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7. Author(s) Dimitrios G. Goulias, Shmuel Yahalom, I-Jy Steven Chien		8. Performing Organization Report No.
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16. Abstract

Occupancy of travel lanes during construction and road maintenance are ordinary activities frequently undertaken to maintain the well-being of road infrastructure. When these activities take place impact traffic flow and generate delays on the users. Thus, it imposes costs on the users on heavily traveled routes due to traffic slowdowns or even shutdowns. At rush-hour these direct and indirect costs come to a peak. Construction and road maintenance closures can take place at times that the negative impacts would be minimized. This study focused on the appropriate guidelines for lane occupancy charges that would eventually minimize the disutility of traffic lane closure. The project research team examined heavily traveled locations in the NJ region, with the cooperation of NJ DOT engineers, to examine traffic and construction patterns to be used in the analysis and definition of the general occupancy charge guidelines. Information regarding traffic flow with respect to time of day, season, AADT, highway characteristics, etc. were reviewed in this examination. The project considered both economic and simulation analysis for examining the impact on user cost and construction operations due to different patterns of lane closure.

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CHAPTER 1. SUMMARY

Summary

Occupancy of travel lanes during construction and road maintenance are ordinary activities frequently undertaken to maintain the well-being of road infrastructure. When these activities take place impact traffic flow and generate delays on the users. Thus, it imposes costs on the users on heavily traveled routes due to traffic slowdowns or even shutdowns. At rush-hour these direct and indirect costs come to a peak. Construction and road maintenance closures are scheduled events. They can take place at times that the negative impacts would be minimized. This study focused on the appropriate guidelines for lane occupancy charges that would eventually minimize the disutility of traffic lane closure. The project research team examined heavily traveled locations in the NJ region, with the cooperation of NJ DOT engineers, to examine traffic and construction patterns to be used in the analysis and definition of the general occupancy charge guidelines. Information regarding traffic flow with respect to time of day, season, AADT, highway characteristics, etc. were reviewed in this examination. The project considered both economic and simulation analysis for examining the impact on user cost and construction operations due to different patterns of lane closure.

Background

During recent years innovative bidding and contracts (i.e., bonus/rental charge method, cost-plus-time method) have been used in Europe and more recently in the US. FHWA approved this method in 1985, on an experimental basis. To date, several states have used these contractual methods. A national survey was undertaken to examine the experience and use of lane occupancy charges in the 50 US states. The results are presented in a following section of this report. The survey included questions on the definition and methodologies used in defining lane occupancy charges and the type of economic and traffic analysis used.

Generally, each price bid under this method consist of two parts: the first part involves the activities and cost for the work to be performed; the second part describes the number of days to complete the project and the cost associated for the lane rental amount based on the daily rental rates. With this type of contract a disincentive/incentive provision is being included for accounting for any time overruns and/or early completion respectively.

In addition to the benefits in minimizing construction impacts on road users, it can be concluded that this method provides additional advantages: low competitive bidding is still applicable; increase in contract cost is minimal and contractor typically shortens contract times for taking advantage of the bonus option; projects with this option attract contractors with efficient construction and engineering management practice able to keep projects on schedule.

The lane-by-lane rental method is assessed only when the contractor closes a portion of the roadway. The rental charge is based on the number, duration and configuration of lanes closed. For example, the fee for having one lane and one shoulder closed would be less than that for having two lanes closed. In addition, higher rental amounts can be assessed for peak periods of the day. An illustrative example that was used for defining rental charges by some states is shown in table 1.

Table 1. Example of Daily Lane Rental Charges.

CLOSURE/OBSTRUCTION	RENTAL CHARGE ($)
One lane	20,000
One shoulder	5,000
One lane and shoulder	25.500
Two lanes	45.000
Two lanes and shoulder	50,000

Also lane rental may incorporate different charges depending on the time of day lane closure occurs since it affects different traffic level. An example of such charges are shown in table 2.

Table 2. Example of Rental Charge Assessed Hourly.

	HOURLY RENTAL CHARGE ($)	
Closure/Obstruction	6:30-9:00 am, 3:00-6:00 p.m.	All Other Hours
One lane	2,000	500
One shoulder	500	125
One lane and shoulder	2500	625
Two lanes	4500	1250
Two lanes and shoulder	5000	1375

A critical factor in the use of lane rental is the determination of the appropriate rental dollar amount. It has been suggested, and from the survey responses it can be concluded, that appropriate rental charges must be determined for each project, or potentially project type, on a case-by-case basis. The rental amount should be calculated on the basis of road-user costs estimated to be incurred as a result of anticipated delays and accidents during project construction. Rental amounts may also include construction engineering inspection costs and traffic control and maintenance costs that are anticipated to be generated during construction of the project. The calculation of road-user costs should be justified for each project and must be documented. Several references exist today on estimating road-user costs. However in the majority of the cases lane rental charges are based only on travel delays since several of the remaining parameters are variable in time, and are difficult and time consuming to measure and quantify. Further background information on traffic analysis and user cost analysis are provided in chapters 3 and 4 respectively.

To be effective and accomplish the objectives of applying the lane rental provisions, the rental amount must be defined so that the contractor is encouraged to stimulate innovative and fast-track construction methods, without compromising quality, so as to meet tight schedules. Otherwise, there will be little incentive to accelerate production, and the lane rental provisions may not produce the intended results, other than keeping the project on schedule.

Project Objectives

The objective of this study was to address the NJDOT need in developing appropriate guidelines for lane charges that would minimize the closure of traffic lanes. The developed guidelines considered the impact on traffic and road users, depending on the characteristics of the projects. The guidelines identify lane occupancy charges which are suitable to reduce closure of lanes to traffic. The study provides the general lane closure guidelines that can be used on a specific project and with respect to the specific project characteristics related to the AADT during the time of day, season, and type of highway/ lane closure. These guidelines were defined based on the examination of the effects of lane closures on traffic flow. These guidelines were defined based on project types and characteristics identified by NJDOT engineers. It is expected that the criteria used to determine lane rental for maintenance and construction schedule alternatives are, first, able to reduce private and social costs; second, able to impact construction and maintenance costs; and third, acceptable to the public and decision makers.

Organization of the Report

Chapter 1, provides the research background, research objectives and organization of the report. Chapter 2 presents the results of the national survey sent to the 50 states. Chapter 3 provides the methodology and analysis of the traffic analysis and delay evaluation. Chapter 4 presents the economic models and analysis, and Chapter 5 presents the summary and conclusions.

CHAPTER 2. SURVEY RESULTS

From the 50 States to which the lane occupancy survey was sent, it seems that only a few states are using or planning to use this approach in the near future. The responses indicate that only travel delays are used for defining occupancy charges, in many cases occupancy charges were defined on a project by project basis, and typically user cost values used were from the “red book”. In many cases the benefits of using occupancy charges were associated with the reduced construction time for project completion.

The specific responses from the various states that responded to the questionnaire, by January 1999, are included in the Appendix. Table 3 presents a summary of the analysis used in defining lane charges by specific States that provided this information.

States not using Lane Occupancy/Rental Charges

North Dakota, California, Connecticut, Idaho, Louisiana, Massachusetts, Minnesota, Hawaii, North Dakota, Texas, Utah, Washington State, Wyoming, Alaska

States planning to use Lane Occupancy/Rental Charges

Wyoming (considering a $400/lane/km), Utah

States using Lane Occupancy/Rental Charges

Arkansas, Colorado, Indiana, Oregon, Wisconsin.

Table 3. Summary of State responses

	Lane Charge Analysis Based On	
States Using/ Plan to use Lane Charges	Economic Analysis	Traffic Analysis/Simulation
Oregon	User Cost (red book*)	Yes
Arkansas	User Cost (red book*)	Yes (traffic counts)
Wisconsin	User Cost /QUEWZ	Yes (traffic counts & simulation)
Indiana	User Cost /QUEWZ	Yes (traffic counts)
Colorado	User Cost /QUEWZ	Yes (traffic counts & simulation)

*1977 AASHTO publication “A manual on user benefit analysis of highway and bus transit improvements”

CHAPTER 3. TRAFFIC IMPLICATIONS & ANALYSIS

Introduction

Traffic congestion occurs when travel demand exceeds the roadway capacity. Congestion can be either recurrent or non-recurrent. Non-recurrent congestion is caused by incidents, while recurrent congestion occurs at bottlenecks caused by geometric conditions such as the reduction in the number of lanes and lane width for roadway maintenance and/or reconstruction.

The application for delay measures include the traditional capacity improvement, alternatives analysis, operations evaluation, and a wide range of planning evaluations, such as the determination of lane closure configuration over time and space for a roadway maintenance or reconstruction project. In order to perform routine maintenance or reconstruction activities on freeways, lanes and shoulders are frequently closed. Due to physical loss of roadway space and rubbernecking factor, capacity at work zone decreases, thus traffic delays increase. Vehicular delay is often calculated by comparing actual travel speeds to desired travel speeds (e.g., free-flow speed). Many agencies didn't explicitly report the methodology used to calculate delay, but it is assumed that, in most instances, delay is calculated as the difference in average travel speeds and “acceptable or desired” speeds. The magnitude of delay associated with a work zone mainly depends on the distribution of traffic flow over the maintenance period and the corresponding work zone capacity. The estimation of traffic delays caused by freeway work zones is essential for scheduling of maintenance and construction activities as well as for estimating the life-cycle cost of pavement rehabilitation, restoration, resurfacing and reconstruction works (i.e., 4-R) alternatives.

In this study, the delays caused by vehicle deceleration, acceleration and in a queue are classified into moving delay and queuing delay. Deterministic queuing model is widely accepted by practitioners [See references 1,2,3,4,5 and 6] for estimating queuing delay.

However it was usually underestimated because the approaching and shock-wave delays were neglected. (5,7) CORSIM, a microscopic traffic simulation model, can mimic the traffic operation at work zones and thus can be used to estimate queuing delays at work zones. Despite its reliability, tedious work to prepare input files for different geometry, traffic and roadway condition may lessen its application for delay analysis purpose. Therefore it is necessary to develop an analytical model that will replicate the simulated results for estimating queuing delays under various demand, roadway and traffic conditions.

In this study, queuing delay is estimated by combining the simulation results and a deterministic model, while a mathematical model is developed for estimating moving delay. Microscopic simulation model CORSIM in TSIS 4.02 is used for this purpose.

Literature Review

In order to perform the work zone delay analysis, a thorough review of previous studies related to freeway work zone has been conducted and discussed below.

<u>Models for Analyzing Freeway Work Zone Delay</u>

Two well-known types of methods developed for analyzing freeway queuing delay include deterministic queuing models (See references 3,8,9, and 10) and the shock wave models. (11,12) The deterministic queuing model has been used for estimating delays in practice for decades. It is often depicted using a deterministic queuing diagram as shown in Figure 1. The critical inputs to the deterministic queuing diagram (DQD) are the demand volume Q, freeway capacity C, work zone capacity C_w, and work zone duration t_1. The shaded area is the total delay to the traffic stream, and is given by the following equation:

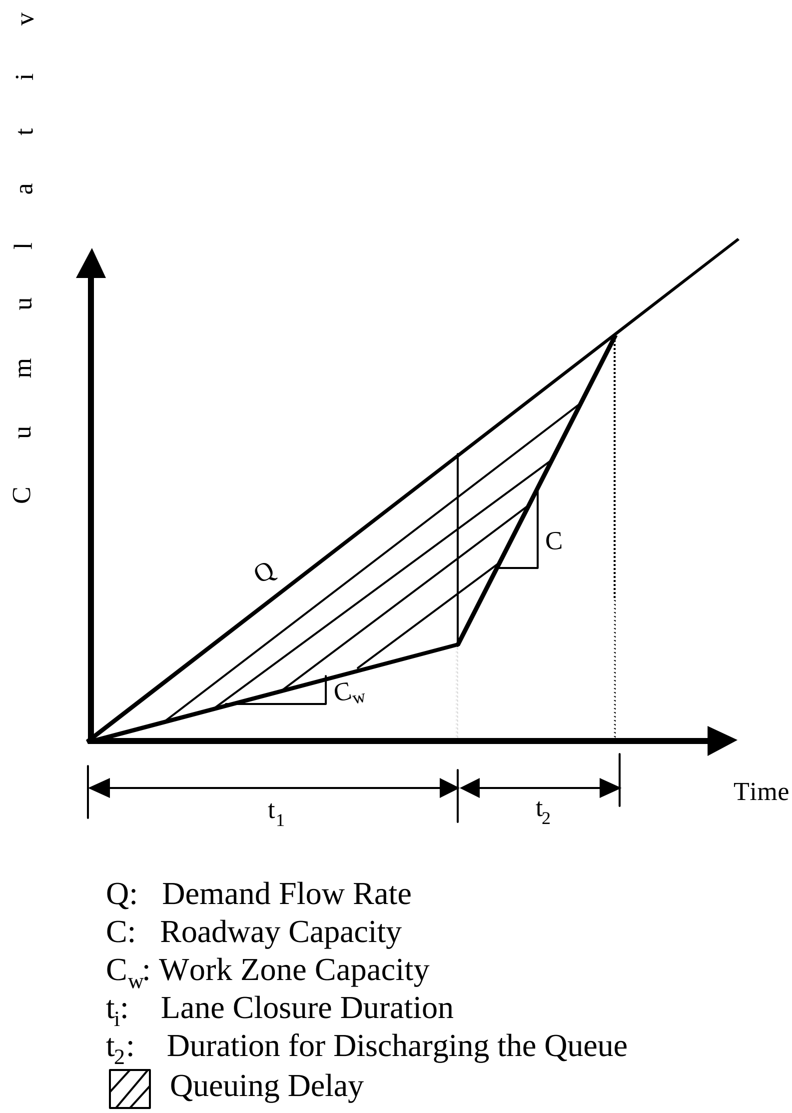


Figure 1. Queuing delay estimated by the deterministic queuing model.

$$Delay = \frac{t_1^2 (C - C_w)(Q - C_w)}{2(C - Q)} \tag{1}$$

The main limitations in the existing deterministic models for estimating work zone congestion are summarized as follows.

