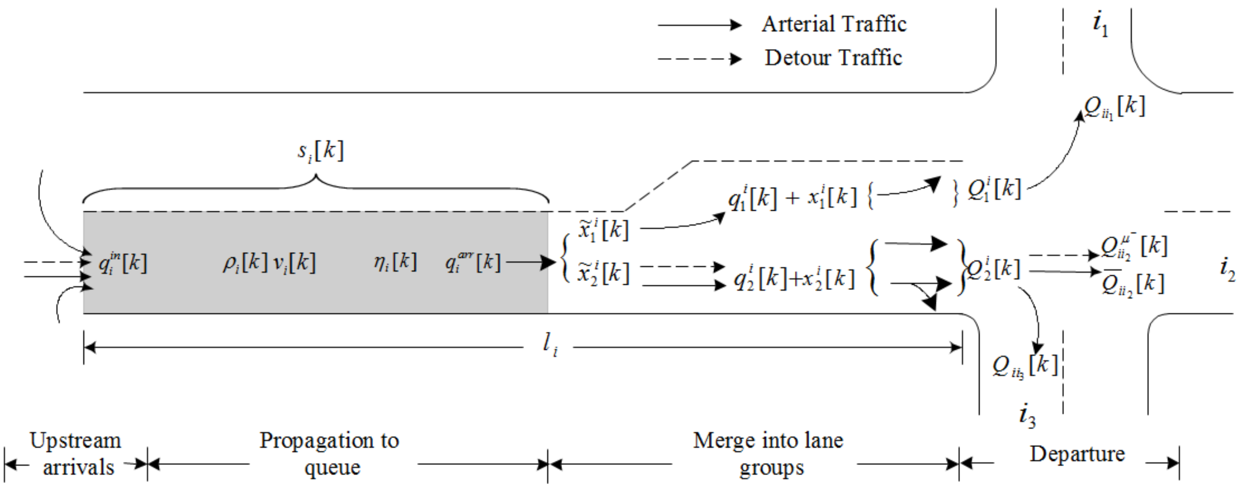


Figure 4-2: Evolution of detour traffic flows on an internal arterial link

To facilitate the presentation, the notations used hereafter are summarized in Table 4-1.

Table 4-1: List of key variables used in the traffic model for arterial dynamics

Δt	Update interval of arterial dynamics (in seconds)
k	Time step index corresponds to time $t = k\Delta t$
$n, n \in S_N$	Index of arterial intersections
$i, i \in S^U$	Index of links
S^{OUT}	Set of outgoing boundary links in the arterial
μ^+, ν^+	Index of the incident upstream on-ramp and off-ramp, respectively (see Figure 4-1)
μ^-, ν^-	Index of the incident downstream on-ramp and off-ramp, respectively (see Figure 4-1)
$p, p \in P_n$	Index of signal phase at the intersection n
S_r	Set of traffic demand entries in the arterial network
$\Gamma(i), \Gamma^{-1}(i)$	Set of upstream and downstream links of link i
l_i, n_i, N_i, Q_i	Length (in meters), # of lanes, storage capacity (in vehs), and discharge capacity (in vph) of link i
$m, m \in S_i^M$	Index of lane groups at link i
δ_m^{ij}, $j \in \Gamma^{-1}(i)$	A binary indicating whether the movement from link i to j uses lane group m
N_m^i, Q_m^i	Storage capacity (in vehs) and discharge capacity (in vph) for lane group m
$\Omega^i[k]$	Blocking matrix between lane groups at link i
$\omega_{m'm}^i[k] \in \Omega^i[k]$	Blocking coefficient between lane group m' and m at step k
$D_r[k], r \in S_r$	Flow rate generated at demand entry r at step k (in vph)
$IN_r[k], r \in S_r$	Flow rate entering the link from demand entry r at step k (in vph)
$w_r[k], r \in S_r$	Queue waiting on the entry r at step k (in vehicles)
$q_i^{in}[k]$	Number of upstream inflow vehicles of link i at step k (in vehicles)
$\overline{N}_i[k]$	Number of vehicles from normal arterial traffic at link i at step k (in vehicles)

$N_i^{\mu^-}[k]$	Number of detour vehicles heading to downstream on-ramp μ^- at link i at step k (in vehicles)
$\bar{\gamma}_{ij}[k],\ j \in \Gamma^{-1}(i)$	Relative turning proportion of normal arterial traffic from link i to j
$\gamma_{ij}^{\mu^-},\ j \in \Gamma^{-1}(i)$	A binary value indicating whether detour traffic at link i heading to downstream on-ramp μ^- will use downstream link j or not
$\eta_i[k]$	Fraction of normal arterial traffic in total traffic at link i at step k
$s_i[k]$	Available space of link i at step k (in vehicles)
$x_i[k]$	Number of vehicles in queue at link i at step k (in vehicles)
$q_i^{arr}[k]$	Number of vehicles arriving at end of queue of link i at step k (in vehicles)
$q_m^{i,pot}[k]$	Number of vehicles potentially to merge into lane group m of link i at step k (in vehicles)
$q_m^i[k]$	Number of vehicles joining the queue of lane group m at step k (in vehicles)
$x_m^i[k]$	Queue length of lane group m at link i at step k (in vehicles)
$\widetilde{x}_m^i[k]$	Number of arrival vehicles with destination to lane group m queued outside the approach lanes due to blockage at link i at step k (in vehicles)
$\lambda_m^{ij}[k],\ j \in \Gamma^{-1}(i)$	Percentage of traffic in lane group m going from link i to j
$\bar{\lambda}_m^{ij}[k],\ j \in \Gamma^{-1}(i)$	Percentage of normal arterial traffic in lane group m going from link i to j
$Q_m^i[k]$	Number of vehicles departing from lane group m at link i at step k (in vehicles)
$Q_{ij}^{pot}[k]$	Number of vehicles potentially departing from link i to link j at step k (in vehicles)
$Q_{ij}[k]$	Total flows actually departing from link i to link j at step k (in vehs)
$\overline{Q}_{ij}[k]$	Normal arterial traffic flows actually departing from link i to link j at step k (in vehicles)
$Q_{ij}^{\mu^-}[k]$	Detour traffic flows heading to downstream on-ramp μ^- actually departing from link i to link j at step k (in vehs)
$g_n^p[k]$	Binary value indicating whether signal phase p of intersection n is green or not at step k

Demand entries

Arterial demand entries are modeled as follows:

$$IN_r[k] = min\left[D_r[k] + \frac{w_r[k]}{\Delta t}, Q_i, \frac{s_i[k]}{\Delta t}\right] \tag{4-1}$$

$$w_r[k+1] = w_r[k] + \Delta t\left[D_r[k] - IN_r[k]\right] \tag{4-2}$$

Equation (4-1) indicates that the flows, from demand entry r to downstream link i, depend on the existing queue length at r, the discharge capacity of the link i, and the available space in the link i. Equation (4-2) updates the queues waiting at the demand entry during each time step.

Upstream arrivals

Formulations for upstream arrivals focus on the evolution of flows to the upstream of the link over time. Eqns. (4-3) and (4-4) define the flow dynamics for different types of links. For internal arterial links, the inflows to link i can be formulated as the sum of the actual departure flows from all upstream links, including both normal arterial and detour volumes:

$$q_i^{in}[k] = \sum_{j\in \Gamma(i)} \bar{Q}_{ji}[k] + \sum_{j\in \Gamma(i)} Q_{ji}^{\mu^-}[k] \tag{4-3}$$

For source links (connected with demand entry r), inflows can be formulated as:

$$q_i^{in}[k] = IN_r[k] \cdot \Delta t \tag{4-4}$$

Where, $\overline{Q}_{ji}[k]$ and $Q_{ji}^{\mu^-}[k]$ represent the actual flows departing from upstream link j to link i for normal arterial traffic and detour traffic, respectively.

Joining the end of queue

This module represents the evolution of upstream inflows to the end of the queues with the average approaching speed. The mean speed of vehicles, $v_i[k]$, depending on the density of the segment between the link upstream and the end of the queue, $\rho_i[k]$, can be described with the following equation:

$$v_i[k] = \begin{cases} v_i^{free}, & if \quad \rho_i[k] < \rho^{\min} \\ v^{\min} + (v_i^{free} - v^{\min}) \cdot \left[1 - \left(\dfrac{\rho_i[k] - \rho^{\min}}{\rho^{jam} - \rho^{\min}}\right)^{\alpha}\right]^{\beta} & , \rho_i[k] \in [\rho^{\min}, \rho^{jam}] \\ v^{\min}, & if \quad \rho_i[k] > \rho^{jam} \end{cases} \tag{4-5}$$

Where, $v_i[k]$ represents the mean approaching speed of vehicles from upstream to the end of the queue at link i at step k; $\rho^{\min}$ is the minimum critical density; $v^{\min}$ is the minimum traffic flow speed corresponding to the jam density (ρ^{jam}); and α, β are constant model parameters to be calibrated. The segment density from the link upstream to the end of the queue, $\rho_i[k]$, is computed with the following equation:

$$\rho_i[k] = \frac{\overline{N}_i[k] + N_i^{\mu^-}[k] - x_i[k]}{n_i(l_i - \dfrac{1000 \cdot x_i[k]}{n_i \cdot \rho^{jam}})} \tag{4-6}$$

Where, the term, $\overline{N}_i[k] + N_i^{\mu^-}[k] - x_i[k]$, represents the number of vehicles moving on the segment between the link upstream and the end of the queue, and $l_i - 1000 \cdot x_i[k]/(n_i \cdot \rho^{jam})$ depicts the queue length on that segment over time. Then, the number of vehicles arriving at the end of the queue at link i can be dynamically updated with the following expression:

$$q_i^{arr}[k] = \min\left\{\rho_i[k] \cdot v_i[k] \cdot n_i \cdot \Delta t, \overline{N}_i[k] + N_i^{\mu^-}[k] - x_i[k]\right\} \tag{4-7}$$

Where, the term, $\rho_i[k] \cdot v_i[k] \cdot n_i \cdot \Delta t$, represents the flows arriving at the end of the queue at time step *k*, which is limited by $\overline{N}_i[k] + N_i^{\mu^-}[k] - x_i[k]$.

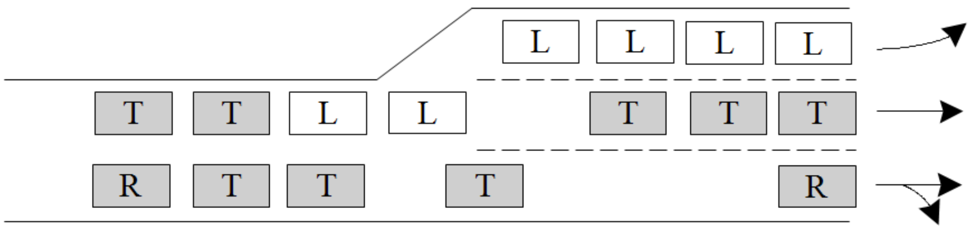
Merging into lane groups

After vehicles join a link queue, they will try to merge into different lane groups based on their destinations. Most previous studies assumed that the arriving vehicles could always merge into their destination lanes without encountering blockage. However, such an assumption may not be realistic under the following scenarios: (1) the intended lane group has no more space to accommodate arriving vehicles (e.g., a fully occupied left-turn bay); and (2) the overflowed queues from other lane groups block the target lane group (see Figure 4-3). Therefore, arriving vehicles that could not merge into their destination lane group m due to either overflows or blockage will spill back to neighboring lanes, denoted by $\tilde{x}_m^i[k]$. Note that the demand level at the intersections due to detour operations could surge to the bottleneck level. Thus, it is critical for the proposed model to capture the traffic interactions on these bottlenecks and to reflect their impacts on the design of control plans.

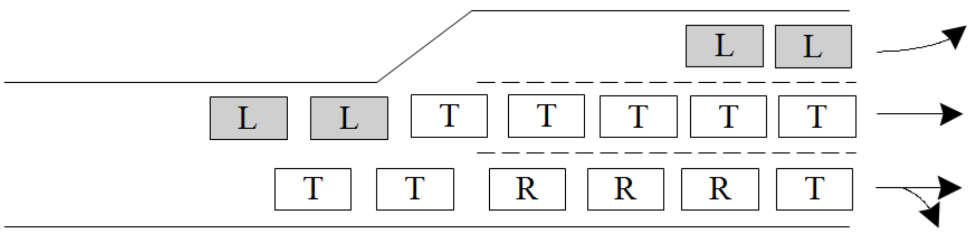
To illustrate such scenarios, it should be noted that the number of vehicles allowed to merge into lane group *m* at time step *k* depends on its available storage capacity computed as follows:

$$\max\left\{N_m^i - x_m^i[k], 0\right\} \tag{4-8}$$

Further, the blocking impacts between different lane groups can be classified into complete blockage and partial blockage (see Figure 4-3). In order to model dynamically such queue interactions between every pair of lane groups, this study defines a blocking matrix for



i) Left-turn lane group partially blocks the right-through lane group



ii) Right-through lane group completely blocks the left-turn lane group

Figure 4-3: Blockages between Lane Groups

each arterial link i, denoted by $\Omega^i[k]$. The dimension of the blocking matrix is $M_i \times M_i$, and M_i is the number of lane groups at link i. The matrix element, $\omega^i_{m'm}[k]$, takes a value between 0 and 1 to depict the blocking effect on lane group m due to the queue spillback at lane group m' at time step k. The factor, $\omega^i_{m'm}[k]$,is modeled as follows:

$$\omega^i_{m'm}[k] = \begin{cases} 1 & x^i_{m'}[k] > N^i_{m'}, complete \;\; blockage \\ \phi_{m'm} \cdot \dfrac{q^{i,pot}_{m'}[k]}{\sum_{m \in S^M_i} q^{i,pot}_m[k]} & x^i_{m'}[k] > N^i_{m'}, partial \;\; blockage \\ 0 & no \;\; blockage \;\; or \;\;\; x^i_{m'}[k] \le N^i_{m'} \end{cases} \tag{4-9a}$$

$$q^{i,pot}_m[k] = \tilde{x}^i_m[k] + \sum_{j \in \Gamma^{-1}(i)} q^{arr}_i[k] \cdot [\eta_i[k] \bar{\gamma}_{ij}[k] + (1 - \eta_i[k]) \gamma^{\mu^-}_{ij}] \cdot \delta^{ij}_m \tag{4-9b}$$

Where, $q^{arr}_i[k]$ is the total flows arriving at the end of the queues on link i at time step k; $q^{arr}_i[k] \cdot \eta_i[k] \cdot \bar{\gamma}_{ij}[k]$ represents the normal arterial traffic flow going to link j at time step k, and $q^{arr}_i[k] \cdot (1 - \eta_i[k]) \cdot \gamma^{\mu^-}_{ij}$ denotes the detour traffic flow to link j at time step k; δ^{ij}_m is a binary value indicating whether the traffic going from link i to j uses lane group m. Hence, one can approximate $\tilde{x}_m^i[k] + \sum_{j\in\Gamma^{-1}(i)} q_i^{arr}[k]\cdot[\eta_i[k]\bar{\gamma}_{ij}[k] + (1-\eta_i[k])\gamma_{ij}^{\mu^-}]\cdot\delta_m^{ij}$ as the potential level of flows that may merge into lane group m at time step k, denoted as $q_m^{i,pot}[k]$.

To ensure that the blocking matrix can effectively discriminate complete blockage from partial blockage, one can specify the blocking impact between any given pair of lane groups based on the geometric features in a target intersection approach. For example, the impact of the left-turn lane group on the right-through lane group in Figure 4-3 forms a partial blockage, while the impact of the right-through lane group on the left-turn lane group is a typical complete blockage. Thus, at each time step, the model can "understand" the blocking types and evaluate each element in the blocking matrix if any queue spillback has occurred among the target lane groups.

