A MODEL FOR THE ECONOMIC ANALYSIS OF ROAD PROJECTS IN AN URBAN NETWORK WITH INTERRELATED INCREMENTAL TRAFFIC ASSIGNMENT METHOD

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ABSTRACT

In an urban network, any change to the capacity of a road or an intersection will generally result in some traffic changing its route. In addition the presence of intersections creates the need for frequent stops. These stops increase the fuel consumption by anywhere between thirty to fifty percent as evidenced by published standardised vehicle fuel consumption figures for urban and for country driving. Other components of vehicle operating costs such as tyre and brake wear and time costs will also be increased by varying amounts. Yet almost all methods in use for economic evaluation of urban road projects use open road vehicle operating costs (sometimes factored to represent an average allowance for stopping at intersections) for one year or sometimes two years in the analysis period and then make assumptions about how the year by year road user benefits may change throughout the period in order to complete the analysis.

This thesis will describe a system for estimating road user costs in an urban network that calculates intersection effects separately and then adds these effects to the travel costs of moving between intersections. Daily traffic estimates are used with a distribution of the flow rate throughout the twenty-four hours giving variable speed of travel according to the level of congestion at different times of the day. For each link, estimates of traffic flow at two points in time are used to estimate the year-by-year traffic flow throughout the analysis period by linear interpolation or extrapolation. The annual road user costs are then calculated from these estimates. Annual road user benefits are obtained by subtracting the annual road user costs for a modified network from the annual road user costs for an unmodified network. The change in the road

network maintenance costs are estimated by applying an annual per lane maintenance cost to the change in lane-kilometres of road in the two networks. The Benefit Cost Ratio is calculated for three discount rates. An estimate of the likely range of error in the Benefit Cost Ratio is also calculated.

The system is modular and is fully integrated with a commercial traffic forecasting package in common use. Provision is made to allow the user to construct alternate distributions of the rate of traffic flow throughout the day and to construct daily speed versus traffic flow curves necessary for the traffic estimation phase. This means that the same assumptions about traffic speed that are used during the traffic estimation phase are also used to estimate the road user operating costs. Primary inputs into the system are the unit prices for the components that make up road user costs such as the purchase price of vehicles, cost of vehicle repairs, tyres, fuel and oil and average hours of use per year. This information is then combined with network information of traffic composition, speed versus hourly rate of flow for each type of vehicle, hourly and daily road capacities, type of road and speed limit to develop aggregate cost per kilometre of travel for the traffic stream versus the ratio of traffic flow to road capacity. The final step is to read the network files containing the estimated traffic for the modified and unmodified networks, the capital cost of the road works, the three discount rates to be used and to compute the benefit cost ratios.

The present system is a single mode road based system. With a relatively small amount of work it can be upgraded to a full multi mode system with the option of continuing to use it as a single mode system. A literature search conducted by ARRB Transport

Research in 1997 for Austroads for a suitable urban evaluation system (Tsolakis et al 1998) failed to find any other system that estimated the costs of stopping at intersections separately to the costs of travelling between intersections. Following that search, Austroads funded the updating of the vehicle operating consumption relationships in the system so that it could form the basis of improving the harmonisation of urban project evaluation (Lloyd & Tsolakis 2000).

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ABBREVIATIONS

AON	All or nothing traffic assignment
BCR	Benefit cost ratio
CBRds	Commonwealth Bureau of Roads
FYRR	First year rate of return
IRR	Internal rate of return
MP	Multi path traffic assignment
NPV	Net present value
PBP	Pay back period
RAC	Royal Automobile Club
RUC	Road user cost
VOC	Vehicle operating cost
vpd	vehicles per day

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DEFINITIONS

Accident costs – the cost to the community of road crashes.

Benefit – cost analysis - a systematic quantitative method of assessing the desirability of projects or policies when it is important to take a long term view of future effects and a broad view of possible side effects.

Capacity indicator – the number used to index the speed flow relationship for a link.

Commercial time cost - for commercial drivers and persons travelling on employers business, this is the paid rate for the employee plus the overhead cost of the employer.