(1) Some methods used peak hour factors instead of actual traffic counts to estimate traffic demand during work zone period.

(2) Data on traffic counts and work zone times are often not collected simultaneously.
(3) The speeds used to estimate work zone delay are not the actual speeds through the work zone queues.
(4) An assumption that the initial demand level is smaller than freeway capacity is not valid under peak conditions.

The shock wave model estimates queuing delay by assuming that (1) the traffic flow is analogous to fluid flow, and (2) the shock wave speed propagates linearly. In the determination of queuing delay, the shock wave speed is approximated based on traffic density, which is considered difficult to measure form flow density relations. In 1978, Wirasinghe developed a model based on shock wave theory to determine individual and total delays upstream of incidents [(10)]. The model was formulated considering traffic conditions under different densities and areas which are formed by shock waves in the time-space plot. Later, in 1995, Al-Deck, Garib, and Radwan presented a method which utilized detailed incident and traffic data collect simultaneously in several traffic surveillance systems at different locations in the U.S [(10)]. In that study, recurrent and non-recurrent congestion can be identified, while shock wave theory was used to estimate incident congestion. The method was applied on the Rt I-880 project in Alameda County, California [(10)]. Satisfactory results were achieved for both isolated and multiple incident cases.

In 1984, Memmott and Dudek developed a computer program, called Queue and User Cost Evaluation of Work Zones (QUEWZ), which can assess the work zone user costs, including the user delay and vehicle operating costs [(6)]. QUEWZ was developed based on traffic data collected from Texas highways. In QUEWZ, a deterministic queuing model is used to estimate queue delay, while approach speed, calculated by using the equations taken from the Highway Economic Evaluation

Model and an assumed speed-volume relations, is used to estimate delay through the lane-closure section [13].

In 1998, Chien and Schonfeld developed a mathematical model to optimize work zone length on four lane (two-lane two-way) highways where one lane in each direction at a time was closed for performing maintenance activities[10]. In that study, deterministic queuing theory was used to estimate user delay caused by the lane closure. The optimal work zone length was determined by minimizing the total cost including the agency, accident, and user delay costs. In addition to the queuing delay cost, the moving delay incurred by vehicles traversing through work zone was considered to formulate the user delay function.

In 1999, Jiang conducted a study for Indiana Department of Transportation, in which the work zone related delays were classified into (1) deceleration delay: incurred by vehicle deceleration before entering work zones, (2) moving delay: incurred by vehicles passing through work zones with lower speed, (3) acceleration delay: incurred by vehicles acceleration after existing work zones, and (4) queuing delay caused by ratio of vehicle arrival and discharge rates [4].

In a recent study, Nam and Drew found that deterministic queuing models always underestimate the delays comparing with that estimated by shock wave models [7].

Traffic Operations and Capacities at Freeway Lane Closures

Previous studies [See references 14,15,16 and17.] that dealt with traffic operations and capacities at freeway lane closures are reviewed, which provide valuable information in designing simulation networks, determining calibration parameters and evaluating delays in this study. In 1985, Nemeth and Rathi conducted a simulation study for a hypothetically created freeway network by using FREESIM and indicated the potential impact of speed reduction at freeway lane closures [14]. They found that compliance with the reduced speed limit had no significant impact on the number of uncomfortable decelerations, but it reduced variance in speed distribution over the work zone. The results showed that the speed reduction at work zones does not create hazardous disturbances in traffic flow.

In 1985, Rouphail and Tiwari investigated speed characteristics near freeway lane closure areas [(15)]. They identified factors affecting speed through a lane closure, including (1) geometric related factors (i.e., the configurations of lane closures before and within the work zone, grade and curvatures, effective lane width and lateral clearance, sight distance and proximity to on and off ramps), (2) traffic related factors (i.e., flow rates passing through work zone areas and truck percentage in traffic stream), (3) traffic control related factors (i.e., arrow board, and canalization devices, speed zoning signs, the presence of flagmen), and (4) work zone activity related factors (i.e., location, crew size, equipment type, noise, dust level, and length of work zone). They also found that the vehicle mean speed through a work zone decreased while (1) the intensity of construction and maintenance activities increased, and (2) the construction and maintenance activities moved closer to the travel lanes. Later, in 1997, Pain, McGee, and Knapp conducted a comprehensive speed studies and found that the mean speed significantly varied with the configurations of lane clousres (e.g., right lane closure, left lane closure, and a two-lane bypass), traffic control devices (e.g., cones, tubular cones, barricades, and vertical panels), and locations within work zones [(17)].

Later in 1988, Rouphail, Yang, and Fazio derived various mean values and coefficients of variation to describe the speed changes in different work zones [(16)]. They found that the average speed in a work zone did not vary considerably under light traffic conditions; however, the speed recovery time took longer as traffic volumes increased.

Capacity reduction is the most critical factor that influences traffic delays. Several studies identified that the capacity at freeway work zone mainly depends on (1) lane closure configuration, (2) on-ramp and off-ramp proximity, (3) lane narrowing, (4) physical barriers, (5) percentage of heavy vehicles in the traffic stream, (6) additional warning signs, (7) reduced speed limit, and (8) grade (See references 3,15,18, and 19.). However, the detailed procedure for estimating freeway work zone capacity that can capture the influence of above variables was not developed.

Previous studies also developed different methods to identify capacities of freeway work zones. Dudek and Richards identified work zone capacity as the hourly traffic volume under congested conditions [3]. In this analysis capacity was calculating by considering the traffic volume that can pass through work zones in an hour, and considering the queue formed at up stream from the lane closure. The 1994 Highway Capacity Manual provided typical capacity values of freeway work zones. As Dixon, Hummer, and Lorscheider indicated, these values were obtained using traffic data collected on roadways in Texas, which may not represent the roadway capacity in other states because of different freeway characteristics and driving behaviors [18].

Characteristics of Simulation Models

CORSIM (CORridor SIMulator), a microscopic simulation model developed by Federal Highway Administration (FHWA), contains the features of NETSIM and FREESIM. It is viewed as one of the most comprehensive traffic simulation model, which can simulate traffic operations, including incident conditions (i.e., work zones and accidents), surface streets and freeways.

CORSIM runs on a microcomputer and simulates various traffic flows (i.e., volumes, vehicle compositions) operating on roadways with different geometric conditions (i.e., grades, radius of curvature, super-elevations on the freeway, lane additions/drops) and freeway incidents (i.e., accidents, work zones rubbernecking factor) while considering various driver types (i.e., cautious, aggressive) and vehicle types (i.e., auto, truck, carpool, bus) characteristics (i.e., length, acceleration/deceleration rate). The vehicle movements are modeled based on car following, lane changing, and crash avoidance maneuvers programmed in the CORSIM model. [20] Many researchers have employed CORSIM for freeway operational analysis, such as velocity and capacity studies. [1,2,14] In 1999, Vadakpat, Stoffels and Dixon calibrated and validated CORSIM model for work zones [20]. They found that the default value of CFSF and 50 percent rubbernecking factor can reasonably replicate the vehicle and driver behavior at work zones based on the work zone data collected from several sites in North Carolina.

Freeway Work Zone Capacity

Traffic flow and roadway capacity are the principal determinants of traffic delays. In general, as the traffic flow exceeds the capacity that can be accommodated by a work zone (if a number of lanes are closed for maintenance or reconstruction activities), a queue forms, whose length depends on the magnitude of the excess flow and the duration to reopen the closed lanes.

A microscopic traffic simulation model, CORSIM, developed by Federal Highway Administration (FHWA) US Department of Transportation (USDOT), is extensively used for the approximation of work zone capacity and delay analysis. In order to reduce tremendous simulation time due to simulating various work zone configurations while considering various traffic (e.g., traffic volume and composition) and geometric conditions (lane width, grade section percentage and length, and numbers of normal and closed lanes), the capacity adjustment factor based on the capacity under ideal conditions defined in the Highway Capacity Manual (HCM) is introduced here for traffic engineering studies, such as estimating delays.

Estimation of Work Zone Capacity under Ideal Conditions

According to the definition of “capacity” in 1994 HCM. it is “the maximum hourly rate which persons or vehicles can reasonably be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions [22]." The maximum equivalent hourly flow rate is determined based on a maximum fifteen-minute flow rate under ideal conditions. The ideal conditions represent 12 feet minimum lane width, 6 feet minimum lateral clearance between the edge of the travel lane and the nearest roadside or median obstacle or object influencing traffic behavior, all passenger cars in the traffic stream, and a driver population dominated by regular and familiar users of the facility.

Simulation approaches have been used to approximate freeway capacity for years. CORSIM, a microscopic traffic simulation model, is able to simulate the exact number of vehicles passing through a designated link (containing a work zone) during a specific time period. Thus, the work zone capacity defined in this study is the maximum hourly flow passing through the zone approximated by CORSIM. In order to approximate work zone capacity, the entry flow rate, the number of vehicles passing a point in a unit time, is gradually increased. The maximum flow is identified when the entry flow exceeds the observed flow passing through a work zone (see figure 2).

In order to reduce the statistical variance incurred by using simulation approaches for the analysis (e. g., the maximum observed flow varies while the random number seed is changed), the maximum discharged flow rate (capacity) is determined based on the average of flow approximated from the average maximum flows obtained from 10 one-hour simulation runs with different random number seeds. The work zone capacities under ideal conditions for various zone configurations are summarized in table 4, where the average link speed is 65 mile per hour (mph).