As shown in Equation 4-9(a), for a complete blockage or no blockage scenario, one can set $\omega_{m'm}^i[k]$ to be 1 or 0, based on the approach's geometric features.

For partial blockage, $\omega_{m'm}^i[k]$ can be approximated with $\phi_{m'm}\cdot q_{m'}^{i,pot}[k]/\sum_{m\in S_i^M} q_m^{i,pot}[k]$; where, $\phi_{m'm}$ is a constant parameter between 0 and 1 and is related to a driver's response to the lane blockage; and $q_{m'}^{i,pot}[k]/\sum_{m\in S_i^M} q_m^{i,pot}[k]$ approximates the fraction of space on the merging lanes occupied by the overflowed traffic from lane group m' at time step k.

Taking the link shown in Figure 4-3 as an example, it has two lane groups: left-turn and right-through (named as L and R-T, respectively). Therefore, the blocking matrix has a 2×2 dimension, constructed as $\begin{bmatrix} \omega_{L,L}^i[k] & \omega_{L,R-T}^i[k] \\ \omega_{R-T,L}^i[k] & \omega_{R-T,R-T}^i[k] \end{bmatrix}$.

All elements in the matrix will be updated as follows:

$$\omega_{L,L}^i[k] = \begin{cases} 1 & if \quad x_L^i[k] > N_L^i \\ 0 & if \quad x_L^i[k] \le N_L^i \end{cases}$$, *L* blocks itself;

$$\omega_{L,R-T}^i[k] = \begin{cases} \phi_{L,R-T}\cdot\dfrac{q_L^{i,pot}[k]}{q_L^{i,pot}[k] + q_{R-T}^{i,pot}[k]}, & if \quad x_L^i[k] > N_L^i \\ 0 & if \quad x_L^i[k] \le N_L^i \end{cases}$$, *L* partially blocks *R-T*;

$$\omega^{i}_{R-T,L}[k]=\begin{cases}1 & if \quad x^{i}_{R-T}[k]>N^{i}_{R-T}\\0 & if \quad x^{i}_{R-T}[k]\le N^{i}_{R-T}\end{cases}$$, *R-T* completely blocks *L*;

$$\omega^{i}_{R-T,R-T}[k]=\begin{cases}1 & if \quad x^{i}_{R-T}[k]>N^{i}_{R-T}\\0 & if \quad x^{i}_{R-T}[k]\le N^{i}_{R-T}\end{cases}$$, *R-T* blocks itself;

Considering the lane blockage impact represented by the matrix, the number of vehicles allowed to merge into lane group m at time step k is restricted by the following expression:

$$\max\left\{q^{i,pot}_{m}[k]\cdot\left[1-\sum_{m'\in S^{M}_{i}}\omega^{i}_{m'm}[k]\right],0\right\} \tag{4-10}$$

Where, according to the definition of $\omega^{i}_{m'm}[k]$, $1-\sum_{m'\in S^{M}_{i}\wedge m'\neq m}\omega^{j}_{m'm}[k]$ is the residual fraction of the capacity to accommodate those vehicles merging to lane group m.

Finally, the number of vehicles that are allowed to merge into lane group m at time step k should be the minimum value of Eqns. (4-8) and (4-10), as shown below:

$$q^{i}_{m}[k]=\min\left\{\max\{N^{i}_{m}-x^{i}_{m}[k],0\},\max\left\{q^{i,pot}_{m}[k]\cdot\left[1-\sum_{m'\in S^{M}_{i}}\omega^{i}_{m'm}[k]\right],0\right\}\right\} \tag{4-11}$$

Departing process

The number of vehicles potentially departing from link i to link j at time step k can be represented with the following expressions:

$$Q^{pot}_{ij}[k]=\sum_{m\in S^{M}_{i}}\min\left\{q^{i}_{m}[k]+x^{i}_{m}[k],Q^{i}_{m}\cdot g^{p}_{n}[k]\right\}\cdot\lambda^{ij}_{m}[k] \tag{4-12}$$

And, $\lambda^{ij}_{m}[k]$ can be estimated by:

$$\lambda^{ij}_{m}[k]=\frac{\delta^{ij}_{m}\cdot[\eta_{i}[k]\bar{\gamma}_{ij}[k]+(1-\eta_{i}[k])\gamma^{\mu^{-}}_{ij}]}{\sum_{j\in\Gamma^{-1}(i)}\delta^{ij}_{m}\cdot[\eta_{i}[k]\bar{\gamma}_{ij}[k]+(1-\eta_{i}[k])\gamma^{\mu^{-}}_{ij}]} \tag{4-13}$$

Where, $\min\{q_m^i[k]+x_m^i[k], Q_m^i \cdot g_n^p[k]\}$ depicts the flows potentially departing from lane group m at time step k; $\lambda_m^{ij}[k]$ is the percentage of traffic in lane group m from link i to j. Therefore, $\min\{q_m^i[k]+x_m^i[k], Q_m^i \cdot g_m^p[k]\} \cdot \lambda_m^{ij}[k]$ reflects the flows potentially departing from link i to j in lane group m, and its summation of overall lane groups in link i is shown in Eq. (4-12).

Assuming that one group of flows is to depart from link i at time step k, $[\eta_i[k]\bar{\gamma}_{ij}[k]+(1-\eta_i[k])\cdot\gamma_{ij}^{\mu^-}]$ will be the number of flows within that one group to go to link j, and $\delta_m^{ij}\cdot[\eta_i[k]\bar{\gamma}_{ij}[k]+(1-\eta_i[k])\cdot\gamma_{ij}^{\mu^-}]$ will be the total flows going to link j by lane group m, and $\sum_{j\in\Gamma^{-1}(i)}\delta_m^{ij}\cdot[\eta_i[k]\bar{\gamma}_{ij}[k]+(1-\eta_i[k])\cdot\gamma_{ij}^{\mu^-}]$ will be the total flows departing from lane group m.

Hence, $\lambda_m^{ij}[k]$ can be approximated with Eq. (4-13).

Similarly, the percentage of normal arterial traffic volume in lane group m moving from link i to j, $\bar{\lambda}_m^{ij}[k]$, can be approximated by Eq. (4-14):

$$\bar{\lambda}_m^{ij}[k]=\frac{\delta_m^{ij}\cdot\eta_i[k]\cdot\bar{\gamma}_{ij}[k]}{\sum_{j\in\Gamma^{-1}(i)}\delta_m^{ij}\cdot[\eta_i[k]\bar{\gamma}_{ij}[k]+(1-\eta_i[k])\gamma_{ij}^{\mu^-}]} \tag{4-14}$$

Then, the percentage of detour traffic in lane group m from link i to j can be obtained with $(\lambda_m^{ij}[k]-\bar{\lambda}_m^{ij}[k])$.

In addition to Eq. (4-12), the actual number of vehicles departing from link i to link j at time step k is also constrained by the available storage space of their destination link j. Since the total flow to one destination link j may consist of several flows from different upstream links, this study assumes that the free storage space of link j allocated to accommodate upstream flows is proportional to its potential departing flows. Therefore, the actual departing flows from link i to link j at time step k is given by the following equation:

$$Q_{ij}[k]=\min\left\{Q_{ij}^{pot}[k], \frac{Q_{ij}^{pot}[k]}{\sum_{i\in\Gamma(j)}Q_{ij}^{pot}[k]}\cdot s_j[k]\right\} \tag{4-15}$$

Where, $s_j[k]$ is the available space in link j at time step k, and $\frac{Q_{ij}^{pot}[k]}{\sum_{i\in\Gamma(j)} Q_{ij}^{pot}[k]}$ is the proportion of the available space in link j allocated to accommodate flows from link i.

Then, the flows actually departing from lane group m can be easily obtained by:

$$Q_m^i[k] = \sum_{j\in\Gamma^{-1}(i)} Q_{ij}[k]\cdot\delta_m^{ij} \tag{4-16}$$

Finally, the actual departing flows, which are not part of the detour traffic from link i to link j at time step, k are given by:

$$\overline{Q}_{ij}[k] = \sum_{m\in S_i^M} Q_m^i[k]\cdot\overline{\lambda}_m^{ij}[k] \tag{4-17}$$

The actual departing flows, from the detour traffic, from link i to link j heading to on-ramp μ^- at time step k are given by:

$$Q_{ij}^{\mu}[k] = \sum_{m\in S_i^M} Q_m^i[k]\cdot(\lambda_m^{ij}[k]-\overline{\lambda}_m^{ij}[k]) \tag{4-18}$$

Flow conservation

Note that in the dynamic evolution process, the lane-group based queues are advanced as follows:

$$x_m^i[k+1] = x_m^i[k] + q_m^i[k] - Q_m^i[k] \tag{4-19}$$

Queues outside the approach lanes due to overflows or blockages are advanced as follows:

$$\widetilde{x}_m^i[k+1] = \widetilde{x}_m^i[k] - q_m^i[k] + \sum_{j\in\Gamma^{-1}(i)} q_i^{arr}[k]\cdot[\eta_i[k]\overline{\gamma}_{ij}[k] + (1-\eta_i[k])\gamma_{ij}^{\mu^-}]\cdot\delta_m^{ij} \tag{4-20}$$

Then, the total number vehicles queued at link i can be estimated as follows:

$$x_i[k+1] = \sum_{m\in S_i^M}\left(x_m^i[k+1] + \widetilde{x}_m^i[k+1]\right) \tag{4-21}$$

The evolution of the total number of normal arterial vehicles at link i can be stated as:

$$\overline{N}_i[k+1] = \overline{N}_i[k] + \sum_{j\in\Gamma(i)}\overline{Q}_{ji}[k] - \sum_{j\in\Gamma^{-1}(i)}\overline{Q}_{ij}[k] \tag{4-22}$$

The evolution of the total number of detour vehicles present at link i can be stated as:

$$N_i^{\mu^-}[k+1] = N_i^{\mu^-}[k] + \sum_{j\in\Gamma(i)} Q_{ji}^{\mu^-}[k] - \sum_{j\in\Gamma^{-1}(i)} Q_{ij}^{\mu^-}[k] \tag{4-23}$$

The time-varying fraction of normal arterial traffic volume at link i can be updated as follows:

$$\eta_i[k+1] = \frac{\overline{N}_i[k+1]}{\overline{N}_i[k+1] + N_i^{\mu^-}[k+1]} \tag{4-24}$$

Finally, the available storage space of link i can be computed as follows:

$$s_i[k+1] = N_i - \overline{N}_i[k+1] - N_i^{\mu^-}[k+1] \tag{4-25}$$

Freeway and Ramps

The macroscopic traffic flow model proposed by Messmer and Papageorgiou (1995) was used in this study to model the freeway traffic evolution. Its key concept is to divide the freeway link into homogeneous segments and update the flow, density, and speed within each segment at every time interval (ΔT). A detailed description of the formulations is available in the literature.

Since on-ramps and off-ramps are to exchange diversion flows between the freeway and arterial, this study has used the lane-group-based concept to model their interactions. As illustrated in Figure 4-4, the on-ramp μ^- can be modeled as a simplified arterial link with only one lane group and one downstream link. The only difference between an on-ramp and an arterial link is the departing process.

Since the update step for freeway (ΔT) is usually larger than the one for arterial (Δt), this study has used the approach by Van den Berg (2001) to keep consistency between the indices of time steps for the two systems (t is the time index for the freeway and k is for the arterial, and $k = l \cdot t$, $l = \Delta T / \Delta t$). Therefore, the actual flow that departs from on-ramp μ^- to the freeway at time step k between $l \cdot t$ and $l \cdot (t+1) - 1$ is given by:

$$Q_{\mu^-}[k] = \frac{Q_{\mu^-}[t] \cdot \Delta T}{l}, \qquad \forall k \in [l \cdot t, l \cdot (t+1) - 1] \tag{4-26}$$

and

$$Q_{\mu^-}[t] = \min\left(\frac{x^{\mu^-}[l \cdot t] + \sum_{k=lt}^{l(t+1)-1} q_{\mu^-}^{arr}[k]}{\Delta T}, Q_{\mu^-} \cdot R_{\mu^-}, Q_{\mu^-} \cdot \min[1, \frac{\rho^{jam} - \rho_{i+1,0}[t]}{\rho^{jam} - \rho_i^{crit}}] \right) \tag{4-27}$$

Where, $x^{\mu^-}[l \cdot t] + \sum_{k=lt}^{l(t+1)-1} q_{\mu^-}^{arr}[k]$ is the potential number of vehicles to merge to the freeway mainline from on-ramp μ^- at the update time step *t*; R_{μ^-} is the metering rate at on-ramp μ^-; Q_{μ^-} is the discharge capacity of on-ramp μ^-; ρ^{jam} is the jam density for freeway, $\rho_{i+1,0}[t]$ is the density of the freeway segment immediately downstream from the on-ramp μ^-, and ρ_i^{crit} is the critical density of freeway link *i* where the incident occurs.

Similarly, the off-ramp could also be modeled as an arterial link if the upstream arrival process is modified properly, as shown in Figure 4-4. The actual flow rate that enters off-ramp v^+ at each arterial time step *k* between $l \cdot t$ and $l \cdot (t+1) - 1$ can be modeled as follows:

$$q_{v^+}^{in}[k] = \frac{q_{v^+}^{in}[t] \cdot \Delta T}{l}, \qquad \forall k \in [l \cdot t, l \cdot (t+1) - 1] \tag{4-28}$$

$$q_{v^+}^{in}[t] = \min\{\rho_{i-1,N(i-1)}[t] \cdot v_{i-1,N(i-1)}[t] \cdot n_{i-1,N(i-1)} \cdot (\gamma_{v^+}^T + \beta_{v^+}^T \cdot Z_{v^+}^T), \; Q_{v^+}, \frac{s_{v^+}[l \cdot t] + \sum_{k=lt}^{l(t+1)-1} \sum_{j \in \Gamma^{-1}(v^+)} Q_{v^+ j}[k]}{\Delta T}\} \tag{4-29}$$

Where, $\rho_{i-1,N(i-1)}[t]$, $v_{i-1,N(i-1)}[t]$, and $n_{i-1,N(i-1)}$ represent the density, speed, and number of lanes at the segment immediately upstream from the off-ramp v^+, respectively. $\gamma_{v^+}^T$ is the normal exit rate for off-ramp v^+ during control time interval T; $Z_{v^+}^T$ is the diversion control rate to be determined during the control interval T; $\beta_{v^+}^T$ is the driver compliance rate with the detour operation during control interval *T*; Q_{v^+} represents the discharge capacity of off-ramp v^+, and $s_{v^+}[l \cdot t] + \sum_{k=lt}^{l(t+1)-1} \sum_{j \in \Gamma^{-1}(v^+)} Q_{v^+ j}[k]$ is the available space at off-ramp v^+.