Equilibrium – when used in the context of network modelling, equilibrium has three different meanings
a) at the link level – the speeds assumed during path building are the speeds that the assigned traffic could travel at
b) as an assignment procedure where the travel times on all routes between two points actually used by traffic are equal and less than on any unused route
c) the travel times used to build the trip tables are the same times that result from assigning the trip table to the network.
To avoid confusion over which meaning is intended, it is only used in this work to describe the assignment procedure. Other words are used when describing the other two cases.

Freight time cost – this is the time value of freight in transit.

Private time cost – this is a notional value derived from analyses of cases where people pay money to save travel time, such as paying a fee on a toll road rather than use a slower un-tolled road.

Road user costs – (RUCs) this is the sum of vehicle operating costs, freight and time costs.
RUCs are defined to include "… all costs in providing and operating a road transport system other than costs incurred by the agency responsible for providing and operating the road infrastructure" (Austroads 1997). RUCs comprise:

- vehicle operating costs (VOCs)

  ⇒ costs to own, operate and maintain vehicles which are incurred by vehicle operators;

- time costs (driver and passenger)

  ⇒ costs incurred by vehicle operators and occupants;

- payload costs (travel time, reliability of travel time, and protective packaging and damage)

  ⇒ costs of delay and damage to freight in transit;

- road crash costs

  ⇒ costs incurred by the community through insurance, health systems and policing; and

- external costs

⇒ environmental costs arising from traffic noise, air pollution and greenhouse emissions which are incurred by the general community.

Time costs – the cost of a person's time.

Vehicle operating costs – the costs of owning and operating a vehicle

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CHAPTER 1

INTRODUCTION

1.1 Introduction

There are many demands on funds for Transport projects to provide access to new developments, to improve the service in the existing network and to maintain the existing assets. To ensure that the available funds are spent wisely means that in addition to other criteria, projects must be subjected to an economic analysis. The results of these analyses are used to ensure value for money spent, and to rank projects in a priority order, as not all worthwhile projects can be funded. The aim of the work reported here is to:

- improve the accuracy of the assignment method
- to develop an evaluation process that is integral with the assignment process
- that both assignment and evaluation use the same assumptions about the distribution of traffic speeds throughout the day
- that both methods take into account as many of the variables as possible that influence traffic speed and cost
- the method to require limited increase in the workload of network coding and does not require any changes to the network file format
- that any increase in computer time is reasonable
- there is a process in place to build up macro level relationships such as speed flow and road user cost functions from basics

1.2 Economic analysis

Economic analysis is a systematic procedure that compares the capital cost of a project with the quantifiable benefits to the community and the continuing maintenance cost over the life of the project. The benefits to the community of a road project arise from a reduction in the road user and road crash costs. Road user costs are vehicle operating costs plus the time value of any freight carried plus the time costs of the vehicle occupants plus road crash costs. Vehicle operating costs are the costs of owning and operating a vehicle comprising interest and depreciation, repair and maintenance, tyre wear and fuel and oil costs. Other benefits of road projects are the reduction in greenhouse gases and the reduction in noise. As yet, there is no agreement on the monetary value to be put on these changes so for now they are listed in other benefits and are not taken into account in cost benefit analysis. Costing gas emissions could be added to the system when prices become available.

1.3 General traffic forecasting process

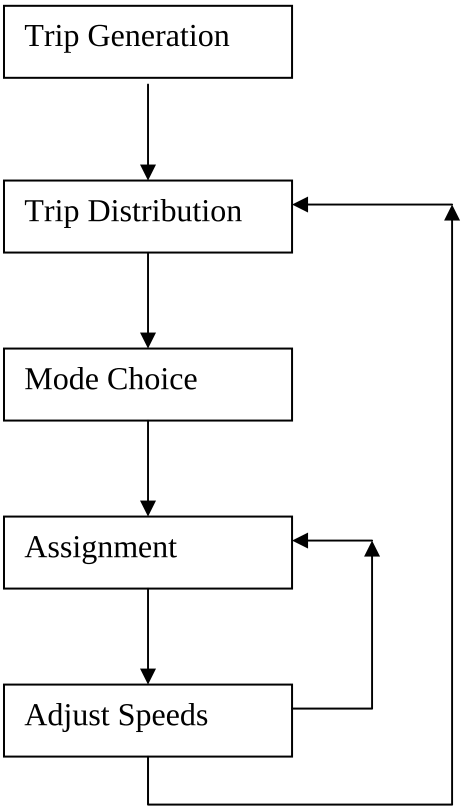
In an urban road network, there are usually alternate routes available for most journeys. The result is that if there is any change to the travel time between two points in the network, some traffic is likely to change its route, causing the traffic volume on some road links to increase and others to decrease. In consequence, it is not possible to forecast future traffic volumes on urban road links by projecting the trend in historical traffic counts.