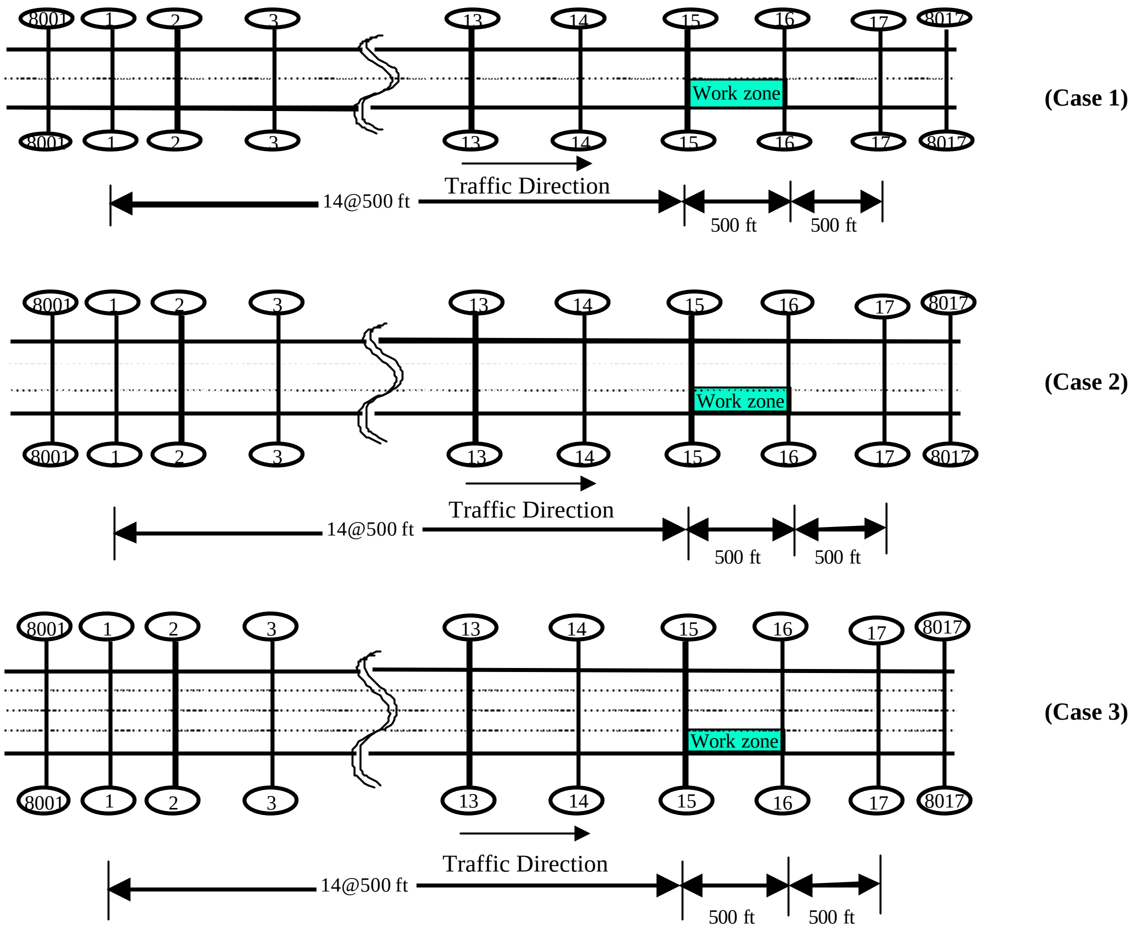


Figure 2. Typical work zone configurations used for estimating delays by CORSIM.

Table 4. Work zone capacities for various zone configurations.

Freeway Types (Lanes per direction)	Work Zone Capacity with One Blocked Lane (vph)
2	1450
3	4000
4	6550

Adjustment of Freeway Capacity under Prevailing Conditions

Any prevailing conditions differing from the ideal conditions defined in the HCM will reduce the maximum service flow rate, the capacity. These conditions may come from a single factor or a combination of factors including heavy vehicle factor f_{HV}, lane width and lateral clearance factor f_W and driver population factor f_P.

As suggested by the 1994 HCM, the adjusted hourly maximum flow rate (vph) under prevailing condition can be approximated by using the correction factors:

$$V = vNf_W\, f_{HV}\, f_P \tag{2}$$

where:

V= service flow rate under prevailing roadway and traffic conditions

v = peak flow rate under ideal conditions (passenger cars per hour per lane - pcphpl)

N = number of opened lanes,

f_W = factor to adjust for the effect of restricted lane widths and lateral clearances,

f_{HV} = factor to adjust for the effects of trucks and recreational vehicles, and

f_P = factor to adjust for the effect of recreational or unfamiliar driver population

The heavy vehicle factor f_{HV} can be calculated from equation 3, which was discussed in equations 3-5 of the 1994 HCS.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \tag{3}$$

where:

E_T = passenger car equivalents for trucks/buses in the traffic stream,

E_R = passenger car equivalents for recreational vehicles in the traffic stream,

P_T = proportion of trucks/buses in the traffic stream, and

P_R = proportion of recreational vehicles, in the traffic stream.

The equivalent number of passenger cars per truck was investigated and summarized in the 1994 HCM, where tables 5 and 6 are used for converting given vehicle compositions to the corresponding equivalent numbers of passenger cars. In 1997, a freeway capacity analysis by Chien and Chowdhury developed a method to find the equivalent passenger cars per truck using simulation approach [(1)]. In that study, they found the results are consistent with the 1994 HCM when the grade is small and the section length is short. In equation 3, the variables E_T and E_R can be found from tables 5 through 8, while other factors, such as f_W and f_P, can be obtained from tables 9 and 10, respectively.

Table 5. Passenger car equivalents on general freeway segments.

	TYPE OF TARRAIN		
CATEGORY	LEVEL	ROLLING	MOUNTAINOUS
E_T for trucks and buses	1.5	3.0	6.0
E_T for recreational vehicles	1.2	2.0	4.0

Source: Table 3-3, 1994 Highway Capacity Manual (HCM)

Table 6. Passenger car equivalents for trucks and buses on specific upgrades

GRADE (%)	LENGTH (MI)	E_T								
		PERCENT TRUCKS AND BUSES								
		2	4	5	6	8	10	15	20	25
< 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
2	0- ¼	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	¼- ½	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	½- ¾	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	¾-1	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	1-1 ½	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	>1 ½	4.5	3.5	3.0	3.0	2.5	2.5	2.0	2.0	2.0
3	0-¼	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	¼- ½	3.0	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5
	½- ¾	6.0	4.0	4.0	3.5	3.5	3.0	2.5	2.5	2.0
	¾-1	7.5	5.5	5.0	4.5	4.0	4.0	3.5	3.0	3.0
	1-1 ½	8.0	6.0	5.5	5.0	4.5	4.0	4.0	3.5	3.0
	>1 ½	8.5	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
4	0-¼	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	¼- ½	5.5	4.0	4.0	3.5	3.0	3.0	3.0	2.5	2.5
	½- ¾	9.5	7.0	6.5	6.0	5.5	5.0	4.5	4.0	3.5
	¾-1	10.5	8.0	7.0	6.5	6.0	5.5	5.0	4.5	4.0
	>1	11.0	8.0	7.5	7.0	6.0	6.0	5.0	5.0	4.5
5	0- ¼	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	¼- 1/3	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0
	1/3- ½	9.0	7.0	6.0	6.0	5.5	5.0	4.5	4.0	3.5
	½- ¾	12.5	9.0	8.5	8.0	7.0	7.0	6.0	6.0	5.0
	¾-1	13.0	9.5	9.0	8.0	7.5	7.0	6.5	6.0	5.5
	>1	13.0	9.5	9.0	8.0	7.5	7.0	6.5	6.0	5.5
6	0-¼	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
	¼- 1/3	9.0	6.5	6.0	6.0	5.0	5.0	4.0	3.5	3.0
	1/3- ½	12.5	9.5	8.5	8.0	7.0	6.5	6.0	6.0	5.5
	½- ¾	15.0	11.1	10.0	9.5	9.0	8.0	8.0	7.5	6.5
	¾-1	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5
	>1	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5

NOTE: If the length of grade falls on a boundary, apply the longer category; interpolation may be used to find equivalents for intermediate percent grades.
Source: Table 3-4, 1994 Highway capacity manual (HCM)

Table 7. Passenger car equivalents for recreational vehicles on specific upgrades.

GRADE (%)	LENGTH (MI)	E_R								
		PERCENT RECREATIONAL VEHICLES								
		2	4	5	6	8	10	15	20	25
≤ 2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
3	0- ½	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>½	2.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
4	0-¼	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	¼- ½	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> ½	3.0	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5
5	0-¼	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	¼- ½	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	>½	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
6	0- ¼	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	¼- ½	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	> ½	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0

NOTE: If the length of grade falls on a boundary, apply the longer category; interpolation may be used to find equivalents for intermediate percent grades.
Source: Table 3-5, 1994 Highway capacity manual (HCM)

Table 8. Passenger car equivalents for trucks and buses on specific downgrades

DOWN GRADE (%)	LENGTH OF GRADE (MI)	PASSENGER CAR EQUIVALENT E_T			
		PERCVENT TRUCK/BUSES			
		5	10	15	20
<4	All	1.5[a]	1.5[a]	1.5[a]	1.5[a]
4	≤ 4	1.5[a]	1.5[a]	1.5[a]	1.5[a]
4	> 4	2.0	2.0	2.0	1.5
5	≤ 4	1.5[a]	1.5[a]	1.5[a]	1.5[a]
5	> 4	5.5	4.0	4.0	3.0
≥ 6	≤ 4	1.5[a]	1.5[a]	1.5[a]	1.5[a]
≥ 6	> 4	7.5	6.0	5.5	4.5

[a]Value for level terrain
Source: Table 3-6, 1994 Highway capacity manual (HCM)

Table 9. Adjustment factor for restricted lane width and lateral clearance.

DISTANCE FROM TRAVELED WAY TO OBSTRUCTION[a] (FT)	ADJUSTMENT FACTOR					
	OBSTRUCTIONS ON ONE SIDE			OBSTRUCTIONS ON TWO SIDES		
	LANE WIDTH [a](FT)					
	≥ 12	11	10	≥ 12	11	10
≥ 6	1.00	0.95	0.90	1.00	0.95	0.90
4	0.99	0.94	0.89	0.98	0.93	0.88
2	0.97	0.92	0.88	0.95	0.90	0.86
0	0.92	0.88	0.84	0.86	0.82	0.78

[a]Interpolation may be used for lane width or distance from traveled way to obstruction.
Source: Table 3-2, 1994 Highway Capacity Manual (HCS)

Table 10. Adjustment factor for driver population.

TRAFFIC STREAM TYPE	ADJUSTMENT FACTOR (f_p)
Weekday, commuter (familiar user)	1.00
Recreational or other	0.75-0.99

Source: Table 3-7, 1994 Highway Capacity Manual (HCM)

Traffic Delays at Freeway Work Zones

The estimation of traffic delays at freeway work zones is essential for planning and scheduling maintenance and construction activities. Traffic delay mainly incurred by motorists waiting in queues as well as traveling within work zones below their desired speeds due to the limited capacity caused by either lane closure or rubbernecking factor.

Traffic delays consist of those in congested and not congested traffic conditions. When the arrival flow rate exceeds the work zone capacity, traffic congestion occurs and therefore results in vehicle queues. On the other hand, as the arrival rate is below the work zone capacity, vehicles may pass through the work zone smoothly with lower speed than that under normal condition. The proposed method for estimating work zone related delays with CORSIM in conjunction with the1994 HCM and queuing theory is developed and discussed below.

In this study, the delays due to vehicle deceleration, acceleration, and in queue are aggregated, called queue delay, and estimated by CORSIM. The delay due to reduced travel speed through the work zone is called moving delay, which is estimated by a mathematical model.

The definition of user delay is the difference between the average travel times under normal (without work zone situation) and roadway maintenance (with work zone) situation, multiplied by the number of vehicles passing through the zone in a given time period. The magnitude of delay associated with a work zone mainly depends on the variation of traffic flow over the maintenance period and the corresponding work zone capacity, which can be classified into moving and queuing delays. The moving delay is incurred by vehicles traveling within the work zone, which increases as the average zone speed decreases. The speed reduction is mainly caused by the disturbance of work zone barriers and the variation of traffic density. In addition, motorists may suffer another type of delay, called queuing delay, when they stop-and-go in queues at the upstream of the work zone. A queue will form once the traffic flow exceeds the work zone capacity, whose length changes dynamically because of flow variation over time.

Furthermore, if the inflow demand exceeds work zone capacity during a given time period (the duration of time periods t_p are assumed to be one hour in this study), vehicles can not be completely discharged before the end of the time period. Thus, the queue discharging time will be extended to the next time period. If inflow rates continuously exceed the capacity in a series of time periods, the queue growing rate varies with the inflow rates in different time periods. In general, the total number of vehicles in a queue can be fully discharged until the cumulative inflow rates reaches the cumulative capacity over a number of time periods. In addition, while forming the queue, the shock wave delay associated with the discharged and in-coming flows is a fraction of queue delay. Unfortunately, it is hard to formulate mathematically.