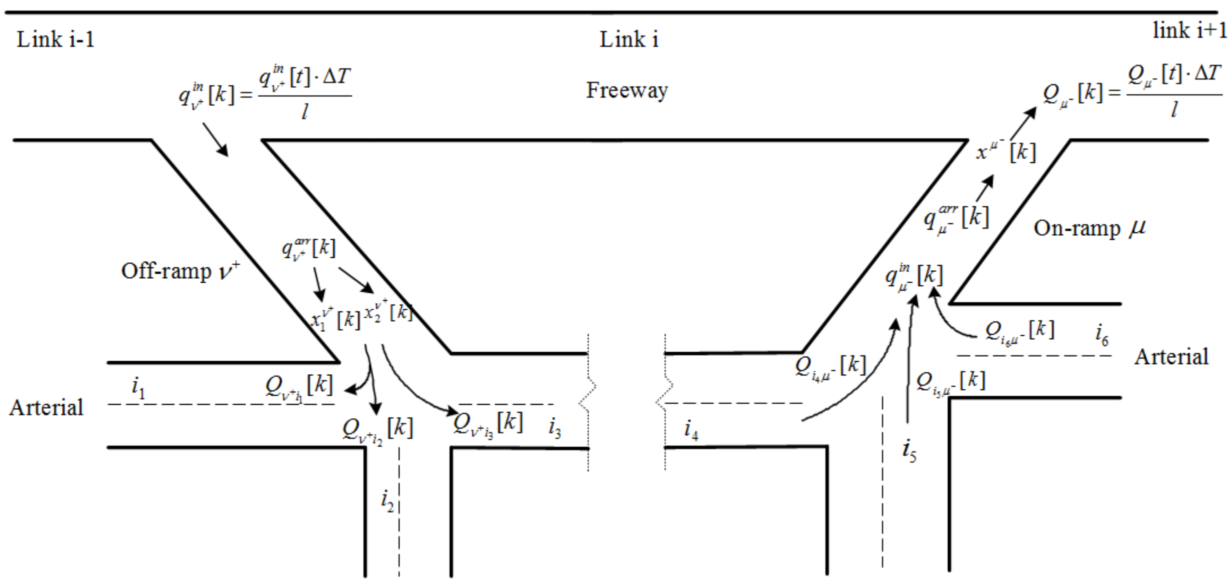


Figure 4-4: Traffic flow interactions at on- and off-ramps

It should be mentioned that the integrated control process presented in this study has the flexibility to accommodate any new formulations for freeway or arterial flow dynamics.

4.3 An Integrated Traffic Control Model

Based on the above network flow formulations, this section presents a multi-objective control model to determine the set of control strategies that can efficiently explore the control effectiveness under different policy priorities between the target freeway and the available detour route.

Objective Function

Given the control horizon *H*, the first objective of the control model is to maximize the use of the parallel arterial to relieve the freeway congestion. This objective can further be stated as maximizing the total throughput of the freeway corridor during the incident management period. The first objective function can be stated as follows:

$$\max \qquad \sum_{t=1}^{H} q_{i+1,0}[t]\cdot\Delta T + \sum_{k=1}^{H}\sum_{i\in S^{OUT}} q_i^{in}[k] \tag{4-30}$$

Where, $q_{i+1,0}[t]$ is the flow rate entering the freeway link (*i+1*) downstream from the on-ramp μ^-; S^{OUT} is the set of outgoing links in the arterial network.

The second objective function, reflecting the expectation of detour travelers, focuses on minimizing their total times on the detour route to ensure their compliance with the routing guidance. This objective is given by:

$$\min \qquad \sum_{k=1}^{H}\left[\sum_{i\in S^U} N_i^{\mu^-}[k] + N_{\nu^+}^{\mu^-}[k] + N_{\mu^-}^{\mu^-}[k]\right]\cdot \Delta t \tag{4-31}$$

Where, $N_i^{\mu^-}[k]$, $N_{\nu^+}^{\mu^-}[k]$, and $N_{\mu^-}^{\mu^-}[k]$ represent the number of detour vehicles at link *i*, off-ramp ν^+, and on-ramp μ^- within the control area at time step *k*, respectively.

Decision Variables

The control variables to be solved in the optimization formulation include:

$\{C^T, T \in H\}$: Common cycle length of the target arterial for each control interval;

$\{\Delta_n^T, \forall n \in S_N, T \in H\}$: Offset of intersection n for each control interval;

$\{G_{np}^T, \forall n \in S_N, p \in P_n, T \in H\}$: Green time for phase p of intersection n for each control interval;

$\{R_{\mu^+}^T, R_{\mu^-}^T, T \in H\}$: Metering rate at on-ramps μ^+ and μ^- for each control interval; and

$\{Z_{\nu^+}^T, T \in H\}$: Diversion rate at off-ramp ν^+ for each control interval;

Constraints

Representing the traffic state evolution along different parts of the traffic corridor, network formulations presented in previous sections constitute the principal constraints for the integrated control model. Moreover, the following constraints are common restrictions for those control decision variables:

$$C^{\min} \le C^T \le C^{\max}, \forall T \in H \tag{4-32}$$

$$G_{np}^{\min} \le G_{np}^T < C^T \quad, \forall n \in S_N, p \in P_n, T \in H \tag{4-33}$$

$$\sum_{p\in P_n} G_{np}^T + \sum_{p\in P_n} I_{np} = C^T, \forall n \in S_N, p \in P_n, T \in H \tag{4-34}$$

$$0 \le \Delta_n^T < C^T \quad , \quad \forall n \in S_N, T \in H \tag{4-35}$$

$$R^{\min} \leq R^{T}_{\mu^+}, R^{T}_{\mu^-} \leq R^{\max}, T \in H \tag{4-36}$$

$$\beta^{T}_{\nu^+} \cdot Z^{T}_{\nu^+} + \gamma^{T}_{\nu^+} \leq Z^{\max}, T \in H \tag{4-37}$$

Where, $C^{\min}$ and $C^{\max}$ are the minimum and maximum for the cycle length, respectively; P_n is the set of signal phases at intersection n; $G^{\min}_{np}$ is the minimal green time for phase p of intersection n; and I_{np} represents the clearance time for phase p of intersection n; $R^{\min}$ and $R^{\max}$ are the minimum and maximum metering rates at on-ramps, and $Z^{\max}$ is the maximum percentage of traffic that can diverge from the freeway to arterial.

Eq. (4-32) restricts the common cycle length to be between the minimal and maximal values. Eq. (4-33) requires that the green time for each phase should at least satisfy the minimal green time, but not exceed the cycle length. The sum of green times and clearance times for all phases at intersection n should be equal to the cycle length (see Eq. (4-34)).

Furthermore, the offset of intersection n will be constrained by Eq. (4-35), and lie between 0 and the cycle length. Eq. (4-36) limits the metering rates for on-ramps, and the diversion rate is bounded by Eq. (4-37).

Note that, the arterial traffic flow equations are not explicitly related to the signal control variables C^T, Δ^T_n, and G^T_{np}. To represent the signal status of phase p at each time step k, the binary variable $g^p_n[k]$ is used to indicate whether or not the corresponding phase p is green. For a signal controller with a set of phases P_n shown in Figure 4-5, the following equations can be used to model the relations between the phase status at time step k and signal control parameters:

$$(\delta'^{p}_{n}[k]-0.5)\cdot \mathrm{mod}(k-\Delta^T_n, C^T) \leq (\delta'^{p}_{n}[k]-0.5)\cdot \sum_{j=1}^{p-1}(G^T_{nj}+I_{nj}) \\ p\in P_n, n\in S_N, T\in H, k\in T \tag{4-38}$$

$$(\delta''^{p}_{n}[k]-0.5)\cdot \mathrm{mod}(k-\Delta^T_n, C^T) > (\delta''^{p}_{n}[k]-0.5)\cdot \left(\sum_{j=1}^{p-1}(G^T_{nj}+I_{nj})+G^T_{np}\right) \\ p\in P_n, n\in S_N, T\in H, k\in T \tag{4-39}$$

$$g^p_n[k] = 1-\delta'^{p}_{n}[k]-\delta''^{p}_{n}[k], \qquad p\in P_n, n\in S_N, k\in T \tag{4-40}$$

$\{\delta'^{p}_{n}[k], \delta''^{p}_{n}[k], p\in P_n, n\in S_N, k\in T\}$ are a set of auxiliary 0-1 variables.

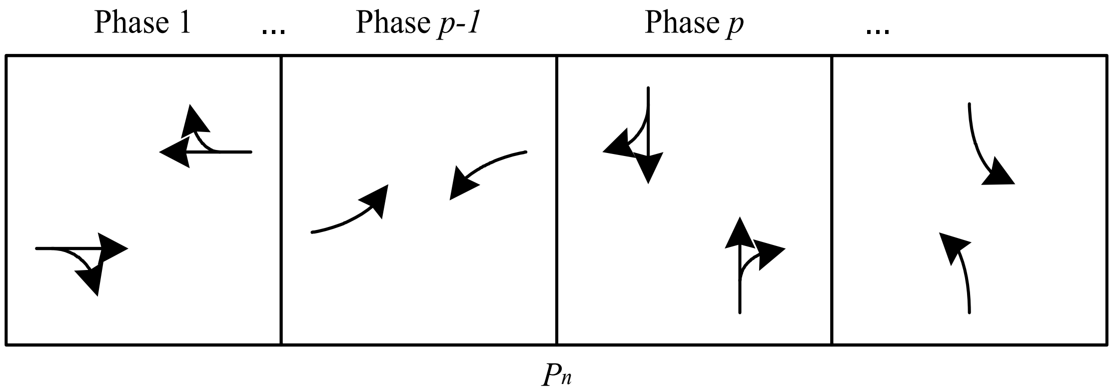


Figure 4-5: A signal controller with a set of phases P_n

Other constraints include nonnegative constraints and initial values of the link's state variables in the corridor network, which can be obtained from the on-line surveillance system to reflect the network condition preceding the onset of an incident. The mathematical description of the integrated corridor control can be summarized as follows:

$$\min \qquad \Phi(s) = \begin{bmatrix} f_1(s) \\ f_2(s) \end{bmatrix}$$

$$f_1 : -\left[\sum_{t=1}^{H} q_{i+1,0}[t] \cdot \Delta T + \sum_{k=1}^{H} \sum_{i \in S^{OUT}} q_i^{in}[k] \right]$$

$$f_2 : \sum_{k=1}^{H} \left[\sum_{i \in S^U} N_i^{\mu^-}[k] + N_{\nu^+}^{\mu^-}[k] + N_{\mu^-}^{\mu^-}[k] \right] \cdot \Delta t$$

$$s.t. \quad s : [C^T, \Delta_n^T, G_{np}^T, R_{\mu^+}^T, R_{\mu^-}^T, Z_{\nu^+}^T] \in S \tag{4-41}$$

Where, S denotes the feasible set defined by the network flow constraints and operational constraints.

4.4 Solution Method

Considering the large number of decision variables over different control intervals, application of the proposed large-scale, non-linear, and multi-objective control model is challenging. Also, solving such a large-scale control system requires a reliable projection of traffic conditions over the entire control horizon, which is also difficult due to the expected fluctuation of traffic flows and the discrepancy of driver responses to the control strategies under non-recurrent congestion. To contend with the above critical issues, this study uses a rolling-horizon method for the GA-based heuristic, in which the model input and control strategy can be regularly updated to improve the computing efficiency and effectiveness under time-varying traffic conditions and potential system disturbance.

This section first details the key components in the GA-based heuristic and then illustrates the rolling-horizon framework.

The GA-based Heuristic

The GA-based approach includes the following key steps:

Objective function normalization

Note that the first objective of the corridor model computes the number of vehicles, whereas the second objective measures the total vehicle-minutes. These two objectives cannot be directly compared or assigned weights. Hence, these two objective functions need to be normalized into a common satisfaction scale, as follows:

$$\overline{f}_m(s) = \frac{f_m(s) - f_m^{\min}}{f_m^{\max} - f_m^{\min}} \in [0,1], s \in P, m = 1,2 \tag{4-42}$$

Where, $\overline{f}_m(s)$, $f_m^{\min}$, and $f_m^{\max}$ are the normalized, minimum, and maximum value of objective function m.

Regret value computation

At each population in the evolution process of GA, the algorithm evaluates the performance of an individual solution by defining a regret value r, as follows:

$$r(s) = \left(\sum_{m=1}^{M} w_m \cdot \left| \overline{f}_m(s) - \overline{f}_m^* \right|^M \right)^{1/M}, s \in P \tag{4-43}$$

Where, $s \in P$ represents the solution s in the current population P; $w_m (m = 1 \cdots M)$ is the weight assigned to objective function m to emphasize its degree of importance; $\overline{f}_m(s)$ denotes the value of normalized objective function m corresponding to solution s, and $\overline{f}_m^*$ is the value of normalized objective function m at the best point, which will be zero, according to Eq. (4-42). Considering the bi-objective model proposed in this study, Eq. (4-43) can be specified as:

$$r(s)=\sqrt{w_1\cdot\left(\frac{f_1(s)-f_1^{\min}}{f_1^{\max}-f_1^{\min}}\right)^2+w_2\cdot\left(\frac{f_2(s)-f_2^{\min}}{f_2^{\max}-f_2^{\min}}\right)^2},s\in P \tag{4-44}$$

Proxy ideally best point

Note that $f_m^{\min}$ and $f_m^{\max}$ in Eq. (4-44) for the proposed problem are difficult to obtain, as is the regret value. Thus, this study adopted a concept of proxy ideally best point to replace the real one. The proxy ideally best point is the best point corresponding to the current generation but not to the given problem, so it is easily obtained at each generation. This study used $f_m^{\min}(P):\min\{f_m(s)\mid s\in P\}$ as the proxy ideally best point for objective function *m* corresponding to the current population *P*, and $f_m^{\max}$ was replaced by $f_m^{\max}(P):\max\{f_m(s)\mid s\in P\}$. During the evolution process, the proxy ideally best point will gradually evolve to the real one. Thus, Eq. (4-44) is converted into the following:

$$r(s)=\sqrt{w_1\cdot\left(\frac{f_1(s)-f_1^{\min}(P)}{f_1^{\max}(P)-f_1^{\min}(P)}\right)^2+w_2\cdot\left(\frac{f_2(s)-f_2^{\min}(P)}{f_2^{\max}(P)-f_2^{\min}(P)}\right)^2},s\in P \tag{4-45}$$

Fitness value computation

Finally, one needs to convert the regret value to the fitness value to ensure that better individuals have a better chance to evolve. For a minimization problem, the fitness value of an individual solution *s* in population *P* can be stated as:

$$eval(s)=\frac{r_{\max}-r(s)+\varepsilon}{r_{\max}-r_{\min}+\varepsilon},s\in P \tag{4-46}$$

Where, $r_{\max}$ and $r_{\min}$ denote the maximum and minimum regret value in population *P*, respectively; ε is a positive value between 0 and 1 that functions to prevent Eq. (4-46) from zero division and makes the adjustment between the fitness proportional and pure random selections.

Decoding for control variables

To generate feasible control parameters to satisfy the operational constraints, this study also used the following decoding scheme:

- *Arterial signal control variables*: According to the phase structure shown in Figure 4-5, within each control interval *T*, a total number of $NP_n + 1$ fractions
- ($\lambda_j^T, j = 1 \ldots NP_n + 1$) are generated for the controller at intersection n from decomposed binary strings, where NP_n is the number of phases at intersection n. Those $NP_n + 1$ fractions are used to code the green times, cycle length, and offsets as shown in the following equations:

$$G_{np}^T = G_{np}^{\min} + (C^T - \sum_{j \in P_n} G_{nj}^{\min} - \sum_{j \in P_n} I_{nj}) \cdot \lambda_p^T \cdot \prod_{j=1}^{p} (1 - \lambda_{j-1}^T),\ p = 1 \ldots NP_n - 1, n \in S_N \quad (4\text{-}47)$$

$$G_{np}^T = G_{np}^{\min} + (C^T - \sum_{j \in P_n} G_{nj}^{\min} - \sum_{j \in P_n} I_{nj}) \cdot \prod_{j=1}^{p} (1 - \lambda_{j-1}^T),\ p = NP_n, n \in S_N \quad (4\text{-}48)$$

$$C^T = C_{\min} + (C_{\max} - C_{\min}) \cdot \lambda_{NP}^T \quad (4\text{-}49)$$

$$\Delta_n^T = (C^T - 1) \cdot \lambda_{NP+1}^T \quad (4\text{-}50)$$

Eq. (4-49) constrains the random cycle lengths generated through the binary strings within the maximum and minimum allowable cycle lengths.