The method used estimates travel in stages (or steps). This is known as the “four step process”. The first step (known as trip generation) is to estimate the need for people to travel, such as to work, to shops or to school etc. This is done from forecast estimates of the number of people living in an area and the number of jobs by type that are available in the area. To improve the accuracy of the end result, travel is divided into separate purposes, such as “to and from work”, “to and from school”, “to and from shops” etc. Usually, the number of purposes used increases with increasing city size. Two estimates of travel are needed, one estimate is for the number of trips that start in an area, and the second estimate is for the number of trips that terminate in an area.

The second step (known as trip distribution) is to connect the two ends of trips into a matrix. This is done with a gravity model and a matrix of the generalised cost (a combination of time and distance) that it takes to get from each area to every other area. The third step (known as mode choice) uses the time and cost of travel on each mode and car availability to divide the matrix of person travel into several matrices according to the mode of travel used to make the trip eg car driver, car passenger, public transport rider, or walk all the way. The fourth step (known as assignment) is to assign vehicle drivers to the road links that they will use to make the trip. Public transport riders are similarly assigned to the public transport mode (bus, rail) and route (bus route 72, 103 etc).

When all four steps are used, trip generation is for person travel. When the percentage of travellers riding public transport is very low, the trip generation and mode choice steps are combined into one by generating vehicle driver trips directly. These procedures are set out in the Bureau of Roads Traffic Assignment Manual (1964) (see Figure 1.1). These four steps are still current today, except that it is now possible to add a fifth step, the calculation of the change in road user costs from one iteration to the next to determine when enough iterations of building the trip tables have been done.

Figure 1.1 Building trip tables



The speed of travel on a road depends on the volume of traffic per lane attempting to use the road, the speed limit and the type of road. In an urban network, there are additional delays imposed by the presence of intersections. The amount of the delay at the intersection depends on the type of intersection control, the volume of traffic on each of the other approaches and the speed of approach. For public transport, travel time depends on the frequency of stops, on the speed of travel, and on the frequency of service, which depends on the number of passengers.

To initiate the traffic forecasting process, one has to assume values for speed of travel and service frequency and then build the trip matrices and assign the matrices to the networks. The speed of travel (and the service frequency) is then revised to suit the estimated volume of traffic. If the speed adjustment is too small or too great, the process is repeated until a satisfactory closure is achieved.

In the 1970s, the means available to a user to compare the differences between the initial assumed speed and the final speed were limited to comparing the system total distance and total hours of travel and the system average speed. Alternately one could run the compare matrices program to compare the matrices of zone to zone travel times. This resulted in a table of numbers of zone to zone interchanges in difference ranges. Both these methods left the user with a difficult choice to make on whether another iteration of either assignment or trip table building was necessary. Given the long computer times (and costs) involved, using too few iterations to achieve a satisfactory balance, particularly for the trip table cycle, was the more common outcome.

1.4 Road user costs

When traffic can travel long distances without having to stop at intersections, such as on country roads, the flow is referred to as ‘uninterrupted flow’. Where traffic has to make frequent stops at intersections, such as occurs on most types of urban roads, the flow is referred to as ‘interrupted flow’. This frequent stopping causes a significant increase in both travel time and fuel consumption with lesser increases in other components of road user costs.