Estimation of Moving Delays

The moving delay is incurred by motorists traveling through a work zone with reduced travel speed. The speed reduction may be caused by the lack of roadway clearance, narrowed lanes, rubbernecking factors, etc. The moving delay can be obtained by the product of the travel time difference of travel times (under normal and work zone conditions) and the flow rate passing through the work zone. Depending on the relation of the workzone capacity C_w, the inflow volume $Q(i)$, and a queue accumulate from the previous time period $q(i)$, the moving delay $t_M(i)$ of time period i is formulated based on different situations discussed below.

Situation 1: $Q(i) + q(i) \leq C_w$

In this situation, the total inflow volume can pass through the work zone in the same time period. Therefore, the moving delay is:

$$t_M(i) = \left(\frac{L}{V_w} - \frac{L}{V_a}\right)[Q(i) + q(i)] \quad (4)$$

where V_a, V_w and L represent average operating speed without the work zone, average work zone speed and work zone length, respectively. In equation 4 $q(i)$ can be determined by the excess traffic flows accumulated from previous time periods, and formulated as follows:

$$q(i) = \sum_{j=k}^{i-1} Q(j) - (i-k)C_w \quad \text{where } i > k, \forall i \tag{5}$$

where k is the beginning time period as demand $Q(k)$ is greater than capacity C_w.

For example, if $k = 3$, the queue length at the beginning of the 6th time period is:

$$q(6) = \sum_{j=3}^{6-1} Q(j) - (6-3)C_w = Q(3) + Q(4) + Q(5) - 3C_w \tag{6}$$

Situation 2: $Q(i) + q(i) > C_w$,

If $Q(i) + q(i) > C_w$, the term $[Q(i) + q(i)]$ in equation 4 is replaced by C_w subject to the work zone capacity constraint, such that the moving delay $t_M(i)$ at time period i is

$$t_M(i) = \left(\frac{L}{V_w} - \frac{L}{V_a}\right)C_w \tag{7}$$

Note that the average work zone speed V_w can be determined from the data collected from roadway surveillance systems in the study sites or empirical speed functions (e.g., BPR functions), to reflect the realistic travel speed varying with the change of traffic volume and roadway capacity ratio.

<u>Estimation of Queuing Delays</u>

In this section, a model, integrating simulation results and a deterministic queuing model, is developed for estimating queuing delay. In order to estimate the queuing delays with CORSIM, a computerized freeway segment on the east bound Rt I-80 in New Jersey is established for simulation. The major data were collected from NJDOT, including road geometry, traffic volumes, and average speeds at specific data stations. Some traffic data were found from an HOV lane evaluation study report by Parsons Brinkerhoff, Garmer Associates, and New Jersey Institute of Technology [(10).]

The simulation model is calibrated by fine tuning parameter such as car following sensitivity factors, vehicle startup delay, and driver response leg times to reflect the realistic traffic operations on I-80. After validating the calibrated model, three typical freeway work zone configurations, such as shown in figure 2, are simulated with various input of entry volumes, and work zone capacities, while the corresponding queue delay can be observed from simulation output.

As defined previously, the total queuing delay is the product of the travel time difference between the average travel times with and without work zone conditions and the demand. In order to estimate queuing delays, both normal and work zone (one blocked lane) conditions with various entry volume and work zone capacity (V/C_w) ratios are simulated. The duration of each simulation run, which is also the duration of lane closure, is determined based on the assumption that all entry vehicles can pass through the work zone before the end of simulation.

After conducting simulation analysis, it is found that if the traffic volume is low (e.g. at V/C_w = 0.4 or less), the queuing delay is relatively small compared with that as V/C_w > 0.5 and thus is not considered. Table 11 shows different hourly entry volumes represented by V/C_w ratios for the three work zone cases.

The queuing delay corresponding to the entry volume can be determined by the difference between the delays with and without work zone conditions. To reduce statistical variance of delay estimated by simulation, the traffic delays observed from simulation are averaged by simulating 10 times for any given entry volume with different random number seeds. The average queuing delay (min/veh) corresponding to each entry volume can be obtained from simulated total delay by dividing by the entry volume. The mean and the standard deviation of queuing delays for each of the three cases with various V/C_w ratios are obtained and summarized in table 12 and shown in figures 3, 4, and 5.

Table 11. Work zone capacity and flow rates for various cases.

Case #	C_w : Work Zone Capacity (vph)	Flow Rates (V/C_w Ratio)
1	1450	From 0.5 to 1.8 with the increment of 0.1
2	4000	From 0.5 to 1.7 with the increment of 0.1
3	6550	From 0.5 to 1.4 with the increment of 0.1

Table 12. Queuing delay vs. V/C ratio vs. delays with various cases.

V/C_w Ratio	Average Delay (min./veh.)		
	Case 1	**Case 2**	**Case 3**
0.5	*0.017 (0.011)	0.039 (0.019)	0.056 (0.011)
0.6	0.042 (0.009)	0.080 (0.028)	0.115 (0.016)
0.7	0.054 (0.018)	0.140 (0.026)	0.246 (0.032)
0.8	0.075 (0.019)	0.250 (0.040)	0.556 (0.046)
0.9	0.193 (0.048)	0.872 (0.100)	1.175 (0.060)
1	0.681 (0.502)	2.841 (0.157)	2.722 (0.164)
1.1	4.171 (1.132)	6.015 (0.246)	5.754 (0.103)
1.2	8.639 (0.432)	9.686 (0.226)	9.272 (0.271)
1.3	12.780 (0.846)	13.637 (0.495)	13.148 (0.242)
1.4	17.552 (0.980)	17.865 (0.532)	16.974 (0.131)
1.5	21.701 (0.826)	21.958 (0.463)	
1.6	25.960 (0.764)	25.877 (0.506)	
1.7	30.686 (1.412)	30.254 (0.551)	
1.8	35.263 (1.006)		

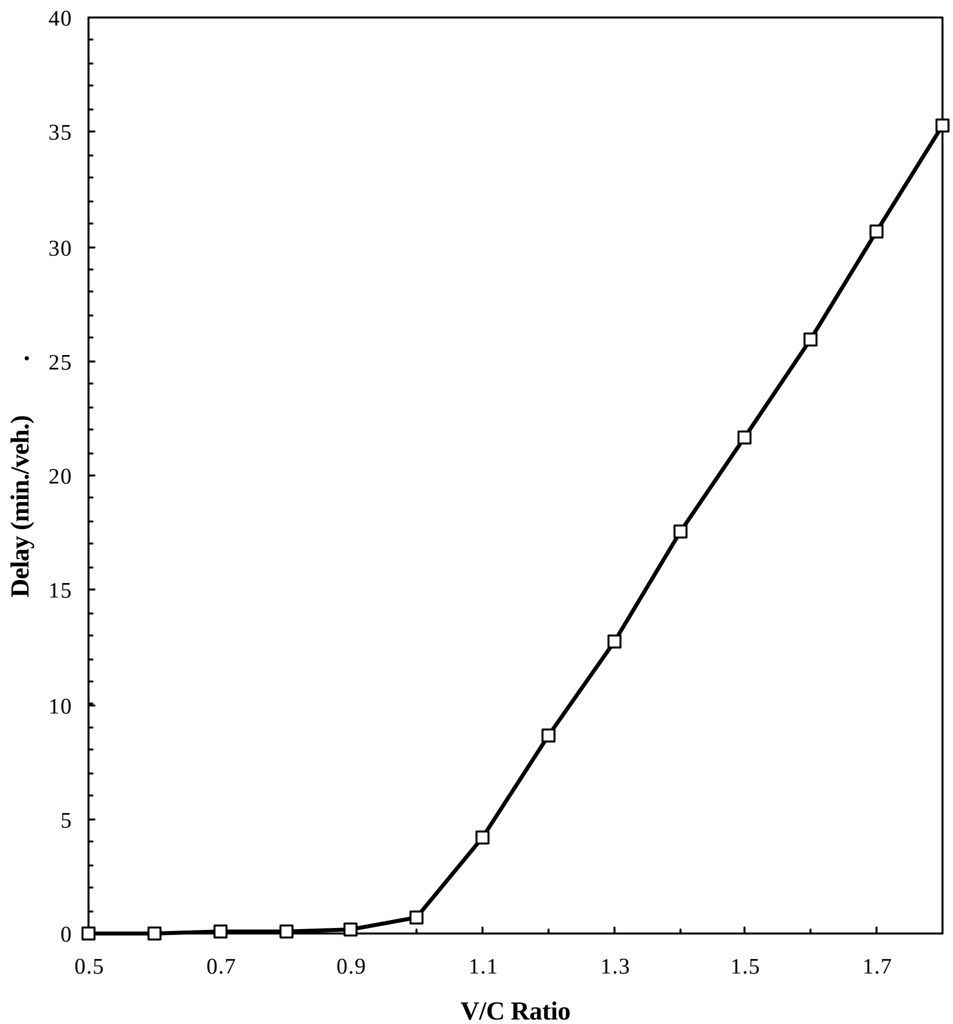


Figure 3. Average delay vs. V/C ratio (two lane freeway with one blocked lane without trucks)

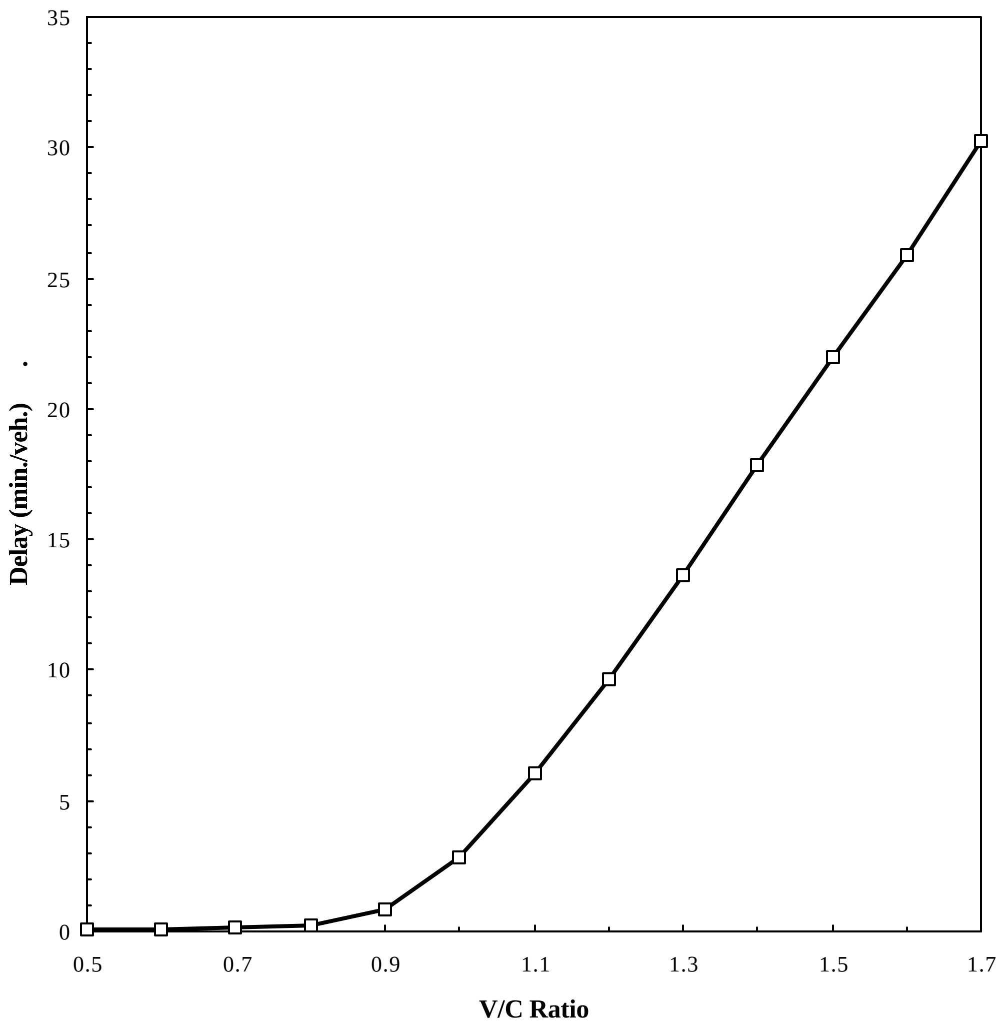


Figure 4. Average delay vs. V/C ratio (three lane freeway with one blocked lane without trucks)