Diversion and metering rates: The following equations are used to constrain the random diversion and metering rates generated within the maximum and minimum allowable range:

$$Z_{v^+}^T = \frac{(Z^{\max} - \gamma_{v^+}^T)}{\beta_{v^+}^T} \cdot \lambda_{v^+}^T \quad (4\text{-}51)$$

$$R_{\mu^+}^T = R^{\min} + (R^{\max} - R^{\min}) \cdot \lambda_{\mu^+}^T \quad (4\text{-}52)$$

$$R_{\mu^-}^T = R^{\min} + (R^{\max} - R^{\min}) \cdot \lambda_{\mu^-}^T \quad (4\text{-}53)$$

Where, $\lambda_{v^+}^T$, $\lambda_{\mu^+}^T$ and $\lambda_{\mu^-}^T$ are the fractions generated through the decomposed binary string over each control interval.

The Rolling-horizon Approach

The rolling-horizon approach is a common practice for making decisions in a dynamic environment. One key issue for using a rolling-horizon framework in traffic control is to keep the consistency between the variation of arterial signal timings and the update of the control time interval. Two types of strategies are commonly reported in the literature: 1) arterial signal timings are represented with *G/C* (green time/cycle length) ratios and updated at every constant time interval, or 2) a constant network cycle length is preset to keep consistency with the control update interval. However, some limitations embedded in these strategies may limit their applications:

- The system implementation cannot be based only on the G/C ratios for arterial intersections. It still needs an additional interface to work with a compatible microscopic local controller to compute the signal timings, green times, and offsets;
- A preset network cycle length may not be able to accommodate the traffic fluctuation under incident conditions; and
- A constant control update time interval may not be sufficiently responsive to the changes in signal control parameters, potentially causing the loss of some phases.

To address the above issues, this study used the following rolling-horizon structure (see Figure 4-6):

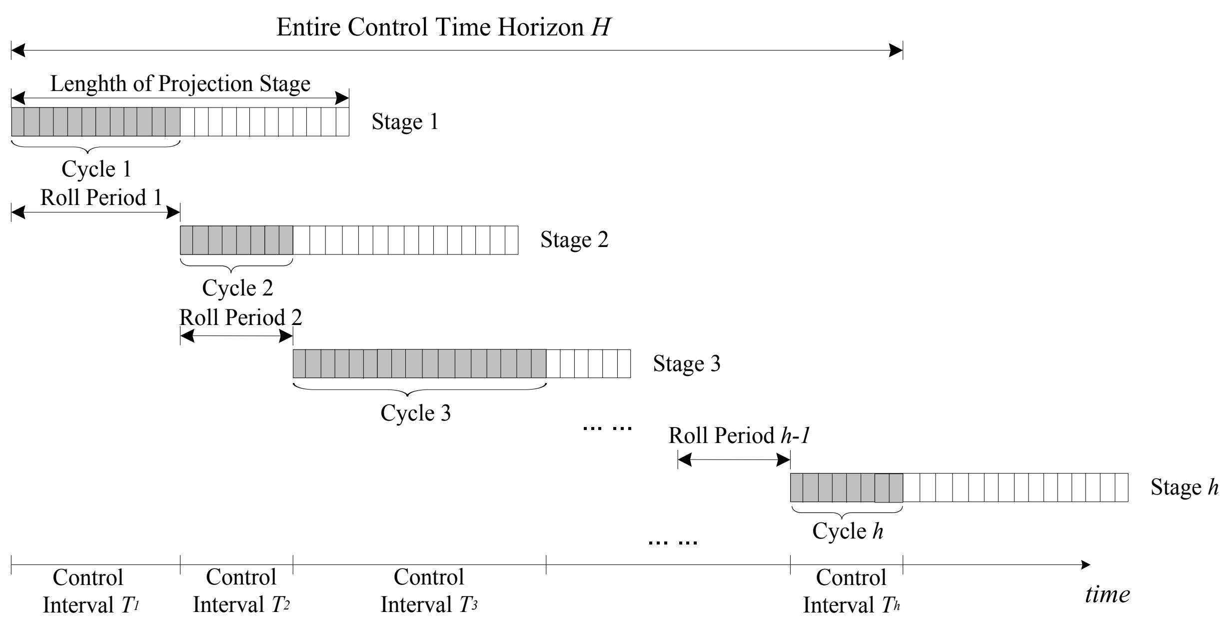


Figure 4-6: Illustration of the rolling-horizon structure

- Control policies are calculated over each projection stage, as shown in Figure 4-6, but implemented only for the control interval T (head section of each stage); and
- After implementing the control plan, the traffic state within the corridor network is updated with real-time measurements from the surveillance system; the optimization process will then begin again by shifting forward one control interval.

4.5 Numerical results

Experimental Network

To conduct the performance evaluation, this study used a corridor segment along I-95 Northbound from Washington, D.C. to Baltimore (shown in Figure 4-7) for experimental analysis. Assuming that an incident occurs on the freeway mainline segment (between node 26 and 44), traffic detouring to MD198 needs to get back to the freeway via MD216. The corridor optimization model will update the control measures, including the diversion rate at node 27, the signal timings at intersections along MD198 and 216 (nodes 68, 69, 65, 67, and 99), and the metering rate at nodes 26 and 43. The entire test period of 35 minutes covers the following three periods: five minutes for normal operations (no incident), 20 minutes with an incident, and 10 minutes for traffic recovery. The experimental analysis included the following four scenarios (see Table 4-2):

- Scenario I: Volume level-I with one lane blocked due to an incident;
- Scenario II: Volume level-I with two lanes blocked due to an incident;
- Scenario III: Volume level-II with one lane blocked due to an incident; and
- Scenario IV: Volume level-II with two lanes blocked due to an incident.

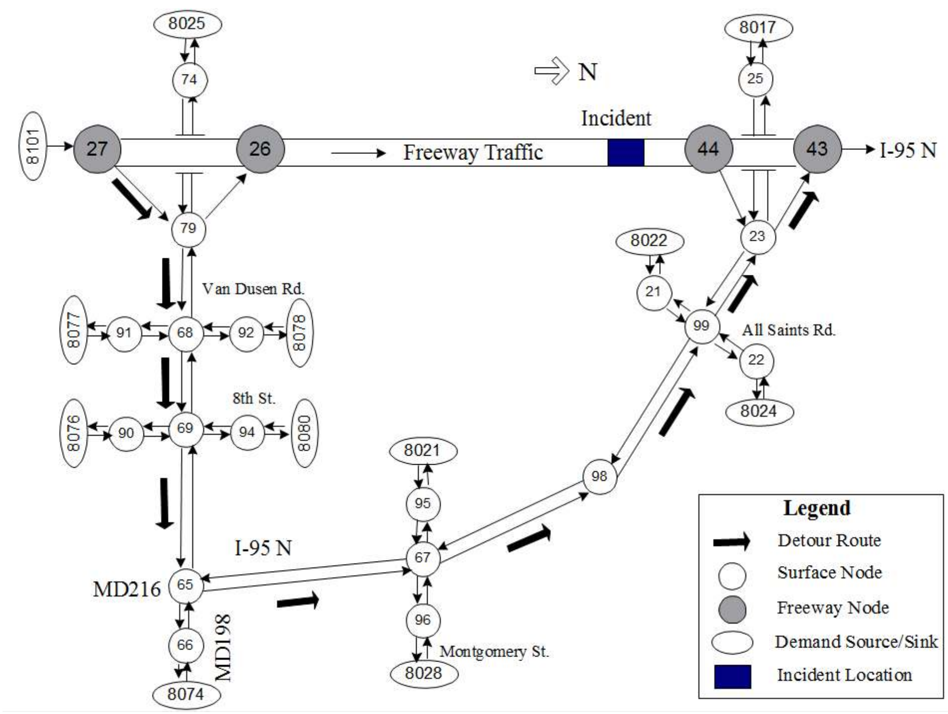


Figure 4-7: Layout of the experimental network

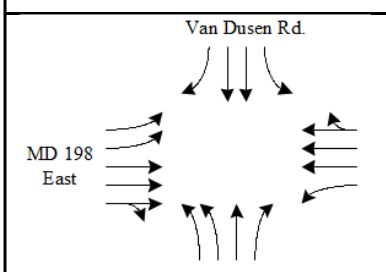
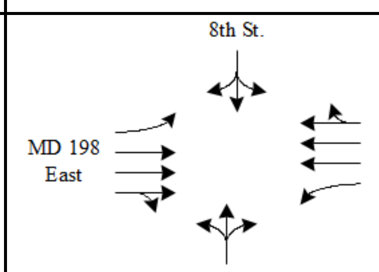
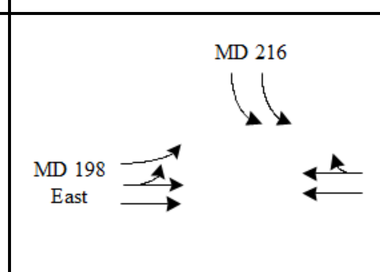
Table 4-2: Volume levels for the experimental analysis

Demand node	Level I (vph)	Level II (vph)
8101	4680	7800
8025	614	1024
8017	564	940
8077	554	924
8078	725	1208
8076	200	400
8080	210	384
8074	550	916
8021	200	400
8028	246	510
8022	187	312
8024	390	684

Key Model Parameter Settings

Within the control area shown in Figure 4-7, I-95 mainline has northbound 4 lanes. On the detour routes, the off-ramp from I-95 North to MD198 East has two lanes, and MD198 East is an arterial street with three lanes in each direction. MD216 is an arterial with two lanes in each direction, and the on-ramp from MD216 to I-95 North has one lane. The lane channelization at each intersection is shown in Table 4-3 and the phase diagram is summarized in Table 4-4.

Table 4-3: Lane channelization at intersections on the detour route

Node 68	**Node 69**	**Node 65**
		
Node 67	**Node 99**	

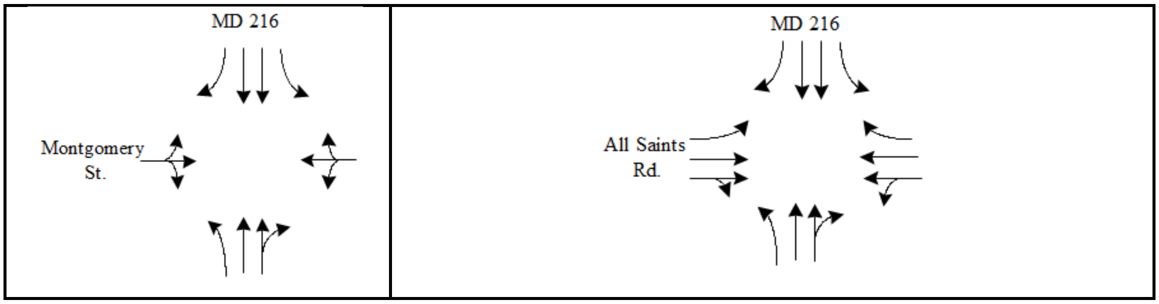
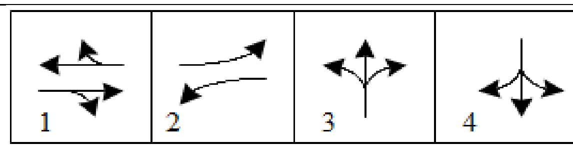
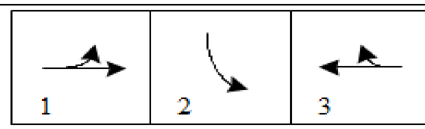
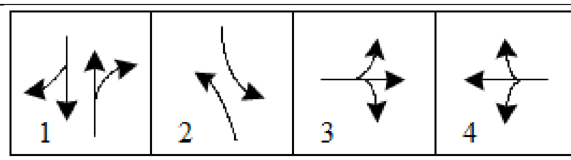


Table 4-4: Phase diagram of intersections on the detour route

Node ID	Phase Diagram
68, 69	
65	
67, 99	

All other parameters related to the network flow models, arterial signals, and solution algorithm are summarized in Table 4-5:

Table 4-5: Key model parameters used in the experimental test

Parameters*	Values
Traffic flow model parameters	
Δt , ΔT (in *secs*)	1, 5
Freeway segment length (in *ft*)	800
ρ^{jam}, $\rho^{\min}$ (in *veh/mile/lane*)	210, 20
Free flow speed at freeway, ramps, arterials (in *mph*)	65, 45, 50
Minimum speed corresponding to jam density (in *mph*)	5
Link discharge capacity for freeway, ramps, arterials (in *vplph*)	2200, 1900, 1800
Average vehicle length (in *ft*)	24
Freeway model parameters: α_f	1.78
Freeway model parameters: τ (in secs)	27
Freeway model parameters: η (in $mile^2/h$)	6
Freeway model parameters: κ (in *veh/mile/lane*)	21
Normal exiting rate at the off-ramp to MD198 East, $\gamma_{v^+}^{T}$	0.0875

Driver compliance rates with the detour operation, $\beta^{T}_{v^{+}}$ (if the detour travel time is less or comparable to the freeway travel time)	100%
Arterial signal parameters	
$C^{\min}$, $C^{\max}$ at arterial intersections (in seconds)	60, 160
$G^{\min}_{np}$, I_{np} at arterials intersections (in seconds)	7, 5
Minimum and maximum ramp metering rates: $R^{\min}$, $R^{\max}$	0.1, 1.0
Maximum diversion rate $Z^{\max}$	0.25
Parameters in solution algorithm	
Weights of importance w_1 / w_2	Assigned from $10/0$ to $0/10$ at an increment of 1.
Population size in GA	50
Maximum number of generation in GA	200
Crossover probability in GA	0.5
Mutation probability in GA	0.03
Fitness selection parameter in GA: ε	0.1
Length of the projection stage in rolling-horizon framework (in minutes)	4
Control update time interval T	one cycle length in each projection stage

Experimental Results

To evaluate the performance of the corridor optimization model, this study took the following steps:

- Step I – evaluate the model performance with systematically varied weights to provide the operational guidelines for decision makers to specify proper weights for both control objectives ;
- Step II - compare the model performance under all experimental scenarios with the following two control strategies:

No control: close the incident upstream on-ramp;

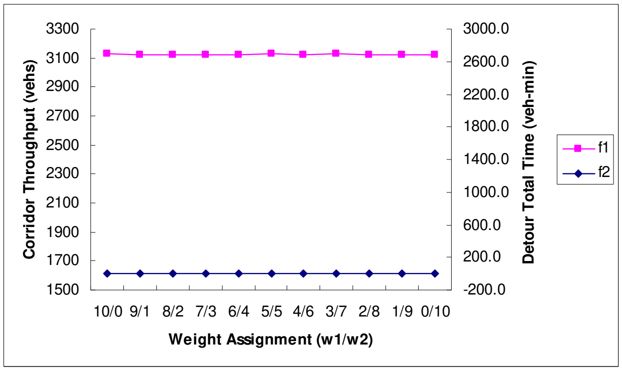
Static diversion control: determine the detour rates by a static user-equilibrium (UE) assignment between the freeway and arterial; compute the intersection signal plans with TRANSYT-7F based on the volume assignment results and operate ramp metering with ALINEA.

The microscopic simulator CORSIM was used as the unbiased performance evaluator. To overcome the stochastic nature of simulation results, an average of 30 simulation runs was used.