Computer models for predicting road user costs in uninterrupted flow conditions as experienced on open country roads became available in Australia in the late sixties when W. D. Scott & Co. (1970) produced the MERRI (Model for Evaluating Rural Road Improvements) model for the Commonwealth Bureau of Roads. Fuel consumption and tyre wear vary with the speed of travel, and with the roughness, grade and curvature of the road. More recently, it has been realised that fuel consumption and tyre wear also depend on the surface texture of the road, but as yet no suitable formulae have been developed. Speed of travel depends on the surface type, lane width, road gradient and curvature, and on the presence of other traffic. The effect of change of speed at rural intersections on road user costs is such a small component that it can be ignored.

Early attempts at evaluating road projects in an urban environment used the uninterrupted road user costs calculated from average network speeds. Bayley and Both (1976) used the MERRI model to produce formulae for estimating road user costs as a function of average travel speed for two urban road types; freeway (or grade separated roads) and other roads. This was done by fitting a curve to data points for speeds over about 70 km /h and assuming that this applied to freeways and to data points under 40 km/h and assuming that this applied to other roads.

One of the problems with these formulae is that for speeds over about 60 km/h, the costs per km on other roads comes out less than on freeways, which is not the case. The second problem is that these formulae did not include the additional vehicle operating costs of having to stop at intersections, as they were derived from data points created by applying a model that calculated uninterrupted flow costs. These formulae (with indexation to current values) are used to calculate road user costs using the average speed on each section of road between intersections. (Johnston and Cullinan 1984). The coefficients for these formulae are still being published in the bi-annual update of prices used in economic evaluations (Austroads 2004)

The American Federal Highways Administration (FHWA) recently developed a model for evaluating urban projects called Surface Transportation Efficiency Analysis Model (STEAM) as described in De Corla-Souza, Cohen, Haling and Hunt (1999). It uses a limited number of vehicle types and the average journey speed of each trip to estimate road user costs. Currently, it is limited to estimating costs at one point in time and the user is left with finding some way of developing the time stream of costs.

Traffic forecasts are usually only available for one or two target years, somewhere in the time period covered by the analysis. Without the benefit of a fully integrated assignment and evaluation system, road user costs are estimated for one or two time periods only. The user is then faced with having to make some assumption about how to estimate the road user costs for all the other years of the analysis. As road user costs per km increase with increasing congestion, the increase in total road user costs as traffic volumes increase is non linear for a linear increase in traffic volumes, so any assumption used to estimate road user costs for the other years becomes another source of error.

Following the fuel crisis of 1977, researchers started measuring fuel consumption under interrupted flow conditions. (Watson et al 1980, Kenworthy et al 1982). This led to the development of a standard mix of ‘drive and stop cycles’ representing a typical average urban journey known as a drive cycle to be used for assessing the fuel consumption of new vehicles to regulate car manufactures to adhere to emission and fuel consumption standards. Figures of fuel consumption for the urban cycle and for a highway cycle, representing open road travel, are now regularly published in motoring magazines when road testing new vehicles. In addition they are available in a booklet published by the National Energy Research Council. Fuel consumption figures from some recent magazines are listed in Table 1.1**.** From this, it can be seen that there is a significant increase in fuel consumption in urban conditions as a result of the continuing change in speed caused by having to stop at intersections. Besides the increased travel time, tyre and brake wear is increased and there is increased wear on the transmission system.

Table 1.1 Fuel consumption figures for driving cycles based on Australian Standards 2877.

Vehicle	Urban Drive cycle Litres/100km	Highway drive cycle Litres/100km	Percentage increase for urban driving
Subaru Liberty Outback H6 Wagon	11.0	8.2	34%
Honda Civic Vi 5-door Hatch (auto)	8.5	6.6	29%
Daihatsu Sirion Gtvi 5-door (auto)	6.8	6.2	10%
Nissan 200SX Spec R 2-door coupe (auto)	11.0	7.0	57%
Toyota Camry Touring Sedan (auto)	11.0	6.8	62%

Source: RAC Road Patrol Magazine February – March 2001 and April – May 2001.

For open road travel in a rural environment, it is a relatively simple matter to collect data and relate vehicle operating costs to the speed of travel, as the need to stop is sufficiently infrequent that it can be ignored (Claffey 1971). In an urban network, this is not the case as the need to stop can add more than fifty percent to the cost of travel where controlled intersections are closely spaced. In the case of grade separated, access controlled roads (freeways), there is no intersection stopping cost.