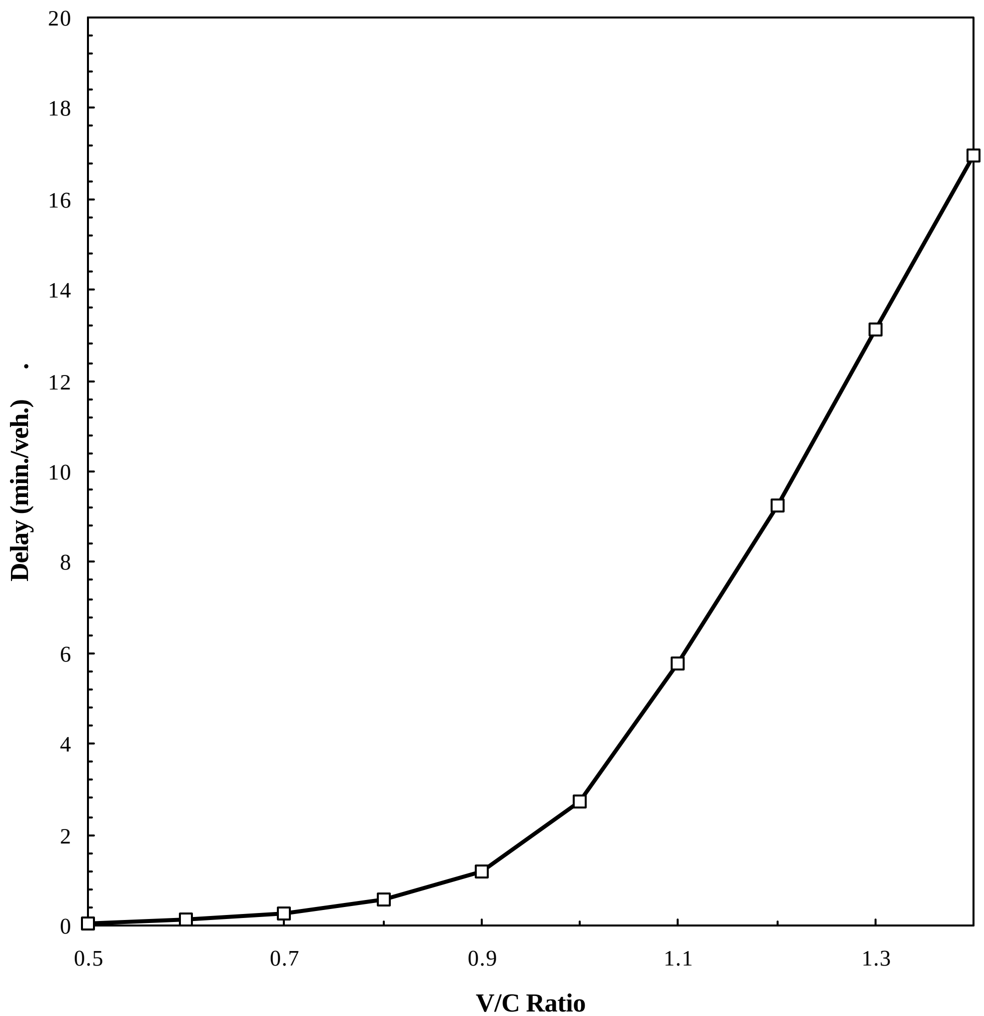


Figure 5. Average delay vs. V/C ratio (four lane freeway with one blocked lane without trucks)

Model Development

In order to avoid simulating huge number of situations (combinations of demand flow rates, traffic composition, geometric conditions, and work zone length and duration), a method integrating the concept of deterministic queuing model and simulation is developed for estimating the queuing delay caused by lane closures (based on work zone configurations) on freeways. The traffic flow distribution over time and work zone capacity are the major input, from the model users to approximate queuing delays. The queuing delay in each time period is calculated based on the queue length accumulated from the previous time period. If the queue length is zero at time period i, the queuing delay $T_Q(i)$ incurred by flow rate $Q(i)$ can be obtained from equation 8.

$$T_Q(i) = t_a(i)Q(i) \tag{8}$$

where $t_a(i)$ representing average queuing delay can be observed based on V/C_w ratio as shown in figures 3, 4 and 5.

However, if there is a queue accumulating from the previous time periods ($q(i) > 0$), the queuing delay is determined based on flow rate $Q(i)$, work zone capacity C_w and the duration to discharge $q(i)$. Two situations are considered to approximate the queuing delay and discussed below.

Situation 1 : $q(i) + Q(i) > C_w$

If the delay experienced by the first and the last vehicles of the studied time period passing through the work zone can be determined, the total queuing delay incurred by $Q(i)$ at time period i can be formulated as follows

$$T_Q(i) = [\frac{t_F(i) + t_L(i)}{2}]Q(i) \tag{9}$$

where $t_F(i)$ and $t_L(i)$ represent queue delays experienced by the first and the last vehicles in $Q(i)$ before entering the work zone, respectively.

Assuming that the vehicles in the queue entering the work zone are based on a first come first serve basis, the queue delay experienced by the first vehicle of $Q(i)$ entering the work zone is equal to the discharging time of queuing vehicles accumulated from the previous time period ($i-1$). Therefore, $t_F(i)$ is

$$t_F(i) = \frac{q(i)}{C_w} \tag{10}$$

In order to find the queuing delay of the last vehicle, the average queuing delay $t_a(i)$ incurred by $[q(i)+Q(i)]$ in time period i for two, three, and four-lane cases can be observed from the curves shown in figures 3, 4, and 5, respectively. After determining the average queuing delay, the total queuing delay $T_{Q+q}(i)$ in time period i can be obtained from equation 9.

$$T_{Q+q}(i) = [q(i)+Q(i)]t_a(i) \tag{11}$$

In order to simplify the vehicle delay diagram shown in figure 6, the queue delay is assumed to be increasing linearly as the demand increases. The total queue delay $T_{Q+q}(i)$ can be formulated as

$$T_{Q+q}(i) = \frac{1}{2}[q(i)+Q(i)]t_L(i) \tag{12}$$

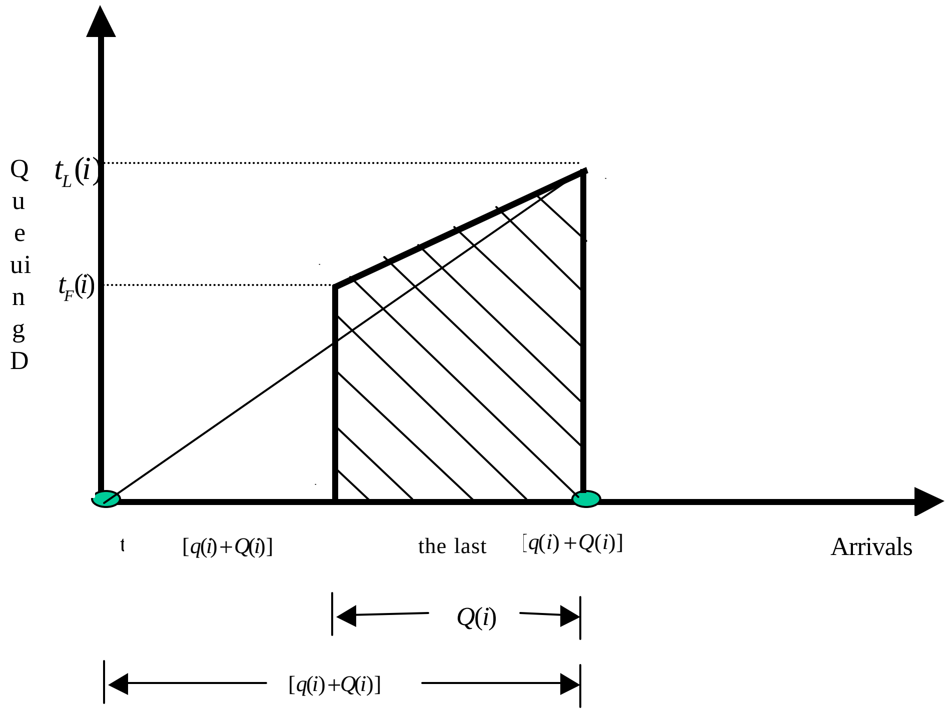


Figure 6. Queuing delay of Vehicles in [q(i) + Q(i)]

By substituting $T_{Q+q}(i)$ in equation 12 into equation 11, the queuing delay experienced by the last vehicle is

$$t_L(i) = 2t_a(i) \tag{13}$$

Based on the values of $t_F(i)$ and $t_L(i)$ obtained from equations 9 and 12, the total queuing delay $T_Q(i)$ can be determined from equation 9.

Situation 2: $q(i)+Q(i)\le C_w$

If the sum of $q(i)$ and $Q(i)$ in time period i is less than or equal to work zone capacity C_w, the volume of $q(i)+Q(i)$ will be discharged by the end of this time period. Thus, only a fraction of approaching demand in time period i will be affected by $q(i)$. The time (hour) t required to discharge the queue is

$$t=\frac{q(i)}{[C_w-Q(i)]} \tag{14}$$

Thus, the total number of vehicles $Q_a(i)$ affected by $q(i)$ in time period i is:

$$Q_a(i)=tQ(i) \tag{15}$$

The queuing delay expressed by $Q_a(i)$ can be estimated by equation 9, in which $t_L(i)$ will be estimated by equation 16.

$$t_L(i)=2t_a(i)\frac{t}{t_p(i)} \tag{16}$$

In equation 16, $t_a(i)$ can be observed from figures 3, 4 and 5, while $V/C_w=1$; and $t_p(i)$ is the duration of time period i.

The queuing delay incurred by the rest of , say $(t_p(i)-t)Q(i)$, can be estimated from equation 8 after replacing $t_a(i)$ by $t_a(i)\frac{t}{t_p}$.

Calculation of Delays by Vehicle Types

The total delay T_D is the summation of moving and queuing delay incurred by all motorists traveling on the freeway during work zone activity hours. Assuming that a work zone activity on a freeway can not be removed on time. The extended duration covers from time period 1 to n. The resulting total delay T_D can be formulated as

$$T_D = \sum_{i=1}^{n} [T_M(i) + T_Q(i)] \tag{17}$$

Since the total delay is incurred by different types of vehicles (e.g., trucks and passenger cars), the delay can be categorized by types of vehicles in the traffic stream using the equation 17.

$$T_D^c = T_D X^c \tag{18}$$

where T_D^c is the total delay incurred by type c vehicles and X^c is the percentage of type c vehicles in the traffic stream.

Comparison of Estimated Queuing Delays

In order to observe the variation and compare the difference among the estimated queue delays obtained from CORSIM, the proposed method and the deterministic queuing model, the total delays caused by various work zone configurations are analyzed and shown in table 14. The flow rates and capacities over four hours (4 time periods) are given in table 13. The total queue delay estimated by CORSIM is obtained by averaging total delays generated by ten simulation runs with different random number seeds. From table 14, it is shown that the queue delay obtained from the proposed method is closure to that observed from CORSIM. However, the deterministic queuing model significantly underestimates the total queuing delay. Since the delay caused by shock wave and acceleration/deceleration, while vehicles are approaching the work zone is not taken into consideration by the deterministic model, the total queuing delay thus is underestimated.

Table 13: Input variables

Time Period (1 hr)	**Case 1 (0% truck)**		**Case 2 (0% truck)**		**Case 3 (0% truck)**	
	Demand (pcph)	**Capacity (pcph)**	**Demand (pcph)**	**Capacit y (pcph)**	**Demand (pcph)**	**Capacity (pcph)**
1	1740	1450	5200	4000	7205	6550
2	1740	1450	4000	4000	7205	6550
3	1450	1450	3600	4000	5895	6550
4	870	1450	3200	4000	5895	6550

Table 14. Estimated Delays from Different Methods

Methods	**Total Delay (veh-hr)**		
	Case 1	**Case 2**	**Case 3**
Proposed Model	1831.46	4283.68	3881.63
Simulation Model	1810.18	4451.6	3707.83
Deterministic Model	1450	3200	2620

Procedure for Estimating Work Zone Delay

Step 1: Estimation of Work Zone Capacity

Determine f_W referring table 9

Determine f_{HV} using equation 3 and referring tables 5 through 8.

Determine f_P referring table 10

Determine Work Zone Capacity using equation 2, which can be obtained by simulation or by using the volumes suggested by 1994 Highway Capacity Manual.