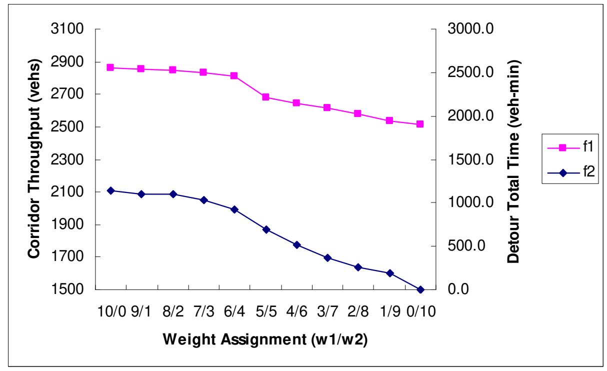
Step-I: Weight Assignment

Figure 4-8 summarizes the performance of the corridor optimization model under different scenarios and weights of importance between two control objectives. The primary findings are the following:

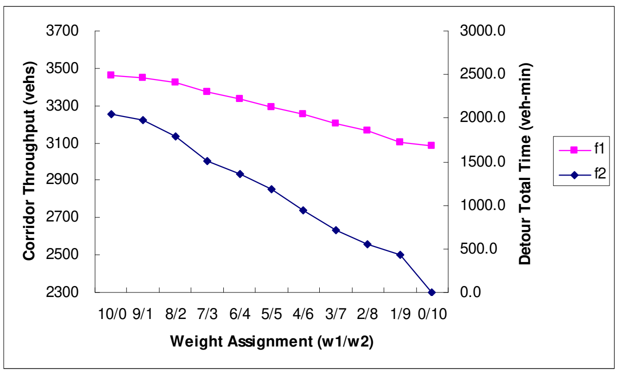
- For Scenario I, the performance of the bi-objective model is not sensitive to the weight variation, as shown in Figure 4-8a. This is likely because the remaining freeway capacity can accommodate the demand at the Volume-I level without detouring traffic. The slight fluctuation in the objective function is probably due to the convergence property of the GA algorithm;
- For Scenario II, the performance of the model seems quite stable as long as $w_1 > w_2$, as shown in Figure 4-8b. That is probably due to the fact that the under-saturated arterial can accommodate sufficient detour traffic volume as long as the freeway system is given priority. However, when $w_1 \leq w_2$, the total corridor throughput exhibits a dramatic drop (from 2808 vehicles to 2680 vehicles) due to the priority switching from the freeway to the arterial. When the arterial is given the highest priority (0/10), the corridor throughput will be at the lowest level (2512 vehicles);
- For Scenarios III and IV, the performance of the model is sensitive to every weight adjustment between two objective functions (see Figs. 4-8c and 4-8d). Every performance improvement for one objective will be at the cost of the other.



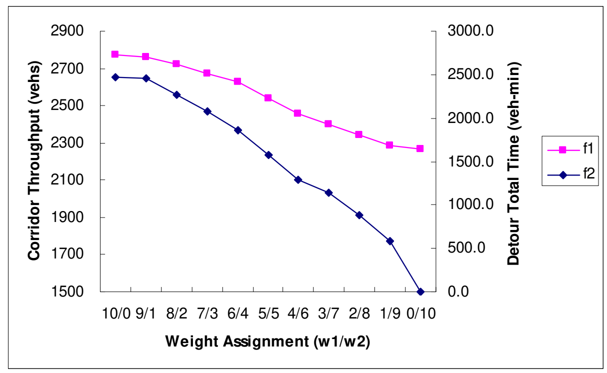
(a) Scenario I



(b) Scenario II



(c) Scenario III



(d) Scenario IV

Figure 4-8: System MOE changes under different weights

To further assist traffic operators in best weighting the importance between both control objectives, the time-varying travel time patterns were investigated on both the detour route and the freeway mainline during the control period under different scenarios (see Figure 4-9 to Figure 4-11) except Scenario I. Some interesting findings are reported below:

- With the weight assignment changing from $w_1 / w_2 = 10/0$ to $w_1 / w_2 = 0/10$, the ratio of detour travel time to freeway travel time decreases under all scenarios;
- The single control objective of maximizing the total corridor throughput (i.e. $w_1 / w_2 = 10/0$) may result in unbalanced travel times between the detour route and the freeway mainline, which could cause unacceptable driver compliance rates and degrade the control performance; and
- There is an optimal weight assignment for each scenario to achieve the target level of driver compliance rate. For example, this case study assumes a 100% driver

compliance rate if the detour travel time is less than or comparable with the freeway travel time. For Scenario I, $w_1 / w_2 = 10/0$ can be set to maximize the use of residual freeway capacity without detour operations. For Scenario II (see Figure 4-9), $w_1 / w_2 = 10/0$ can still be set to fully use the available capacity in the arterial while keeping a high level of driver compliance rates. For Scenario III (see Figure 4-10), one needs to set $w_1 / w_2 = 6/4$ or lower to ensure the acceptable driver compliance rate. Similarly, one needs to set $w_1 / w_2 = 5/5$ or lower to ensure the acceptable driver compliance rate for Scenario IV (see Figure 4-11).

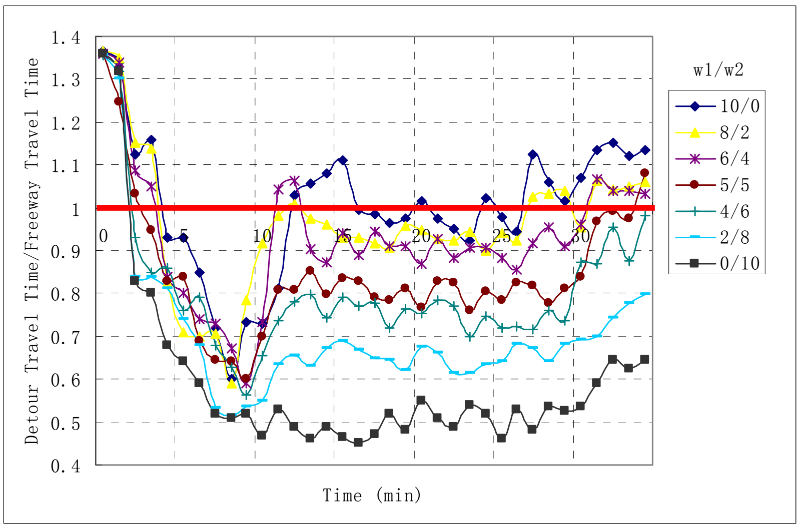


Figure 4-9: Time-varying ratio of detour travel time to freeway travel time with different weights (Scenario II)

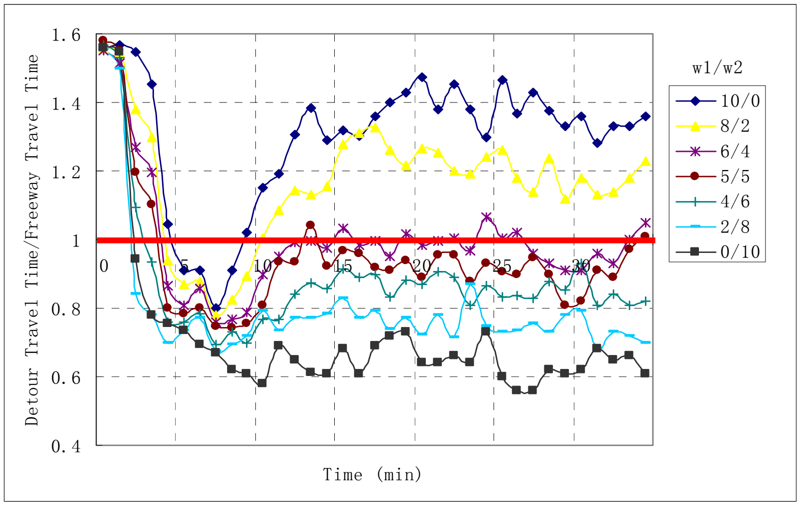


Figure 4-10: Time-varying ratio of detour travel time to freeway travel time with different weights as (Scenario III)

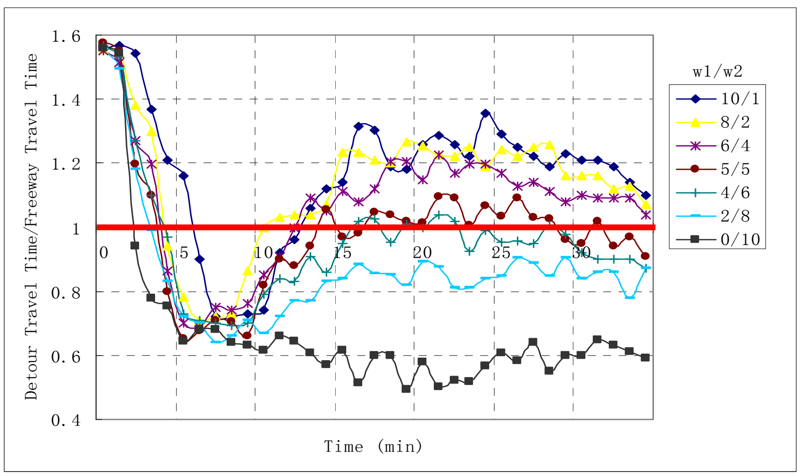


Figure 4-11: Time-varying ratio of detour travel time to freeway travel time with different weights (Scenario IV)

Step-II: performance comparison with other models

This study has also compared the performance of the corridor optimization model with other incident management strategies with respect to the total corridor throughput and the total travel time.

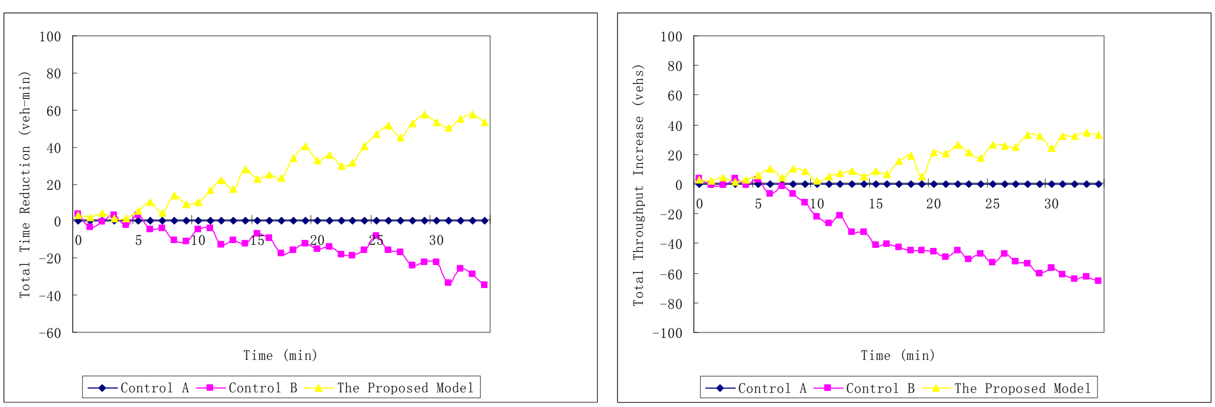
Based on the analysis results from Step I, the weights of control objectives for the four test scenarios are set as follows:

- Scenario I: $w_1 / w_2 = 10/0$;
- Scenario II: $w_1 / w_2 = 10/0$;

- Scenario III: $w_1 / w_2 = 6/4$;
- Scenario IV: $w_1 / w_2 = 5/5$;

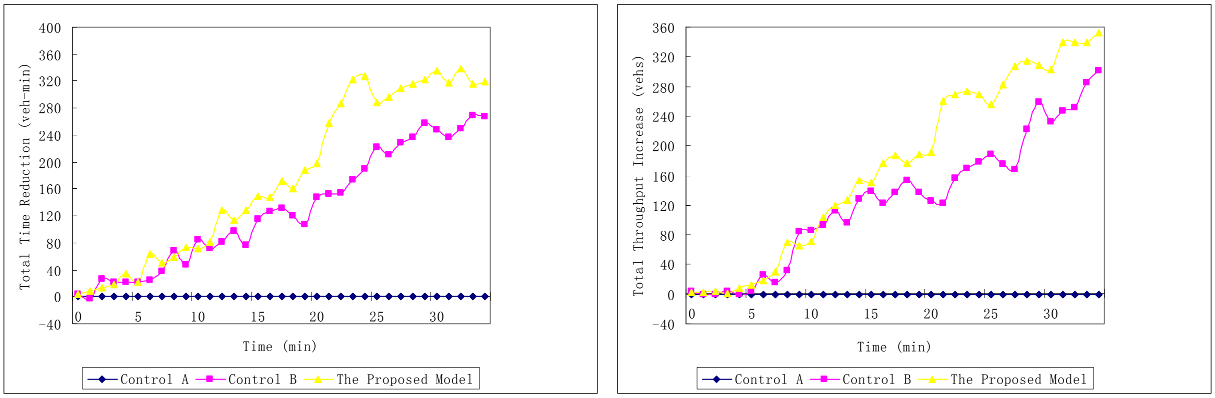
Figs. 4-12 to 4-15 illustrate the comparison results, highlighting the following findings:

- The corridor model can outperform Control A and Control B for all scenarios in terms of both total time savings and total throughput increases at the assumed level of driver compliance rates.
- In Scenario I (see Figure 4-12), since the freeway capacity can accommodate the traffic without implementing detour operations, the superior performance of the corridor optimization model over Control A is likely due to its generation of better signal timings for the arterial than with TRANSYT-7F under light traffic conditions. Control B, however, exhibits a performance inferior to Control A, due to the excessive traffic volume detoured to the arterial set by the static UE.
- In Scenario II (see Figure 4-13), the corridor model, compared with Control A, exhibits a substantial improvement since it aims to maximize the total corridor throughput ($w_1 / w_2 = 10/0$), which also results in a relatively low total travel time.
- In Scenarios III and IV (see Figs. 4-14 and 4- 15), the corridor model significantly outperforms both Control A and Control B due to its integrated control function and the embedded traffic flow equations that are capable of capturing the evolution of detour traffic along the ramps and surface streets and the resulting local bottlenecks under the saturated traffic conditions.