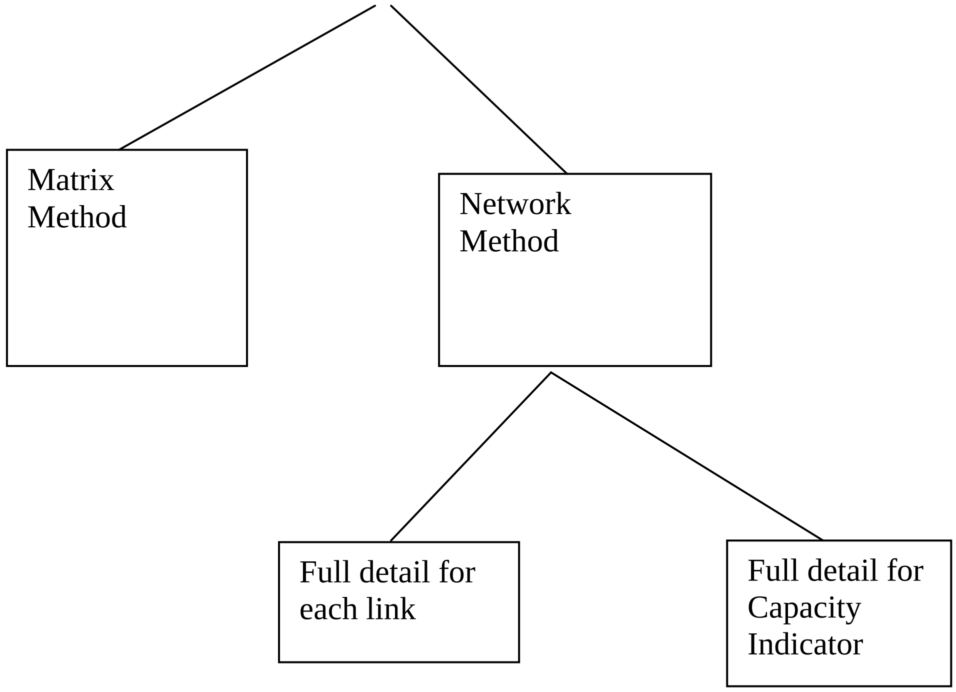
In the larger urban environments an additional problem is that for a portion of the day, travel demand exceeds the local capacity and vehicles are forced to queue and wait for the vehicles ahead to clear. This phenomenon also makes it difficult to estimate traffic volumes and travel time on a link, as the speed of travel can vary quite widely, even over short periods of time. Estimation of road user costs depends on having accurate estimates of traffic volumes and the speed at which they travel.

1.5 Choice of type of economic evaluation system

Estimation of road user costs could be done in one of two ways

- using the matrices of zone to zone travel times, travel distances, number of person trips and number of vehicle trips.
- Working at the link / node level to estimate the road user costs on each link and the costs of negotiating each intersection.

Figure 1.2 Choices of system design for economic evaluation



In the link / node method, there is a secondary choice involving at what stage the road characteristics of roughness, gradient, curvature and traffic composition are introduced. If these were introduced at the link level, the traffic forecasting networks would need to be directly linked to the road inventory to pick up the road features, and to the traffic counting system to pick up the traffic composition. It would also involve excessive computer calculation to perform each project evaluation. If average values of the road features are used for all links with the same capacity indicator, the averages are set up once in a computer file and calculation of unit vehicle operating costs as a function of the volume to capacity ratio will then only have to be done each time unit prices are changed (which on average have been every two years). Project evaluation then only requires a small amount of computer time.

The link / node method using groups of links was chosen for this system (TRAffic Modelling System = TRAMS) for the following reasons.

- The matrix approach can only use average journey speed to estimate vehicle operating costs.
- It is not possible to use a profile of the variation in the traffic flow rate throughout the day.
- It is not possible to vary the road crash rate by type of road or to separately estimate crashes at intersections from crashes that occur between intersections.

These approximations will not give a reliable estimate of the benefits of road projects.