Step 2: Estimation of Moving Delay

Determine $q(i)$ using equation 5.

If $Q(i)+q(i)\leq C_W$, Find $T_M(i)$ using equation 4.

If $Q(i)+q(i)>C_W$, Find $T_M(i)$ using equation 6.

Step 3: Estimation of Queue Delay

If $q(i)=0$, Determine $T_Q(i)$ using equation 8.

If $q(i)\neq 0$

If $Q(i)+q(i)>C_W$, Determine $t_F(i)$ using equation 10.

Determine $t_L(i)$ using equation 13.

Determine $T_Q(i)$ using equation 9.

If $Q(i)+q(i)\leq C_W$,

Determine t using equation 14.

Determine $t_F(i)$ using equation 10.

Determine $t_L(i)$ using equation 13 (use V/C ratio 1)

Determine $T_{Q1}(i)$ using equation 9, where $Q_1(i)=tQ(i)$

Determine $T_{Q2}(i)$ using equation 8, where $Q_2(i) = (1-t)Q(i)$ and $\frac{V}{C} = \frac{Q(i)}{C_W}$

Step 4: Calculation of Delays by Vehi8le Types

Calculate the total delay using equation 17.

Determine the total delay by vehicle types using equation 18.

Sample Calculations

In order to illustrate the use of the developed model, several typical examples with hypothetical conditions are discussed below.

Example 1:

Number of Lanes per direction = 2

Number of lane closed = 1

Work zone Length = .5 mile

Work zone capacity = 1450 pcph

Duration of work = 10 hours

Average approaching speed = 70 mph

Average work zone speed = 50 mph

Flow rates over 10 hours are shown in table 15.

Truck = 0%, 5%, 10%, 15%, and 20%

Grade = 0%

The work zone is scheduled to finish at 5:00 AM; however all lanes of this work zone are opened to the public until 3:00 PM. Details of moving and queuing delays are determined. Moving delay is shown in table 16, while Queuing delay with 0, 5, 10, 15, and 20% truck are shown in tables 17, 18, 19, 20, and 21, respectively. The total delay includes the queuing and moving delays caused by trucks and cars is summarized in table 22 and show in figure 7. Queuing delays at all time periods with

$V / C_w \geq 1$ for 0, 5, 10, 15, and 20 % trucks are shown in figures 8, 9, 10, 11, and 12, respectively.

Table 15. Flow rates (vph) over time

Time Period	**Duration (hr)**	**Demand Flow Rate (vph)**
1	5:00 –6:00	800
2	6:00-7:00	1000
3	7:00-8:00	1200
4	8:00-9:00	1600
5	9:00-10:00	1500
6	10:00-11:00	1200
7	11:00-12:00	1000
8	12:00-13:00	700
9	13:00-14:00	700
10	14:00-15:00	700

Table 16. Moving delay estimation (Example 1)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time Period (i)	**Duration (hrs)**	C_w^p **(vph)**	**Flow Rate** $Q(i)$ **(vph)**	**Queue Length** $q(i)$	$Q(i)+q(i)$	$T_M(i)$ **(veh-hr)**	$\sum_{i=1}^{10} T_M(i)$ **(veh-hr)**
1	5:00-6:00	1381	800	0	800	2.29	
2	6:00-7:00	1381	1000	0	1000	2.86	
3	7:00-8:00	1381	1200	0	1200	3.43	
4	8:00-9:00	1381	1600	0	1600	3.95	
5	9:00-10:00	1381	1500	219	1719	3.95	29.71
6	10:00-11:00	1381	1200	338	1538	3.95	
7	11:00-12:00	1381	1000	157	1157	3.31	
8	12:00-13:00	1381	700	0	700	2.00	
9	13:00-14:00	1381	700	0	700	2.00	
10	14:00-15:00	1381	700	0	700	2.00	

Table 17. Queuing delay estimation ($C_w^p = \mathbf{1450}$, 0 % Truck)

(1) Time Period (I)	(2) Flow Rate $Q(i)$ (vph)	(3) $q(i)$ (veh)	(4) $\frac{Q(i)+q(i)}{C_w^p}$	(5) $t_F(i)$ (min)	(6) $t_L(i)$ (min)	(7) $t_a(i)$ (min/veh)	(8) $T_Q(i)$ (veh-min)	(9) $\sum_{i=1}^{10} T_Q(i)$ (veh-hr)
1	800	0	0.55	-	-	0.03	23.94	
2	1000	0	0.69	-	-	0.05	52.76	
3	1200	0	0.83	-	-	0.11	129.06	
4	1600	0	1.10	-	-	4.33	6920.11	
5	1500	150	1.14	6.21	11.73	-	13453.81	419.72
6	960	200	1.00	8.28	1.09	-	4495.42	
6	240	0	0.83	-	-	011	20.65	
7	1000	0	0.69	-	-	0.05	52.76	
8	700	0	0.48	-	-	0.02	11.49	
9	700	0	0.48	-	-	0.02	11.49	
10	700	0	0.48	-	-	0.02	11.49	

Table 18. Queuing Delay Estimation ($C_w^p = 1403$, 5 % Truck)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Time Period (I)	**Flow Rate $Q(i)$ (vph)**	$q(i)$ **(veh)**	$\frac{Q(i)+q(i)}{C_w^p}$	$t_F(i)$ **(min)**	$t_L(i)$ **(min)**	$t_a(i)$ **(min/veh)**	$T_Q(i)$ **(veh-min)**	$\sum_{i=1}^{10} T_Q(i)$ **(veh.-hr)**
1	800	0	0.57	-	-	.03	26.70	
2	1000	0	0.71	-	-	0.06	55.45	
3	1200	0	0.85	-	-	0.13	158.36	
4	1600	0	1.13	-	-	5.56	8892.19	
5	1500	185	1.19	7.86	16.51	-	18277.29	616.547
6	1200	271	1.04	11.48	4.13	-	9367.61	
7	140	56	1.00	2.38	0.18	-	173.42	
	860	0	0.71	-	-	0.06	6.49	

8	700	0	0.49	-	-	0.02	11.78	
9	700	0	0.49	-	-	0.02	11.78	
10	700	0	0.49	-	-	0.02	11.78	

Table 19. Queuing Delay Estimation ($C_w^p = 1381$, 10 % truck)

(1) Time Period (I)	(2) Flow Rate $Q(i)$ (vph)	(3) $q(i)$ (veh)	(4) $\frac{Q(i)+q(i)}{C_w^p}$	(5) $t_F(i)$ (min)	(6) $t_L(i)$ (min)	(7) $t_a(i)$ (min/veh)	(8) $T_Q(i)$ (veh-min)	(9) $\sum_{i=1}^{10} T_Q(i)$ (veh.-hr)
1	800	0	0.58	-	-	.04	29.46	
2	1000	0	0.72	-	-	0.06	59.07	
3	1200	0	0.83	-	-	0.16	187.66	
4	1600	0	1.16	-	-	6.79	10864.28	
5	1500	219	1.24	9.52	20.99	-	22880.89	835.96
6	1200	338	1.11	14.69	9.57	-	14558.52	
7	410	157	1.00	6.83	0.56	-	1524.07	
	590	0	0.72	-	-	0.06	14.31	
8	700	0	0.51	-	-	0.02	13.11	

9	700	0	0.51	-	-	0.02	13.11	
10	700	0	0.51	-	-	0.02	13.11	

Table 20. Queuing Delay Estimation ($C_w^p = 1355$, 15 % Truck)

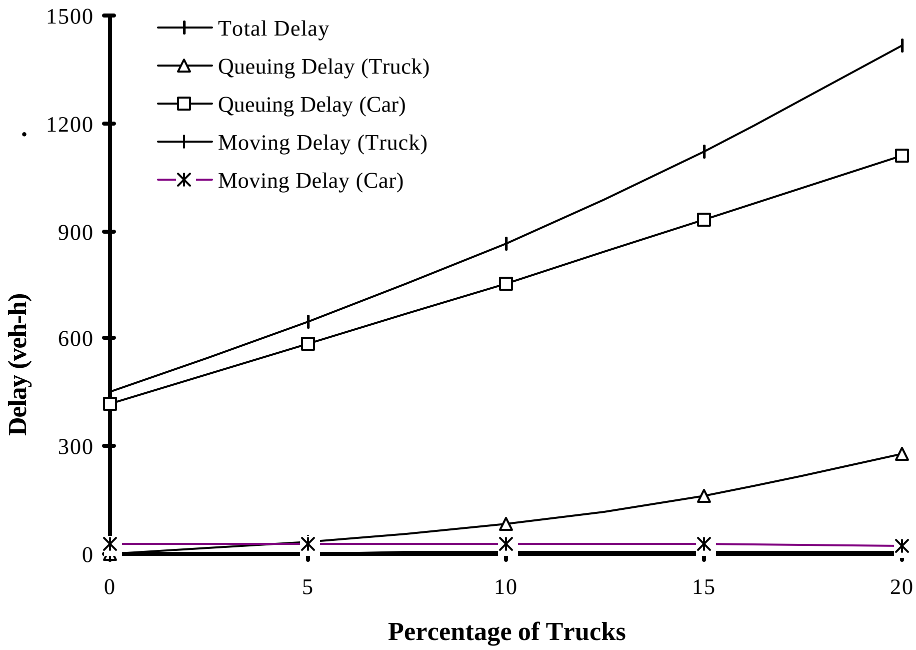
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Time Period (i)	**Flow Rate $Q(i)$ (vph)**	**$q(i)$ (veh)**	**$\frac{Q(i)+q(i)}{C_w^p}$**	**$t_F(i)$ (min)**	**$t_L(i)$ (min)**	**$t_a(i)$ (min/veh)**	**$T_Q(i)$ (veh-min)**	**$\sum_{i=1}^{10} T_Q(i)$ (veh.-hr)**
1	800	0	0.59	-	-	.04	32.22	
2	1000	0	0.74	-	-	0.06	62.69	
3	1200	0	0.89	-	-	0.18	216.95	
4	1600	0	1.19	-	-	8.02	12836.35	
5	1500	251	1.30	11.17	25.42	-	27442.20	1092.71
6	1200	402	1.19	17.90	16.20	-	20457.63	
7	730	254	1.00	11.28	0.99	-	4456.49	
	270	0.00	0.74	-	-	0.06	12.45	
8	700	0.00	0.52	-	-	0.02	15.22	
9	700	0.00	0.52	-	-	0.02	15.22	
10	700	0.00	0.52	-	-	0.02	15.22	

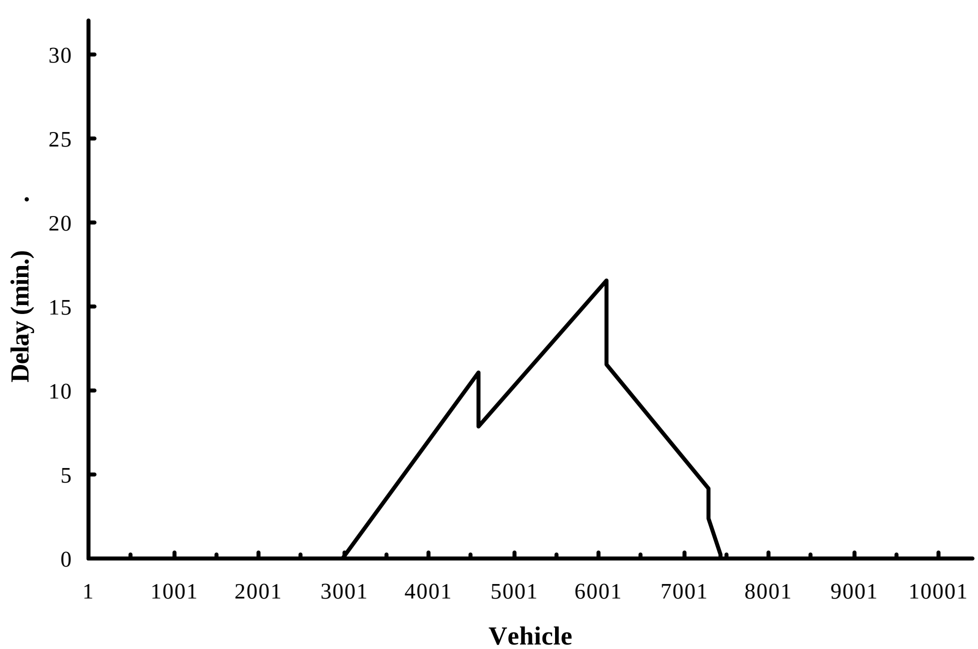
Table 21. Queuing Delay Estimation ($C_w^p = 1311$, 20 % Truck)