(a) Total Time Savings (b) Total Throughput Increases

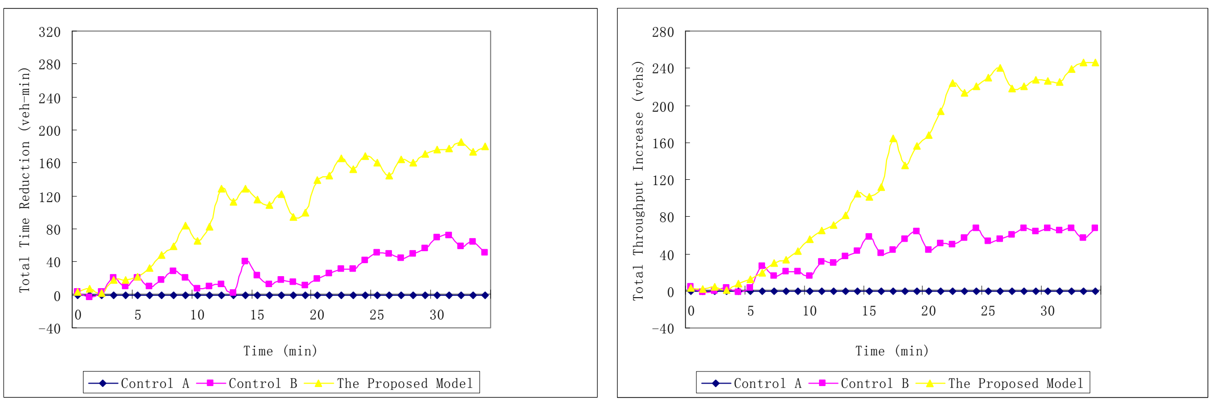
Figure 4-12: Time-varying control performance comparison (Scenario I)



(a) Total Time Savings

(b) Total Throughput Increases

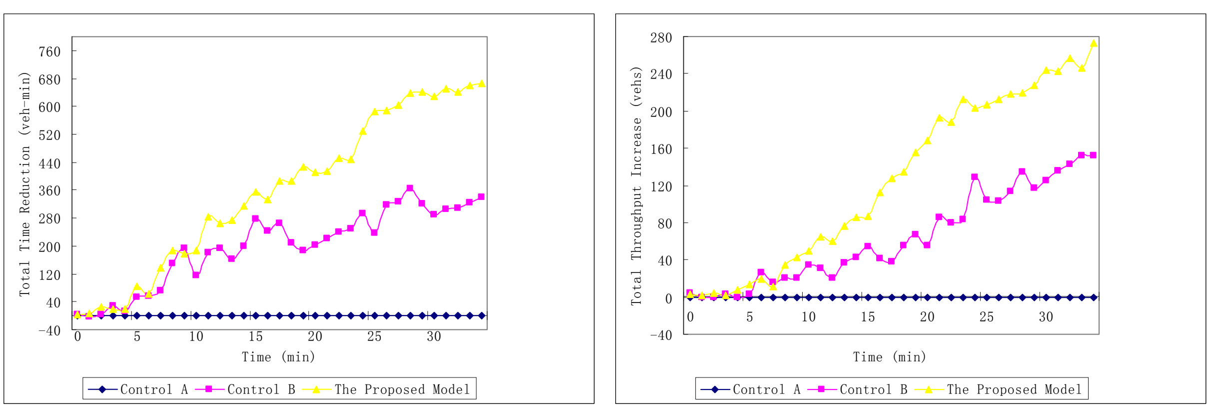
Figure 4-13: Time-varying control performance comparison (Scenario II)



(a) Total Time Savings

(b) Total Throughput Increases

Figure 4-14: Time-varying control performance comparison (Scenario III)



(a) Total Time Savings

(b) Total Throughput Increases

Figure 4-15: Time-varying control performance comparison (Scenario IV)

CHAPTER 5: An Integrated Multi-criteria Support System for Assessing Detour Decisions

5.1 Introduction

Implementing a well-designed detour plan to minimize the impact of non-recurrent congestion has long been adopted by responsible highway agencies based mainly on the estimated incident duration or the number of blocked lanes as reviewed in Chapter 2. Since an effective detour operation necessitates rigorous advanced planning and vast resources for implementation, convincing justification for such actions becomes increasingly essential in practice, especially in view of the diminishing resources for traffic management. This study presents a multi-criteria decision-support system to assist traffic managers in making such decisions, allowing them to take into account associated costs and benefits from various perspectives, such as the operational costs; the resulting benefits from reduced delay, fuel consumption, and emissions; and the likelihood of causing secondary incidents. The impact of potential driver compliance in response to the detouring strategy and the local traffic conditions on the effectiveness of detour operations can also be included in the decision process. The proposed system, with its embedded analytical hierarchical process (AHP) structure and optimal corridor detour model, allows potential users to prioritize all essential decision criteria (based on either the resource constraints or the desire of the general public) and to make the critical decision that can best manage any non-recurrent congestion while maximizing the total resulting socioeconomic benefits.

Figure 5-1 illustrates how to use the proposed detour decision-support system in the incident management process. For instance, if an incident is detected, the related traffic and incident data are collected by emergency response units and the surveillance system.

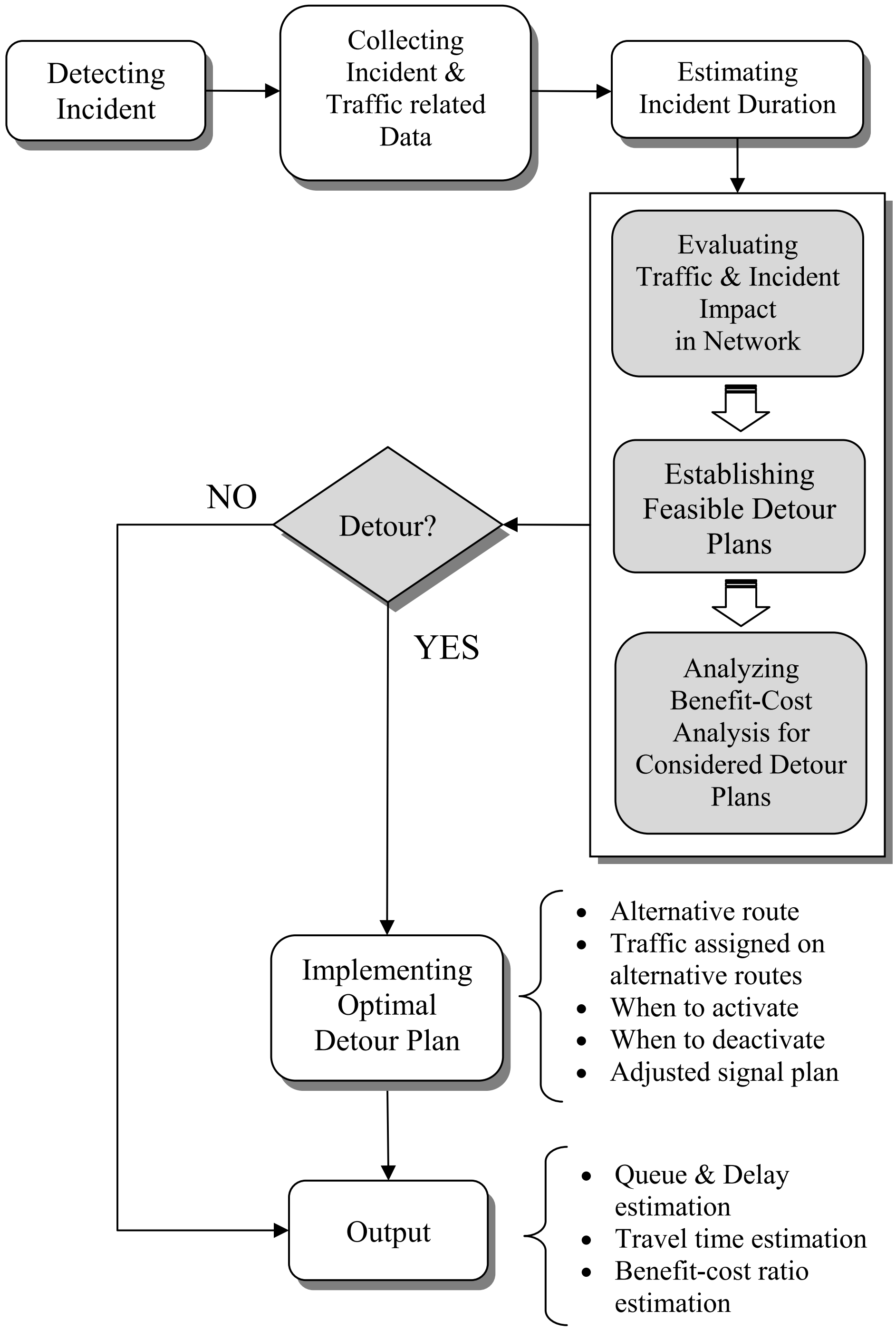


Figure 5-1: The Freeway incident traffic management system

Traffic operation managers should first estimate the incident duration with the incident duration estimation model, based on the collected traffic and incident information, and then evaluate current traffic conditions in the network plagued by the detected incident. At the same time, traffic operation managers should start considering any feasible detour plans and their resulting benefits/costs so as to make an appropriate detour decision. At this stage, the responsible agencies, if they have reliable tools, can make an efficient and effective decision.

If the need to detour traffic is confirmed, control center operators can employ the corridor optimization model to obtain the optimal detour plan that can minimize the total traffic delay. The entire procedure with the developed decision-support system could allow traffic control centers to make incident management detour decisions in an efficient and reliable manner.

The rest of this chapter presents the key logic embedded in each component of the proposed detour decision-support system.

5.2 The Detour Decision-Support System

During the incident management process, multiple factors may affect a traffic manager's final decision on whether or not to implement detour operations, such as the expected benefits and costs, impacts on traffic safety, reliability of travel, and the accessibility and acceptability of detour routes. Detour operations that fail to consider those critical factors may result in a waste of traffic management resources and the exacerbation of traffic congestion in the target corridor.

The traditional decision-making model, when it adopts multiple criteria, usually evaluates them individually in a specific directional flow. Since each criterion is evaluated independently, the importance (weight) of every criterion is identical. However, in many decision-making processes, including the detour decision process, each individual criterion may influence the final decision to a different degree, thus necessitating the prioritization of criteria.

One well-known decision-making process that considers the relative importance of criteria is the AHP developed by Saaty in the early 1970s (Saaty, 1980). The AHP provides a structured system for organizing and analyzing a complex decision problem by decomposing it into a hierarchy of more easily understandable sub-problems (i.e., decision criteria and alternatives). The various elements in the constructed hierarchy are systemically evaluated by comparing them two at a time to observe how they affect an element at a higher level of the structure. In these pair-wise comparisons, decision makers can use either tangible data or their judgments to determine the relative importance of those elements. The AHP converts these evaluations into numerical values which serve as the basis for the final stage — computing the numerical priorities of all decision alternatives to reflect their relative abilities to accomplish the decision goal.

The main advantage of the AHP is that it allows the comparison of both qualitative and quantitative criteria using informed judgments to derive their weights and priorities. Also, the AHP can assist decision makers in discovering the decision that best suits their goal and their understanding of the problem. Further discussions of the AHP are available in the references (Saaty, 1980; Saaty, 1982; Haas and Meixner, 2010; Teknomo, 2006).

Considering the nature of the proposed detour decision problem and the capabilities of the AHP, this study has developed a hybrid decision-support system by integrating the traditional decision-making model with the AHP model. The following section details the system development process.

5.2.1 System framework development

Figure 5-2 describes the overall structure of the developed detour decision system. This process should achieve the decision goal of determining whether or not executing the detour operations is beneficial compared to the anticipated costs for the operations. To reach any conclusion, one would build a procedure to systematically evaluate potential outcomes, which may either positively or negatively affect drivers, traffic networks, or environments. A step-by-step description of the overall system structure is presented below, along with its graphical illustration in Figure 5-2:

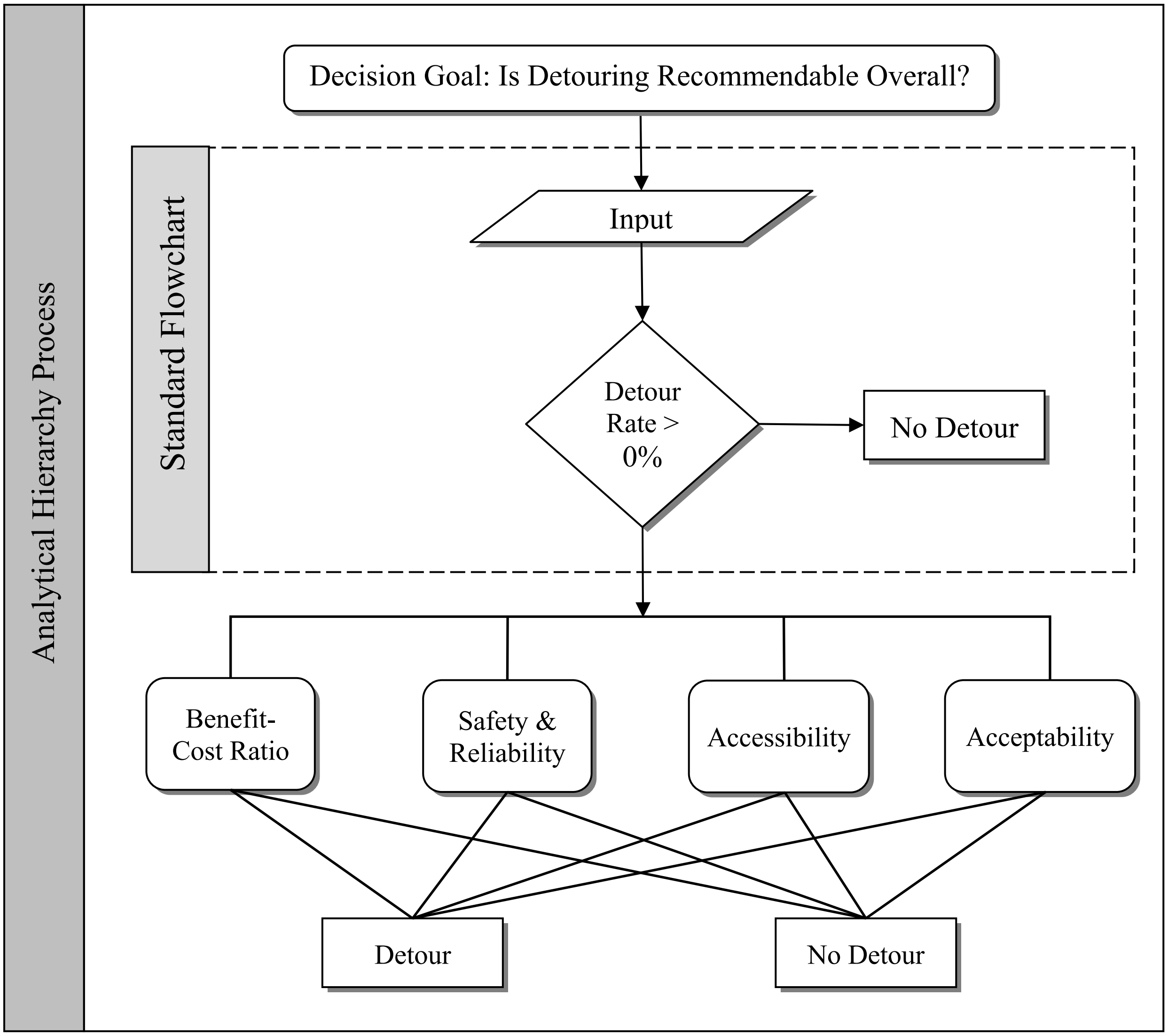


Figure 5-2: Overall structure of the proposed detour decision-support system

Step 1: The decision goal setup

The decision goal, the first level of the hierarchical system for decision makers to establish, is to determine if the proposed detour operation should be implemented with sufficient benefits to justify the operational costs.

Step 2: Model inputs by users

As discussed previously, this level and the following lower level are developed with the standard algorithm flowchart. The model variables entered at this level are used to estimate and evaluate quantitative criteria at the lower levels. At this level, users would input the key variables listed below:

- Incident information: incident duration, lanes blocked, and incident location.
- Network information: number of lanes on the primary (freeway) and detour routes, the number of signals on the detour route, and the distance of the detour path.
- Traffic information: traffic volume on the primary and detour routes, heavy vehicle volume, and speed limit for the detour route.
- Operations information: anticipated compliance rate if detour operations are implemented.

Step 3: Initial assessment for deploying the detour operation

The conditional criterion at this level is to judge the need for the detour operation under the available information, given the objective of minimizing the total delay in the entire network. If the estimated optimal detour rate turns out to be near zero (from the corridor optimization model in Chapter 4), then traffic operators can conclude that the candidate detour plan would not contribute to relieving the incident-induced congestion, and they should consider other detour plans or strategies, if available. A positive estimate for the optimal detour rate should advise the responsible operators to consider additional vital factors before coming to a final conclusion.

As shown in Figure 5-2, if the answer to the question in *Step 3* is "No," the traffic operators would terminate the decision process with "no detour"; otherwise, they would continue the process using additional criteria to reach the definitive conclusion.