Traffic forecasts and hence matrices are usually only available for two points in time within the analysis period. Although the growth in traffic volumes on a link is normally linear, the change in the zone to zone travel times is not linear. The change in road user costs moves even further away from linear as the costs also vary with the speed of travel. Another source of error is having to assume some form of growth rate (linear, compound or other) to connect the costs calculated from the two assignment years.

The link / node approach has the following advantages.

- Estimates of vehicle operating costs can be made as the sum of three components, cruising at constant speed, the acceleration/deceleration phase and time spent idling at intersections.
- For intermediate years, traffic volumes can be estimated by linear interpolation between the forecast years at the individual link level and then the vehicle

operating costs calculated.

- Items like fuel consumption, which are speed dependent, can be calculated using the actual speed on the individual legs of the journey instead of the average journey speed.
- The method has the ability to correctly estimate the costs of stopping at intersections.
- The variation in traffic flow rate throughout the day can be used.
- Road crash rates can be separated by road type and by intersection type.
- The proportions of each type of crash (fatality, injury and property damage only) can be varied by the speed limit on each road.
- It is reasonable to assume a linear growth of traffic on a link between forecast years but the change in road user costs will not be linear due to changing levels of congestion.
- Network changes such as adding a lane or installing traffic control signals will have a measurable effect on the altered links, which in turn affects costs, but the change at the average journey speed level will be minimal.
- The size of the study area used for economic analyses can be varied to suit the problem; eg. adding turning lanes at signals can be analysed by looking at one intersection only and assuming that traffic volumes do not change as a result of the project.

The disadvantages are:

- It is not possible to assign project benefits to people living in defined areas as distinct from trips travelling in those areas. This notional advantage of evaluation systems based on overall journey values is not correct as about one third of all journeys have neither end at home. As these are mostly the high value trips (heavy and light commercial vehicles plus people travelling on employers' business) at least half the benefits would be incorrectly ascribed to the people living in the defined area.

- It is not possible to vary the value of private time savings above and below a threshold value; eg a value is only placed on private time savings when the saving per trip is more than a threshold time period (in minutes) such as time savings of less than say three minutes have no value but savings of more than three minutes have a value.

Private travel time savings resulting from constructing a road project in an urban area are usually made up of quite small savings per person applied to a very large number of people. As such the total value is quite large. At a meeting of State Road Authority representatives to decide on a process of how to arrive at a price for private time, opinion was divided on whether pricing these savings is valid or not, as one is mixing a behavioural cost with community resource costs. Also there were those who argued in favour of only putting a price on private time savings if it exceeded a threshold value per trip, such as being more than three minutes. However this calculation can only be done when working at the matrix level. By doing the evaluation twice, once with a value on private time and once with a nil value on private time, the user can assess the effect of valuing private time on the results. There is seldom any change in the ranking order of projects using benefit cost ratios calculated with or without a value on private time. The absolute value of the benefit cost ratio changes, but for projects in an urban area, the values are high enough to justify the project without putting a value on private time.

1.6 Overview

This thesis describes the latest version of an evaluation system first designed and implemented in the late seventies. In view of the importance of having accurate traffic volumes and speeds in order to correctly estimate road user costs, chapter 2 describes the traffic assignment process in some detail. The methods in use at the time this economic evaluation system was developed are described. Potential problems with Dial's (1971) multi path assignment process are described together with a method of choosing a value for his spread parameter that minimises the problems while retaining the benefits of spreading traffic across multiple paths. Developments in assignment methods in commercial packages that have occurred since the seventies are outlined. Discussion of problems with the various methods leads to the specification of a desirable assignment process in chapter 3. A new assignment method that meets the specification is outlined, together with the results of some sensitivity testing that demonstrates the advantages of the new method.

A second reason for describing the new assignment method is to show that the derivation of daily road user costs and the assignment method are based on the same assumptions of the hourly distribution of flow rate and traffic speeds and work in harmony with each other. In chapter 4, the design requirements for an urban evaluation system are detailed and the main features of the system are outlined. Chapter 5 describes some of the initial functions used to derive the coefficients and data used in the assignment program.