(1) Time Period (i)	(2) Flow Rate $Q(i)$ (vph)	(3) $q(i)$ (veh)	(4) $\frac{Q(i)+q(i)}{C_w^p}$	(5) $t_F(i)$ (min)	(6) $t_L(i)$ (min)	(7) $t_a(i)$ (min/veh)	(8) $T_Q(i)$ (veh-min)	(9) $\sum_{i=1}^{10} T_Q(i)$ (veh.-hr)
1	800	0	0.61	-	-	.04	34.26	
2	1000	0	0.76	-	-	0.07	66.31	
3	1200	0	0.91	-	-	0.24	292.18	
4	1600	0	1.21	-	-	9.21	14736.27	
5	1500	282	1.35	12.83	30.49	-	32491.54	1384.28
6	1200	464	1.26	21.10	22.42	-	26113.18	
7	1000	345	1.02	15.72	2.81	-	9265.13	
8	28	27	1.00	1.24	0.06	-	20.10	
8	672	0	0.53	-	-	0.02	0.73	
9	700	0	0.53	-	-	0.02	17.33	
10	700	0	0.53	-	-	0.02	17.33	

Table 22: Total, Queuing and Moving Delays

Percentage Truck	Total Delay (veh-hr)	Queuing Delay		Moving Delay	
		Truck	Car	Truck	Car
0	449.43	0	419.72	0	29.71
5	646.25	30.827	585.713	1.4855	28.2245
10	865.67	83.596	752.364	2.971	26.739
15	1122.42	163.906	928.8	4.4565	25.2535
20	1413.99	276.856	1107.424	5.942	23.768


Total Delay
Queuing Delay (Truck)
Queuing Delay (Car)
Moving Delay (Truck)
Moving Delay (Car)
Delay (veh-h)
0
300
600
900
1200
1500
0
5
10
15
20
Percentage of Trucks


Delay (min.)
0
5
10
15
20
25
30
1
1001
2001
3001
4001
5001
6001
7001
8001
9001
10001
Vehicle

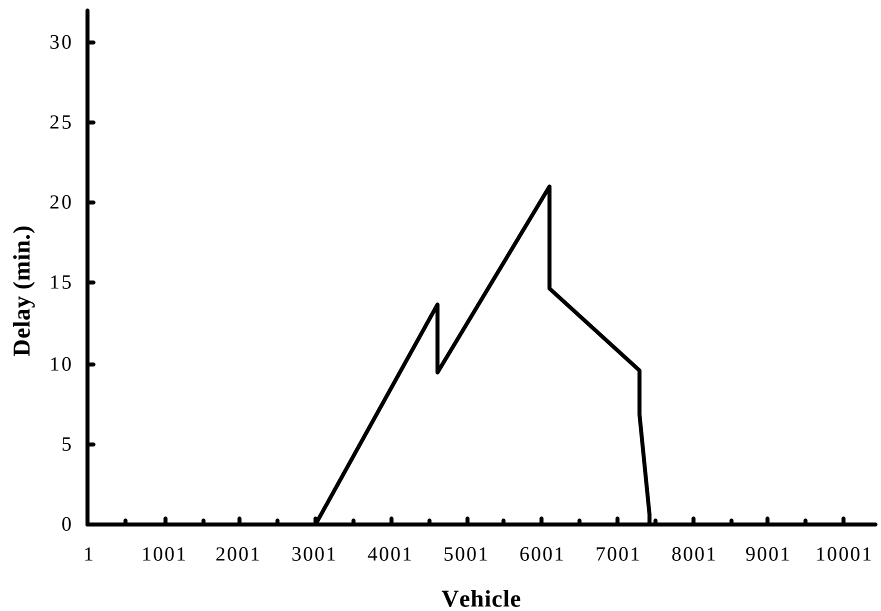


Figure 9. Vehicle arrival vs. queue delay (5% truck).

Figure 10. Vehicle arrival vs. queue delay (10% truck).

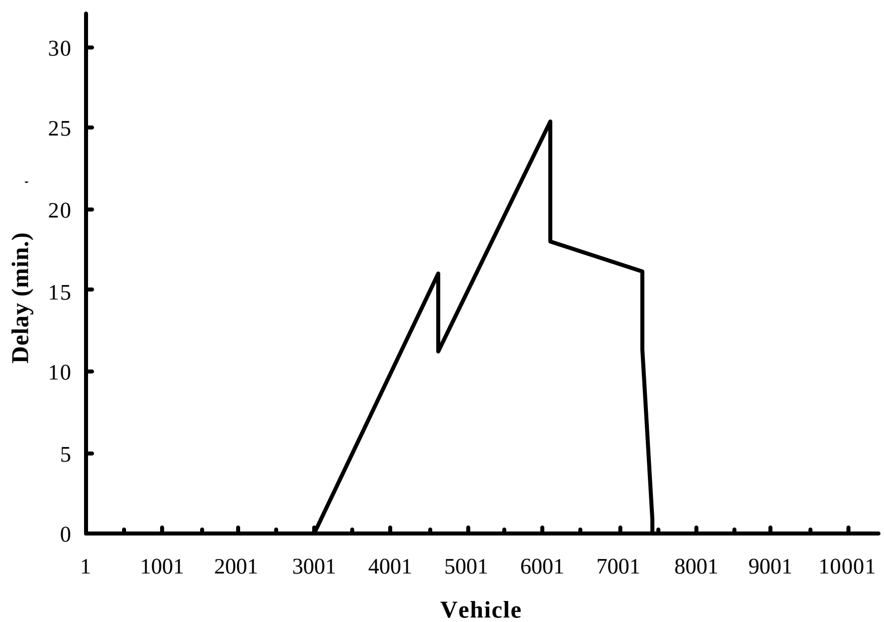


Figure 11. Vehicle arrival vs. queue delay (15% truck).

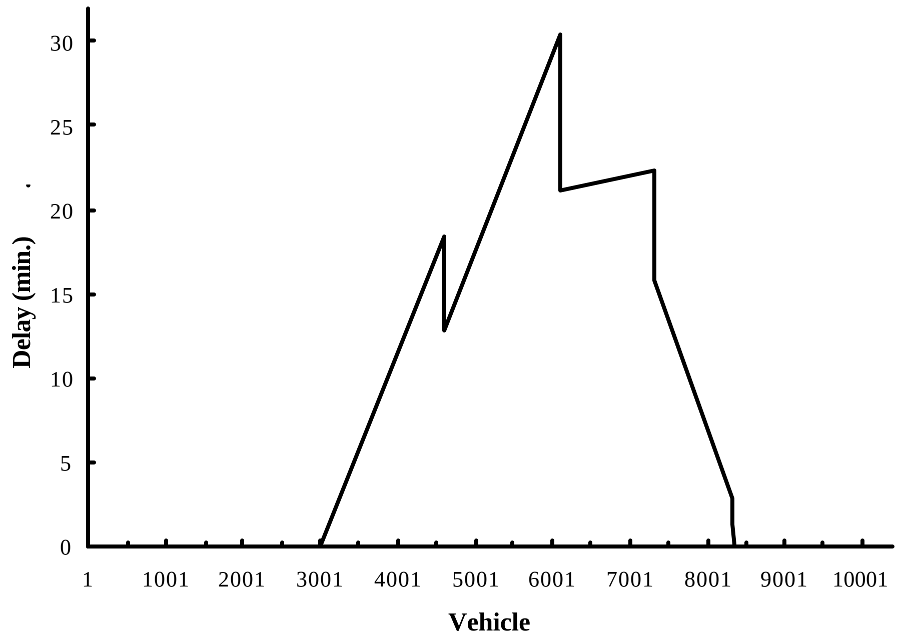


Figure 12. Vehicle arrival vs. queue delay (20% truck).

CHAPTER 4. FORMULATING COSTS & MODELS

Economic Implications and Model

One should note the obvious; movement between points takes time. An individual who travels routinely to work and back budgets for travel time based on experience (allowing a variance, *v*). Thus, he expects that it will take $X \pm v$ minutes to travel and he plans accordingly. However, if the trip breaks away from the travel pattern and takes more than this budgeted amount of time, the individual becomes agitated. Obviously, more so if it takes even more time.

Normal road travel condition could change due to man-made interference. Introducing a work zone along a route could change the trip time. The additional time beyond the budgeted time is the issue of concern. This additional time and its value are the subject of this study. There is also the indirect cost of fuel, etc. but we do not address them. Thus, lane closing could impact directly on this additional time. How does the timing of lane closing effect the trip time during the course of a 24-hour daily cycle? Is it uniform?

In reviewing this issue one finds that there are various studies to consider. However, many of them deal with the UK and other countries, not the US.[(23)] Very few address all the issues of our concern. Thus, we follow with a review of the literature, present the model developed, and provide a methodology that provides an answer to improve transport efficiency.

Background

Lane closing effects all road users. The economic impact depends on the economic agents' socio-economic and demographic characteristics, time of day, duration of lane closing, type of economic activity the agents are engaged in, road characteristics, etc. In order to determine the economic impact of lane closing on economic agents, one needs to know more than the characteristics of the agents. These characteristics have a direct impact on the economic value the agents place on time and a direct cost.

In unraveling those variables, several issues need to be considered. One needs to know if the driving is for leisure or is it work related? Is traveling through the restricted area a part of the work assignment? Is it traveling to or from work? Is the traveling occasional? The analysis will concentrate on additional travel time and cost due to lane closing. This could be based on the time value of travel in general. However, it is preferred if the economic agents' distribution by group and income is available. Thus, it will be considered.

Two quite distinct methodologies have been developed for time evaluation, the distinction being made between time saved in the course of employment and time saved during non-work travel. The distinction is drawn because work time involves lorry drivers, seamen, pilots, etc., not simply in giving up leisure but also in incurring some actual disutility from the work undertaken. Hence, if they could do the same amount of work in less time, these people would be able to enjoy more leisure and also suffer less disutility.[24]

Economic Analysis

Using the traditional economic idea that workers are paid according to the value of their marginal revenue product (MRP), the employer will pay them for the marginal time in addition to doing the job. Thus, one can equate the marginal savings with the marginal wage rate. A different way considers the opportunity cost of time. Delays in getting to work would reduce production. A delay in executing work reduces productivity. Thus, the value of the reduced output is the value of lost time. Again, this associates the MRP and marginal wage paid and can be assumed equal to the full value of the hourly wage. Thus, “Official UK policy is to value work travel time savings as the national average wage for the class of transport user concerned plus the associated cost of social insurance paid by the employer and a premium added to reflect overhead.”[24]

The above, assumed by the employer, implies that employees perceive the disutility of travel time at work hours to be equal to the disutility of work. However, employees might not see it this way. They might consider it a break. Therefore, the value of time should be less than the wage rate base. Others might perceive that it is the opposite. This argument makes it difficult using wage rate as a base.

A different approach to determine the value of non-work travel time is rooted in the behavioral approach. This is based on revealed-preference and stated-preference approaches. The revealed-preference approach considers a trade-off where one is willing to pay in order to save time. This could provide an implicit value of time. Empirical studies frequently use this approach. The trade-off variables frequently used include: route, mode of travel, speed of travel, location of home and work, and destination of travel. Most of these studies address commuters as their subjects.

Using this approach, Waters reports the following (table 23).[25] The most striking outcome that could be used for the purpose of evaluating lane closing on an interstate highway is reported in the USA “interurban (auto)” listings. Waters shows that the *value of time as percent of wage rate* was 86 percent (1970) and 82 percent (1987). In the UK it was 73 percent (1975). For USA “leisure (auto)” the value was 63 percent (1975) and 52-254 percent (1985). In Canada it was 116-165 percent (1990).

Using these figures based on the revealed preference approach, a conservative *value of time as percent of wage rate* would be about 75 percent or better.

Table 23. Computation of estimated values of travel time savings.