Step 4: Development of additional decision criteria and their relative importance for the AHP

If the decision from the initial assessment in *Step 3* is "detour," the decision system will apply the AHP to evaluate the comprehensive impacts of other criteria before making the final decision. The standard hierarchy of the AHP model consists of three levels, with the goal at the top, alternatives at the bottom, and criteria in between. Additional levels of the hierarchy can be added if developers want to break down the criteria into subcriteria, sub-subcriteria, and so forth.

Unlike the simple criteria used in the literature (i.e., the incident duration and the number of lanes blocked), this system employs the following criteria to effectively evaluate the overall benefits of the target decision:

- **Benefits/costs**

- Benefits: total travel time (minutes/vehicle), fuel consumption, and emissions saved from detour operations;
- Costs: operational and maintenance costs to implement detour plans (converted into monetary values to facilitate comparison).

- **Safety and reliability**

Reducing traffic demand on the primary route by the diversion of traffic would alleviate the congestion around the primary incident. This result could reduce secondary incidents. Note that, to quantify such results, one can estimate one of the following MOEs (measures of effectiveness): 1) reduction in secondary incidents; 2) reduction in the probability of having secondary incidents; or 3) reduction in the congestion area (queue length) due to the detour operations. This study uses the maximum queue length on the freeway as the criterion on this aspect.

- **Accessibility**

Some factors — such as longer travel times, distances, delays at traffic signals or stop signs, and lower speed limits on the detour route — may degrade the accessibility of the detour route to travelers. To capture this nature, this study will measure the estimated travel times for the primary and alternative routes and use such information as the accessibility criteria.

- **Acceptability**

The acceptability of a detour plan significantly affects its performance. However, a plan's acceptability depends on the characteristics of drivers (e.g., risk takers, conservative or patient drivers, etc.) and the quality as well as availability of real-time traffic information. Moreover, drivers might not prefer the selected detour route due to the existence of signalized intersections, stop signs, turning movements, and queues. Thus, drivers may downgrade the acceptability of the detour plan. Considering the aforementioned scenarios, this study used drivers' anticipated compliance rate as the criterion for measuring this factor.

Usually, informed judgments by decision makers are used to derive the relative importance of the criteria. They can come from concrete measurements or experts' judgments. A core idea of the AHP methodology is to involve human judgment in the evaluation process. Informed judgments, such as "Criterion A is two times as important as Criterion B" and "Criterion B is three times as important as Criterion C" are expressed in numerical scales of measurement using a series of pair-wise comparisons. The final product from these procedures is a priority ranking of criteria against the goal. Details of the procedures for standard pair-wise comparisons, normalization, and determination of final ranking of priorities are available in the literature (Saaty, 1980; Saaty, 1982; Haas and Meixner, 2010; Teknomo, 2006).

Step 5: Determination of the relative ranking of alternatives under each criterion.

The next task of the AHP development is to determine the relative ranking of alternatives with respect to each criterion. Using the similar method to obtain the relative importance of all criteria, one can derive the preference of each alternative over one another with respect to each criterion.

Step 6: Determination of the overall relative ranking of alternatives concerning the decision goal.

Given the weights for criteria and alternatives from *Step 4* and *Step 5*, the decision makers shall be able to estimate the priorities of alternatives against the goal.

5.2.2 Supplemental models for the system

Completing the system requires several supplemental models to estimate the measurements for some quantitative criteria.

Integrated Control Model for Freeway Corridors under Non-recurrent Congestion

Liu and Chang (2011) developed an integrated control model for freeway corridors under non-recurrent congestion to produce the optimal diversion rates from the freeway mainline to mitigate congestion at the incident segment while concurrently adjusting signal timings along the arterial intersections to best accommodate the detour traffic. Their model, as reported in Chapter 4, has two distinct features:

- explicitly modeling the evolution of detour traffic along the ramps and surface streets with a set of dynamic network flow formulations to prevent local bottlenecks caused by demand surge from diversion operations and to properly set responsive signal timing plans; and

- providing a multi-objective optimization model to maximize the use of the available corridor capacity via detour operations without causing excessive congestion on the arterials and ramps.

Its multi-objective functions can further be stated as:

- maximizing the total throughput of the freeway corridor during incident management by using a parallel arterial as the detour route; and
- minimizing drivers' total times on the detour route to ensure their compliance with the routing guidance.

This integrated control model can also simulate an identified incident and traffic scenario on the given network and provide the optimized detour rate as well as total travel times over the network. For each decision scenario, this model can provide the results for operations with and without the detour. The third step uses the optimal detour rate for the initial decision making, while the derived delay reduced by detour operations serves as the basis for estimating the user benefits for the benefit-cost ratio criterion at the fourth step.

Benefit Estimation Procedure

The primary goal of implementing a detour plan is to ease the congestion and reduce the resulting delay due to incident-caused lane closures. Thus, responsible traffic managers need to compare the resulting benefits to the operational costs. This section briefly illustrates how to estimate the economic benefits contributed by the detour operations.

Given the estimated operational costs, one can approximate the benefit-cost ratio with the following steps for use as the criterion at the fourth step of the system.

Step 1: Compute the difference in travel times between the two scenarios — i.e., operations with and without the detour.

This study uses the ***total travel time over the network*** from the output of the integrated corridor control model to compute the reduced delay due to detour operations.

Step 2: Select other impacts which could also be part of the benefit analysis.

Reducing the delay for any reason may also decrease its associated MOEs. This study includes reductions in fuel consumption and emissions (i.e., HC, CO, NO, and CO_2) in the benefit estimation.

Step 3: Estimate the reduced MOEs using available references

The research team estimates the amount of fuel consumption reduced directly from traffic delays using the following conversion factors: 0.156 gallons of gasoline/hour for passenger cars (Koerner, 2008) and 0.85 gallons of diesel/hour for trucks (Lutsey et al., 2004).

Similarly, reduced emissions can be estimated from either the reduced amount of delay or fuel consumption, using the following conversion factors:

- HC: 13.073 grams/hour of delay (Maryland Department of Transportation, 2000)
- CO: 146.831 grams/hour of delay (Maryland Department of Transportation, 2000)
- NO: 6.261 grams/hour of delay (Maryland Department of Transportation, 2000)
- CO_2: 19.56 lbs CO2/gallon of gasoline (Energy Information Administration, 2009)
  22.38 lbs CO_2/gallon of diesel (Energy Information Administration, 2009)

Step 4: Convert the related delay, fuel, and emissions to monetary values

This step uses the monetary conversion factors listed below to estimate the reduced delay and associated MOEs:

- Delay: $28.57/hour for passenger cars (U.S. Census Bureau, 2009)
  $20.68/hour for truck drivers (U.S. Census Bureau, 2009)
  $45.40/hour for cargo drivers (De Jong, 2000; Levinson and Smalkoski, 2003)
- Fuel: $2.83/gallon for gasoline (Energy Information Administration, 2010)
  $2.99/gallon for diesel (Energy Information Administration, 2010)
- HC: $6,700/ton (DeCorla-Souza et al., 1998)
- CO: $6,360/ton (DeCorla-Souza et al., 1998)
- NO: $12,875/ton (DeCorla-Souza et al., 1998)
- CO_2: $23/metric ton (CBO, 2007)

Maximum Queue Length Estimation

Another key factor that traffic managers should consider when making their decision is the extent to which the congestion mitigation strategy would improve safety and reliability for motorists. To estimate this benefit, the best MOE would be the reduction in secondary incidents. Unfortunately, a rigorous methodology and data availability for this remain research issues (Chou and Miller-Hooks, 2010; Zhan et al., 2009*)*. Meanwhile, this study used the maximum queue length as a proxy variable, because the frequency of secondary incidents correlates highly to the queue length caused by the primary incident (Chou and Miller-Hooks, 2010; Zhan et al., 2009).

The maximum queue estimate model, the tool used here to evaluate the safety and reliability of a candidate detour plan, was developed based on simulation experiments with CORSIM (Kim et al., 2009). The entire network used for these experiments is a four-lane loop format highway similar to I-495 (Capital Beltway) in the Washington D.C. metropolitan area. The simulation did not consider lane drops, grades, and any other local bottlenecks in order to generate a queue solely due to incidents. The queue, defined as the length of the maximum spillback consisting of vehicles moving under 20 mph, was measured from the congestion caused by one isolated incident. In addition, this model development did not consider the queue in the opposite direction caused by the rubbernecking factor. To identify factors contributing to the queue induced by incidents, the simulation experiments explored a number of related variables, such as incident duration, the number of blocked lanes, traffic volume, on- and off-ramp volumes, the percentage of heavy vehicles, rubbernecking, and incident location.

Table 5-1 and Figure 5-3 summarize a regression model for estimating the maximum queue length, developed using 285 samples acquired from the CORSIM output. All 14 variables included in the proposed queue model show reasonable parameter signs, and they are all significant at the 10 percent confidence level. Note that the dependent variable is in a natural logarithm form of the maximum queue, implying that the simulated maximum queues approximately follow a log-normal distribution.

The estimation results show that, as expected, the queue length grows with increases in traffic volume and incident duration. Lane closures for Lanes 2, 3, and 4 have statistically significant impacts on the maximum queue, while rubbernecking effects do not play an important role.

Interestingly, the queue model proves highly sensitive to the locations of incidents. Most variables defined to capture the nature of the incident location (see Table 5-1) show significant contributions to the model, except for the variable *Away_On_1*, defined as 1 if an incident occurred about one mile away after passing an on-ramp and 0 otherwise. It is also noticeable that the variable *Away_On_2/3* (defined in Table 5-1) is much less significant than other incident-location variables. Moreover, variables for incident locations occurring before reaching the next on-ramp (e.g., *Away_Off_1/3*, *Near_Off_Bf*, *Near_Off_Af*, and *Btw_On_Off* in Table 5-1) show greater significances, with higher estimated coefficients. This implies that incidents occurring before reaching the next on-ramp are more likely to increase the queue.

Table 5-1: The maximum queue estimation model and descriptions of variables

$$
\begin{aligned}
\text{Log(queue(ft))} = {} & \underset{(51.07)}{6.6736} + \underset{(3.92)}{0.0191} \text{*HeavyVeh} + \underset{(15.79)}{0.0002} \text{*Main\_Vol} + \underset{(13.53)}{0.0149} \text{*Inc\_Dur} \\
& + \underset{(3.32)}{0.1930} \text{*LnB2} + \underset{(1.97)}{0.1147} \text{*LnB3} + \underset{(2.71)}{0.1528} \text{*LnB4} + \underset{(7.63)}{1.0079} \text{*Away\_Off\_1/3} \\
& + \underset{(6.82)}{0.8094} \text{*Near\_Off\_Bf} + \underset{(9.23)}{1.0020} \text{*Near\_Off\_Af} + \underset{(6.18)}{0.8100} \text{*Btw\_On\_Off} \\
& + \underset{(5.51)}{0.6371} \text{*Near\_On\_Bf} + \underset{(5.66)}{0.6284} \text{*Near\_On\_Af} + \underset{(5.31)}{0.5501} \text{*Away\_On\_1/3} \\
& + \underset{(1.68)}{0.1604} \text{*Away\_On\_2/3}
\end{aligned}
$$

Number of observations used : 285
$R^2 = 0.7360$
F-value for Model = 53.76
P-value for Model = < 0.0001
Note : Numbers in parentheses are *t*-statistic values

Descriptions of Variables

HeavyVeh : Heavy vehicle percentage (%)
Main_Vol : Volume on main lanes (vph)
Inc_Dur : Incident duration in minutes
LnB2 : 1 if Lane 2 is blocked due to the incident; 0 otherwise
LnB3 : 1 if Lane 3 is blocked due to the incident; 0 otherwise
LnB4 : 1 if Lane 4 is blocked due to the incident; 0 otherwise
(Note: Lane 1 is defined as the right-most lane, i.e., adjacent to the right shoulder)
Away_Off_1/3 : 1 if an incident occurred about 1/3 miles before the nearest off-ramp; 0 otherwise (Area 1 in Figure 5-3)
Near_Off_Bf : 1 if an incident occurred near (within 500 ft), but before passing, an off-ramp; 0 otherwise (Area 2 in Figure 5-3)
Near_Off_Af : 1 if an incident occurred near (within 500 ft), but after passing, an off-ramp; 0 otherwise (Area 2 in Figure 5-3)
Btw_On_Off : 1 if an incident occurred somewhere between an on-ramp and off-ramp; 0 otherwise (Area 3 in Figure 5-3)
Near_On_Bf : 1 if an incident occurred near (within 500 ft), but before passing, an on-ramp; 0 otherwise (Area 4 in Figure 5-3)
Near_On_Af : 1 if an incident occurred near (within 500 ft), but after passing, an on-ramp; 0 otherwise (Area 4 in Figure 5-3)
Away_On_1/3 : 1 if an incident occurred about 1/3 miles after passing an on-ramp; 0 otherwise (Area 5 in Figure 5-3)
Away_On_2/3 : 1 if an incident occurred about 2/3 miles after passing an on-ramp; 0 otherwise (Area 5 in Figure 5-3)

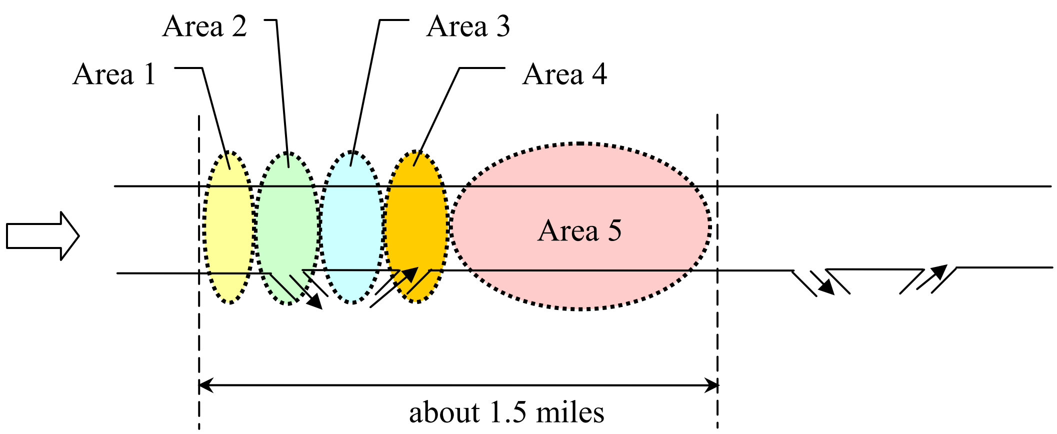


Figure 5-3: Illustrations of incident locations for the queue model

5.3 Illustration of the System Application

This section illustrates the application of the decision-support system to various experimental traffic scenarios and discusses its sensitivity to some key system parameters. The experimental analysis includes five scenarios for comparing the performance of the developed system with state-of-the-practice methods. Tables 5-2 and 5-3 present all data associated with each experimental scenario and the recommendations made by the proposed decision-support system. The weights for benefit-cost ratio, safety and reliability, accessibility, and acceptability in the experimental analysis are set at 0.31, 0.31, 0.18, and 0.20, respectively.