The process of building up macro level vehicle operating cost data for the traffic stream from the primary unit prices and traffic composition is described in Chapter 6. Chapter 7 describes the economic analysis step. Alternate benefit-cost ratios are calculated to show the sensitivity of the results to varying the discount rate and to changing the time of construction of the project. Chapter 8 comments on possible future additions to the system. These include improvements to the traffic composition data and signalised intersection data and how the system can be made fully multi modal. It also compares the system with other systems.

CHAPTER 2

ASSIGNMENT OF TRAFFIC TO A NETWORK

2.1 Introduction

Road user costs vary with the speed of travel and the need to stop at intersections. As reasonably accurate estimates of speed and volume are essential if one is to estimate the change in road user costs caused by network changes, this chapter will discuss the assignment methods in detail to show the problems that existed with the traffic forecasting methods available in the 1970s versions of commercial traffic forecasting packages when TRAMS was developed. As much of the problem stemmed from the assignment step, this will be discussed in some detail in sections 2.2 and 2.3. The question of the accuracy of the internal arithmetic in the programs is outlined in section 2.4. In section 2.5 a method of sharing traffic between the same origin and destination across more than one path is outlined. Some problems associated with the method are described. In section 2.6, a way of limiting the problems while still retaining benefits is outlined. These are further discussed in section 2.7. The path building process is described in detail in section 2.8 to show that the solution to the internal accuracy requires more than just changing the size of the arrays used to hold the link cost values. Section 2.9 describes all the variables that influence the speed of travel on a link. The major developments that have occurred in commercial packages since the 1970s are outlined in section 2.10. The outcome of this discussion is the specification of a new assignment process that is described in the next chapter.

2.2 Selection of the assignment method

The most common method of assigning traffic to a network is to find a deterministic user equilibrium (DUE). In this method, traffic is initially loaded on to shortest routes and these loadings are modified to take account of congestion. The usual solution employs the Frank-Wolfe algorithm (Frank and Wolfe, 1956; Ortúzar and Willumsen, 1994). It is consistent with Wardrop's (1952) first 'principle' that the journey times on all routes actually used between any origin-destination pair are equal, and less than what would be experienced by a single vehicle on any of the unused routes. This method gives an approximation to flows in a congested network but the all-or-nothing character of the Wardrop 'principle' does not reflect actual driver behaviour. Practical aspects of applying this method are discussed in Sections 2.3 and 2.10.

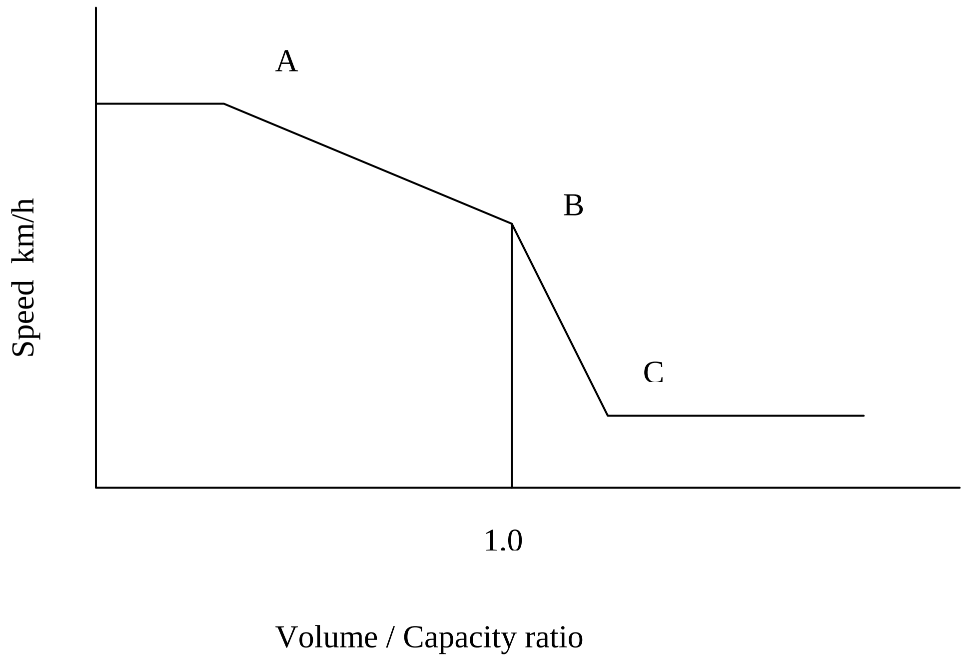
Stochastic user equilibrium (SUE) does take account of the variability of driver choices. One method is to build all-or-nothing paths after first randomising the link costs about their true mean with a nominated standard deviation (Burrell 1968). Then travel times are modified to take account of congestion. Dial (1971) developed a method of sharing traffic between the minimum path and other paths that are only marginally longer using a logit function with a spread parameter. Depending on the value of the spread parameter, Dial's method may be deterministic or somewhere between deterministic and stochastic. However, all these methods in their current use are 'equilibrium' methods requiring an equilibration procedure. The assumption that all traffic on a link in the assignment period travels at the same speed generally does not work satisfactorily in congested urban networks (Sheffi, 1985; Cantarella and Cascetta, 1998, Bell *et al*, 1997; Lam *et al*, 1999).