Study	country	Value of time as % of wage rate	Trip purpose	Mode
Beesley (1965)	UK	33-50	Commuting	Auto
Quarmby (1967)	UK	20-25	Commuting	Auto, Transit
Stopher (1968)	UK	21-32	Commuting	Auto, Transit
Oort (1969)	USA	33	Commuting	Auto
Thomas & Thompson (1970)	USA	86	Interurban	Auto
Lee & Dalvi (1971)	UK	30	Commuting	Bus
		40	Commuting	Auto
Wabe (1971)	UK	43	Commuting	Auto, Subway
Talvitte (1972)	USA	12-14	Commuting	Auto, Transit
Hensher & Hotchkiss (1974)	Australia	2.70	Commuting	Hydrofoil, Ferry
Kraft & Kraft (1974)	USA	38	Interurban	Auto
Mcdonald (1975)	USA	45-78	Commuting	Auto, Transit
Ghosh et al (1975)	UK	73	Interurban	Auto
Guttman (1975)	USA	63	Leisure	Auto
		145	Commuting	Auto
Hensher (1977)	Australia	39	Commuting	Auto
		35	Leisure	Auto
Nelson (1977)	USA	33	Commuting	Auto
Hauer & Greenough (1982)	Canada	67-101	Commuting	Subway
Edmonds (1983)	Japan	42-49	Commuting	Auto, Bus, Rail
Deacon & Sonstelie (1985)	USA	52-254	Leisure	Auto
Hensher & Truong (1985)	Australia	105	Commuting	Auto, Transit
Guttman & Menashe (1986)	Israel	59	Commuting	Auto, Bus
Fowkes (1986)	UK	27-59	Commuting	Rail, Coach
Hau (1986)	USA	46	Commuting	Auto, Bus
Chui & Mcfarland (1987)	USA	82	Interurban	Auto
Mohring et al (1987)	Singapore	60-129	Commuting	Bus
Cole Sherman (1990)	Canada	93-170	Commuting	Auto
		116-165	Leisure	Auto

Source: Waters [25] which contains full references to studies cited. Reprinted from Button, p. 55.

The stated preference approach is where travelers are asked hypothetical questions about the trade-offs between modes of transportation that they would be willing to make. Overall, travelers revealed that they value non-work travel time at 15-45% of hourly income. However, Thomas (1967) found, using USA data, that non-work travel time is valued at 40-83% of average hourly income.[26]

Thus, the behavioral approach suggests that the non-work timesaving is valued below average hourly income. In the USA it is also conservative to use the *value of time as percent of wage rate* to be 75 percent.

The value of time should be part of the standard transport analysis for the purpose of investment analysis. This is the case in the UK, which uses information developed by the UK Department of the Environment, UK Department of Transport and the COBA 9 Manual.[24] In the US, reviewing our survey comments indicates that frequently government agencies used the 1977 "Red Book" in estimating time value. The frequently used figure stated in the survey was $6 an hour. It seems to be too low. Economists frequently used one half of the hourly salary for travel time to work.

In general, an economic agent's value of time differs by activity and income. For example, an hourly paid trucker who is hauling goods is a known expense to his employer. The trucking association calculates the hourly rate for a for-hire trucker at $21 an hour (30 percent of total) or $28.35 with 35 percent overhead, and the independent truckers calculate their value at $27.50 an hour (70 percent of total) or $37.125 with 35 percent overhead.[27] This does not include fuel or other indirect expenses. However, the trucking associations value their time at about $50 an hour. It is difficult to determine the value of an executive traveling to or back from work. Obviously, it will be different than the time of a common laborer. The average hourly income of executive groups ranges from $20 to $40 and even more depending on the executive.[1] Thus, road users should be distinguished by income groups. The time value for each group needs to be estimated using traffic reports and/or surveys. Each group size needs to be estimated to determine its weight in the total. Thus, a weighted average needs to be established to estimate the lane closing social cost. The estimates will have to be sensitive to the time of day as well.

In the absence of this overall weighted average, one looks for some other base. Since "production workers' hourly earnings" are reported cyclically, one can use them as a base. This value, reported for a long time, is over $14 an hour.[2] However, there are also those who earn minimum wage and those who earn much more and those who travel for leisure where their time value is very high. Could one use the government allowance of 31.5¢ a mile to be an indicator for the value of time? Assuming that this is reasonable, a 60-mile an-hour trip on the interstate would equal to $18.90 in an hour.

Observing human behavior illustrates the individual sensitivity to the effective use of time. Thus, it impacts this study and the value of time. An observer of human nature would notice that individuals try to conserve travel time through the increase in use of telecommuting and the Internet. Vehicles are allowed to use higher speeds, better highways, public transportation and communications along the highway to reduce travel time and congestion. The use of cars has increased because people perceive the cars as an extension of home. Supporting evidence of this trend could be noticed in the increase of income and the increase of car use, more expensive vacations, and the increased use of restaurants. The aggregate travel time expenditure on travel per head increased roughly proportional to income.(27) All are indications of premium value on time.

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[1] A Search of "Hourly Wages" in the internet site: WWW. BLS.gov for NJ

[2] Various issues of NJ Economic Indicators, and various issues of NJ Department of Labor News Releases.

In conclusion, one can use the average hourly earning of $14 as a minimum figure. The amount should be larger, probably close to an average of $20 an hour. Thus, using the figure established before for the *value of time as percent of wage rate* of 75% suggests that an hour delay on the road is equal to at least $10.50, but more likely $15 an hour, given the composition of drivers on the NJ highway.

The actual value of delay time should be reviewed very closely since it cuts into work time at full cost. Thus, one should consider delayed travel time at 100% of value of time at work.

Methodology

Using the principles stated above, one can establish the following methodology:

In general, without distinguishing between income groups and with an average hourly earning, we get:

Total Delay Cost = Delay time per vehicle x average earning per minute x number of vehicles or

$$DC = DT/V \times AHE/60 \times n \qquad (19)$$

Where:

DC = Delay Cost

DT = Delay Time

V = Vehicle

AHE = Average Hourly Earning

n = number of vehicles

Alternatively, using a more detailed method which distinguishes between income groups, it modifies the above by including the average income per group and its weight.

Total Delay Cost = Delay time per vehicle x Sum [average hourly earnings per income group/60 minutes x number of vehicles in this income group] or

$$DC = DT/V \times \sum_{i=1}^{k} (AHEG_i/60 \times wn) \qquad (20)$$

Where:

AHEG = average hourly earning per income group

wn = number of vehicles in the income group

i is from 1 to k groups

In both cases, the delay is a function of: time of day, day of the week, number of lanes closed, road characteristics and grade, etc.

Illustration

Using the example of a 2-lane road with one lane closed along a 0.5-mile work zone with work zone capacity of 1450 pcph and 10 hours duration at an average approach speed of 70mph and average work speed of 50mph, before (tables 15 and 22), the queuing and moving delays were calculated for a total delay. The delays are subject to the number of trucks in the system. Using these results and the *value of time as percent of wage rate* at the range of $10.5 to $15 an hour, one can estimate the cost of the delay.

Table 24. Total queuing and moving delay costs

Percentage Truck	Total Delay (veh-hr)	Cost per hour @			Queuing Delay		Moving Delay	
		$6.00*	$10.50	$15.00	Truck	Car	Truck	Car
0	449.43	2696.58	4719.02	6741.45	0	419.72	0	29.71
5	646.25	3877.50	8061.97	10824.69	30.827	585.713	1.4855	28.2245
10	865.67	5194.02	12508.93	16014.90	83.596	752.364	2.971	26.739
15	1122.42	6734.52	18435.69	22728.93	163.906	928.8	4.4565	25.2535
20	1413.99	8483.94	26017.40	31107.70	276.856	1107.424	5.942	23.768

*The $6 an hour is used across the board.

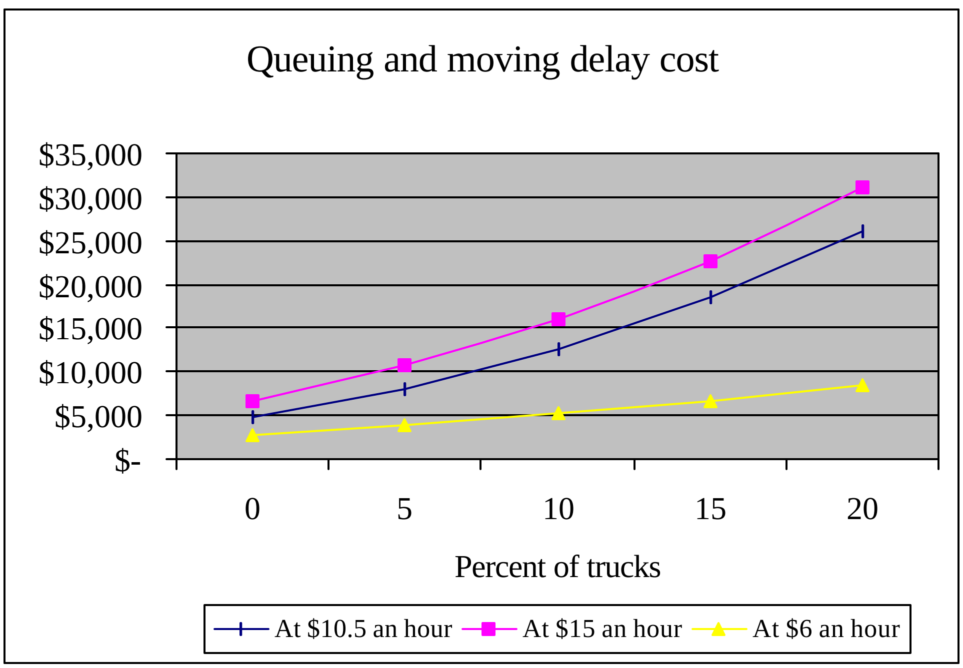

Figure 13. Illustrative example.

Thus, without trucks in the system the cost ranges between $4,719 and $6,741 an hour (table 24). At the present time with $6 an hour charges, the lane closing charges would have been $2,696.58, which is only 57 percent of the calculated minimum. However, one can also show that the cost is much larger with trucks in the system. Taking the average scenario of 10% trucks with an opportunity cost per truck of $50 an hour and cost per other vehicle of $15 an hour, the total delay cost can reach $16,014.84.[3] This is 3.4 times larger than the smaller amount before and almost 6 times larger than the present practice.

Under no circumstances should the road delay charges be less than the minimum of $4,719 per hour. This amount should be modified depending on the type of road and the road use. The road charges should reflect the social cost of closing a lane. Even the minimum charges will recognize this economic cost and provide for better distribution of resources.

In order to determine the cost more accurately one needs to survey the road users in order to determine:

- the mix of users between trucks, buses and cars,
- the income groups of each user category,
- the congestion level per time of day,
- the vehicle hour delay per hour of the day.

Thus, there is a need to obtain a weighted average of users and their value of time to further modify the calculation.

[3] Determined by 83.596 x $50 + 752.364 x $15 + 2.971 x $50 + 26.739 x $15 = $16,014.84

CHAPTER 5. SUMMARY & CONCLUSIONS

Summary & Conclusions

The methodology defined in this research considers the traffic characteristics of specific work zone scenarios and highway characteristics in order to estimate traffic delays for alternative scenarios. CORSIM, a microscopic traffic simulation model, was used to mimic the traffic operation at work zones and thus estimate queuing delays at work zone. Specifically, queuing delay was estimated by combining the simulation results and a deterministic model, while a mathematical model was developed for estimating moving delay.

Lane occupancy charges were then defined using the delay as a function of: time of day, day of the week, number of lanes closed, road characteristics and grade, etc. In addition the methodology for defining lane occupancy charges considers traffic characteristics and demographics of road users income. Alternatively, average values of income may be considered for simplifying the analysis. As it appears from the illustrative example, the methodology is sensitive to the percentage of trucks using the roadway since delays on the moving of goods will provide significant impact on both traffic and revenue loss.

As indicated in chapter 4, in order to determine the lane occupancy charges accurately, one needs to survey the road users in order to determine:

- the mix of users between trucks, buses and cars,
- the income groups of each user category,
- the congestion level per time of day,
- the vehicle hour delay per hour of the day.

Alternatively a weighted average of users and their value of time may be used to simplify the calculations.

The methodology developed and presented herein is flexible enough to consider any model and eventual assumptions that NJDOT engineers feel to better represent the specific conditions where lane occupancy charges are applied.

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