Table 5-2: Descriptions of scenarios

Summary of Case Study Scenarios							
Scenario No.		1	2	3	4	5	6
Scenarios for Incident & Traffic Condition	# of freeway lanes	4	3	2	3	3	3
	# of lanes in the detour route	1	1	1	1	2	1
	freeway volume (vplph)	250	250	250	750	750	250
	local volume 1 (vplph)*	400	200	200	800	800	800
	local volume 2 (vplph)*	600	300	300	200	200	200
	local volume 3 (vplph)*	600	600	300	300	200	300
	# of signals on detour	2	7	5	2	5	3
	compliance rate	0.9	0.6	0.5	0.5	0.6	0.5
	incident location	near off-ramp	middle of segment	near off-ramp	near on-ramp	Near on-ramp	near on-ramp
	incident duration (mins)	15	15	75	60	90	15

	# of lane blockage	1	3	1	3	3	3
	speed limit on detour route (mph)	40	30	30	50	40	40
MOEs for Criteria	optimal detour flow	0.76	0.80	0.25	0.85	0.54	0.77
	total travel time (hr) w/ detour	734	746	1,517	3,232	10,163	703
	total travel time (hr) w/o detour	855	801	1,527	3,617	10,182	787
	saved travel time (hr)	121	55	10	386	19	84
	B/C w/ detour	6.6	2.98	0.33	14.74	0.60	4.58
	B/C w/o detour	0.15	0.34	3.00	0.07	1.68	0.22
	max queue w/ detour (mile)	0.5	0.36	1.26	1.37	2.24	0.59
	max queue w/o detour (mile)	0.58	0.39	1.28	1.66	2.59	0.63
	travel time (min) via freeway	2.52	2.52	2.52	2.52	2.52	2.52
	travel time (min) via detour	7.52	9.15	11.44	6.55	7.52	7.52
Final System Outputs for Criteria and Alternatives							
Scenario No.		1	2	3	4	5	6
B/C	Detour	0.98	0.9	0.1	0.99	0.26	0.95
	No Detour	0.02	0.1	0.9	0.01	0.74	0.05
Safety & Reliability	Detour	0.53	0.52	0.5	0.55	0.54	0.51
	No Detour	0.47	0.48	0.5	0.45	0.46	0.49
Accessibility	Detour	0.25	0.22	0.18	0.28	0.25	0.25
	No Detour	0.75	0.78	0.82	0.72	0.75	0.75
Acceptability	Detour	0.53	0.43	0.38	0.38	0.43	0.38
	No Detour	0.47	0.57	0.62	0.62	0.57	0.62
Final synthesized confidences for alternatives	Detour	**0.62**	**0.56**	0.30	**0.60**	0.38	**0.58**
	No Detour	0.38	0.44	**0.70**	0.40	**0.62**	0.42

* Note: Local volume 1 represents the volume for the road connecting from freeway to detour route.
Local volume 2 represents the volume for the parallel route.
Local volume 3 represents the volume for the road connecting from detour route to freeway.
Operational and maintenance costs for the B/C estimates are provided by Maryland State Highway Administration (Maryland State Highway Administration, 2009).

Some key characteristics associated with each scenario and the resulting recommendations by the decision-support system are summarized below: Note that the summary focuses mainly on the lane blockage status and incident duration, since they are the primary decision criteria used in the literature. For more comprehensive analysis and comparisons, Table 5-3 lists the decisions from various agencies if using their decision criteria, reported in Chapter 2 (see Table 2-1):

Table 5-3: Comparisons of the decisions, using the criteria by different highway agencies and by the proposed system

	Scenario No.	1	2	3	4	5
Decision Criteria (used by agencies in the literature)	Lane Blockage (# of closed lane(s)/total # of lanes)	1/4	3/3	1/2	3/3	3/3
	Incident Duration (minutes)	15	15	75	60	90
Decisions by Agency	NC DOT-main office	N	Y	N	Y	Y
	NC DOT-Charlotte	N	N	N	Y	Y
	NJ DOT	Not clear	Not clear	Y	Y	Y
	Oregon DOT	N	Y	Y	Y	Y
	NY DOT	N	Y	N	Y	Y
	FL DOT	N	N	N	N	N
	ARTIMIS (Ohio/Kentucky)	N	N	N	Y	Y
	Idaho (Ada County)	Not clear	Y	Not clear	Y	Y
	Wisconsin DOT	Not clear	Not clear	Not clear	Not clear	Not clear
	Maryland	N	Y	Y	Y	Y
Decision by Proposed System		Y	Y	N	Y	N

* Note: *Y* and *N* represents "Detour" and "No Detour", respectively, for the decision.
Not clear represents insufficient clarity in the available decision criteria to make a concrete answer.

Scenario 1: The incident causes a partial road closure (one out of four lanes is closed), and its duration is relatively short (15 minutes).

System recommendation: Detour operations are recommended (beneficial), with 62 percent confidence.

Scenario 2: The incident causes a complete road closure on a three-lane highway segment for 15 minutes.

System recommendation: Detour plans are recommended (beneficial), with 56 percent confidence.

Scenario 3: The estimated incident duration is 75 minutes, and it blocks one lane on a two-lane highway segment.

System recommendation: Detour operations are not recommended (not beneficial), with 70 percent confidence.

Scenario 4: The incident causes a complete road blockage on a three-lane segment, and its duration is rather long (60 minutes).

System recommendation: Detour plans are recommended (beneficial), with 60 percent confidence.

Scenario 5: The incident causes a complete road blockage on a three-lane segment, and its duration is rather long (90 minutes).

System recommendation: Detour plans are not recommended (not beneficial) with 62 percent confidence.

Note that our proposed system recommends that properly detouring traffic in Scenario 1, with only partial lane blockage over a short incident duration, can still yield a sufficient total benefit if considered from the economic, environmental, and societal perspectives. The conclusion, however, would be quite different if one employs any of the state-of-the-practice methods shown in Table 2-1. The third column in Table 5-3 accurately represents the discrepancy of decisions between the agencies in the literature and the proposed system.

Similarly, based on those rules reported in Table 2-1, one may reach the conclusion that the incident condition in Scenario 3 justifies a detour operation (see decisions from New Jersey and Oregon DOTs in Table 2-1). However, the proposed decision-support system, by applying multiple criteria from various perspectives, does not recommend the detour implementation with fairly high confidence (70 percent). The system considers that the partial lane blockage and the light traffic demand on the freeway (500 vph) would not cause an excessive delay. Moreover, the long alternative route, with its several signalized intersections and low speed limit, would result in a long detour travel time. Consequently, such an operation may result in low compliance rate and a less favorable benefit-cost ratio.

Scenarios 2 and 5 demonstrate how the decision would change if different decision criteria were used. For instance, the main offices of the North Carolina DOT and New York State DOT use a single factor to make a decision for detour implementation. Based on their decision criterion, these agencies would implement detour operations for both Scenarios 2 and 5, because of the complete closure of the primary route. However, the proposed system makes different recommendations for those two scenarios, since their incident durations and the traffic conditions on the freeway and the alternative route are quite different, which leads to significantly different benefit-cost ratios (see Table 5-2).

By the same token, the New Jersey DOT would make identical decisions for Scenarios 4 and 5 using their criteria, i.e., complete road closure and long incident duration. However, the proposed decision-support system, by considering additional criteria, would make the opposite recommendations for those two scenarios. The major contributor to this discrepancy would be the number of signalized intersections on the alternative route. In Scenario 4, only two signalized intersections lie on the main detour route, whereas Scenario 5 has five of them. Signalized intersections on the alternative route tend to increase its travel times and delays. Thus, the optimization model is less likely to divert traffic to the detour route. Although the estimated optimal detour rate for Scenario 5 is about 54 percent, the total benefits from the saved total travel time are not sufficient to offset the operational expenses. Therefore, the multi-criteria decision-support system recommends no detour operations for Scenario 5, in contrast to the decision by the New Jersey DOT.

5.3.1 How relative weights for the evaluation criteria affect the final results

The results in Table 5-4 show that the final synthesized confidence for the recommendation by the proposed decision-support system varies with the relative weights associated with the set of evaluation criteria employed. Hence, this study has further used Scenario 6 in Table 5-2 as a base case and divided it into three sub-scenarios to illustrate how the responsible agencies' preferred criteria affect the final recommendation. Table 5-4 summarizes all data associated with each sub-scenario and the results of a sensitivity analysis. We present brief conclusions from the analysis below:

1) **Scenario 5-A:** Viewing economic gain and safety as the two most important criteria means that the decision maker should place higher weights on the **benefit-cost ratio** and on **safety and reliability**. Consequently, the decision-support system will yield the following recommendation, even though vehicles taking the detour route may experience much longer travel times than via the freeway:

   "***Detour operations are recommended, with 58 percent confidence***."
2) **Scenario 5-B**: If the decision makers place higher weights on **accessibility** and **acceptability**, factors which may affect compliance rates, the proposed decision-support

system will yield the following recommendation to not implement detour operations, unlike the conclusion for Scenario 5-A:

"Detour operations are not recommended, with 53 percent confidence.

3) **Scenario 5-C**: If all factors are equally important, the system will then yield the following decision:

"Detour operations are recommended, with 53 percent confidence."

Note that the above sensitivity analysis seeks to highlight the fact that choosing whether or not to implement a detour operation, when detecting an incident, is a complex decision-making process that should consider various associated factors, ranging from conventional traffic delay to socioeconomic impacts, such as creating a low-emission environment. The simple rules used in most state of practices may not be sufficient to yield the decision that best fits the traffic operational needs and the socio-environmental concerns. This study presents a comprehensive decision structure for rigorously incorporating all critical factors in making a timely detour decision to contend with non-recurrent congestion. Responsible traffic agencies, however, ought to place proper priorities on those key decision criteria, based on their local constraints, such as available resources, mission for a real-time incident response system, and/or priority concerns of the general public.

Table 5-4: Summary of sensitivity analysis for relative importance of criteria

Scenario No.		6-A	6-B	6-C
Weights for evaluation criteria	B/C	0.31	0.18	0.25
	Safety & Reliability	0.31	0.20	0.25
	Accessibility	0.18	0.31	0.24
	Acceptability	0.20	0.31	0.26
Final synthesized confidences for alternatives	Detour	**0.58**	0.47	**0.53**
	No Detour	0.42	**0.53**	0.47

* Note: The base scenario for this analysis is Scenario 6 in Table 5-2.

CHAPTER 6: Conclusion and Recommendations

6.1 Conclusion

Despite the increasing attention to minimizing incident-incurred congestion with optimal detour operations, effective guidelines are quite limited for determining when and how to make such decisions. Most existing guidelines are based mainly on the incident duration or lane closure status alone, offering no reliable procedure to consider the compound impacts of all related factors on the detouring effectiveness and overall socio-economic benefits.

This research has presented a multi-criteria decision-support system for determining the necessity of detour operations during incident management from an overall socio-economic benefit perspective. The developed system enables responsible agencies to consider all associated critical factors with preferred weights, including the direct benefits and operational costs, safety and reliability, accessibility of detour, and acceptability by travelers. This research is part of our developed integrated incident managing system for SHA that has various essential functions, ranging from prediction of incident duration to estimation of operational benefits. This decision module, based on the AHP methodology, features its computing efficiency and operational flexibility, allowing users to make necessary revisions if more data are available or more criteria need to be included.

This research has also reviewed the major studies on freeway incident traffic management (FITM) over the past decade, focusing on critical issues, existing approaches, and development of detour systems. Research findings from FITM-incidents managed by SHA and the practices by other highway agencies are summarized below:

- Most FITM-incidents managed by SHA involved collision, fatality, and lane closure. Since their resulting durations mostly exceeded several hours, how to improve the efficiency of operations while accommodating the needs of medical response units such as ambulance vehicles is a critical issue.
- The existing FITM operational manual by SHA offers a map-based routing plan for an identified incident location and clearly shows where to exit and return to the primary route, the number of intersections on the detour route, and some key geometric or control features on those detour links that may affect the operational efficiency. The operational

manual also provides a detailed link-based navigation in a table format and the emergency contact phone numbers for detouring travelers.

- Much more information needs to be included in the SHA's FITM operational manual: the locations to place portable DMS to guide the detouring travelers; messages to inform the approaching roadway users; procedures to have the initial estimate of the impact boundaries; the update frequency of the incident management progress; the percentage of traffic to be detoured during different stages of the clearance operations; and when to terminate the detour operations.
- Basing the detour operations either on the number of lanes blocked or on incident duration alone by most highway agencies is not sufficient to maximize the total societal benefits within the resource constraints.
- A reliable tool for traffic control operators to decide when and how to implement detour operations has yet to be developed.
- Given the resource constraints and priority concerns of each highway agency, the methodology to determine the need for detouring traffic during major incidents should consider multiple factors, such as cost/benefit ratio, safety and reliability, accessibility, and acceptability.
- A successful implementation of detour operations needs effective cooperation between freeway and local traffic agencies, especially on setting the detour duration, disseminating traffic information, and adjusting signal time plans on the detour route.
- A reliable model for predicting incident duration and the resulting traffic impact is an essential component of a detour-decision support system.

6.2 Recommendations

This section summarizes the following recommendations for SHA's enhancement of its operations in contending with non-recurrent congestion, based on a review of incident management practices by other states and an analysis of FITM-incidents responded to by CHART:

- SHA should add a traffic management component to the evaluation to address the aspect of FITM.

- SHA should extend its FITM planning and operations to all freeways and highways served by CHART to ensure that all FITM-incidents incurred in SHA's network can be responded to and managed in a timely manner.
- SHA should consider refining and expanding its FITM operational manual to include all essential steps to coordinate effectively with neighboring jurisdictions (e.g., county, VDOT) during major FITM-incidents that generally last over several hours.
- CHART should deploy portable sensors on the detour route during FITM operations, because most local routes that receive detoured traffic flows are not covered by any surveillance system that reflect their volumes and speeds over time during the incident management period. Without the real-time traffic conditions on the detour route, drivers following the instructions during a FITM operation may suffer undue delays.
- SHA should coordinate with local traffic agencies to develop a mechanism that can automatically trigger the adjustment of signal control plans on the detour route during SHA's response to major FITM-incidents. This step is to ensure that the capacity of all intersections on the detour route can be reallocated in time to accommodate the traffic volume surge and flow movement changes during the detouring period.
- CHART should consider offering the information on predicted travel times on both the freeway and the local route during the FITM operations to increase drivers' compliance with the detour suggestion.
- SHA should start deploying the system for estimating the required clearance duration of a detected incident and its resulting impact range, allowing responsible traffic engineers to better assess the need to activate an FITM plan.
- CHART should integrate all recent incident-management related models or systems sponsored by SHA and developed by local universities into an operational system and experiment with their potential effectiveness in minimizing the incident impacts on the regional roadway networks. Such studies include: incident detection algorithms with multisource information, incident duration and impact prediction models, optimal control of the traffic corridor during non-recurrent congestion, and a decision-support system for detour operations.
- CHART should consider deploying the multi-criteria decision-support system developed in this research on its website, allowing traffic engineers to take advantage of the

developed tool in making a real-time decision on whether or not to implement the detour operations, based on multiple criteria such as cost/benefit ratio, safety and reliability, accessibility, and acceptability.

- SHA should consider advancing its FITM operational manual to a web-based incident response and management system, enabling responsible traffic engineers to evaluate candidate management strategies in real time with available tools, monitoring the evolution of traffic conditions during the FITM operations, dynamically adjusting control or guidance strategies, and sharing the management progress to coordinate with neighboring jurisdictions.
- SHA's FITM operational manual should be extended to include other supplemental control strategies such as ramp metering, coordinating off-ramp signals, and variable speed control to maximize the effectiveness and benefits of incident management operations.
- More studies should be conducted on what information should be provided on both the primary incident route and the detour route during the onset of a detected incident and the activation of a FITM plan.
- In-depth surveys and analyses should be conducted to understand key factors playing a critical role in determining a driver's willingness to follow the detour guidance during an FITM operation.
- More training workshops on enhancing the effectiveness of incident management should be conducted, offering more opportunities for SHA traffic engineers to share operational concerns with the traffic research community and also assist them in best use of the available tools.

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