As the traffic flow levels increase during the peak period, some routes become more congested than others so that the change in speed in the peak period is greater on some routes than on the competing routes. Drivers choose their route based on what they expect the conditions will be at their time of travel. Using an incremental approach and the speed of the marginal vehicle about to be added to the system means that the correct estimate of speed is always used for each increment of trips being added to the network. Using Dial's multipath assignment helps share the traffic across competing routes. The following discussions deal with the problems of the methods available at the time, leading to the development of the incremental method.

2.3 Assignment methods in use at the time of system development

The speed of travel along a road depends on the volume of traffic using it. At low flows, speed changes are almost imperceptible with changes in traffic volume. As the volumes increase, the rate of change of speed increases. Each link in a road network represented in a computer is assigned a number to use as an index into a set of speed flow curves. In each speed flow curve, the relationship between speed and flow was described by three data points, (see Figure 2.1). (This has since been increased to a maximum of nine data points in recent versions of the TRIPS package – TRIPS User Manual 2002). For flows less than that represented by point A or more than that represented by point C speed was considered constant. The rapid change of slope at points A, B and C causes instability in the equilibration process.

Figure 2.1 A typical speed flow curve showing the relationship between speed and traffic volume



Early traffic forecasting packages made no provision for making separate estimates of intersection delays (TRIPS User Manual 1971). Traffic volumes were estimated by using a single capacity for each link from which to calculate the time to travel between intersections plus an average time to negotiate the intersection at the exit end of the link. The free speeds on the speed / flow curves (to the left of point A in Figure 2.1) were adjusted downwards from the uninterrupted speeds to make an allowance for the delays that occur at intersections. As the intersection delay is independent of the length of the link, this required having separate speed flow curves for inner, middle and outer areas to allow for the variation in average link lengths, the higher density of traffic signals in inner areas and the variation in speed limits between inner and outer areas. (Johnston & Cullinan 1984, p43)

To do an assignment of a trip matrix to a network, one had to assume the initial speed of travel on all links. The minimum path was built from each origin to all destinations and all the trips between each origin-destination pair were assigned to the minimum path. The travel speeds possible from the resulting traffic volumes did not agree with the assumed speeds. It was then necessary to go through a number of cycles of speed adjustment followed by assignment to try and achieve some agreement between assumed speeds and final speeds. A number of methods were used in this balancing process.

One method was to start with a best estimate of the final speeds and assign a small percentage of all the trips to the network, to factor the volume on each link up to 100% to be used to revise the speeds, then add another percentage increment of trips to the initially assigned volumes, factor these up to 100% to revise the speeds etc (incremental assignment) until all the trips were assigned. The modern day version of this is to start with the free speeds, assign an increment of trips and adjust the speed as if these were the total trips on each link. The next increment of trips is then added to the previous volume and the speeds again revised. This is repeated until all the trips are assigned. At all times, the speeds being used are the average speeds of all vehicles that one would expect if these were the volumes on the link. It is a means of sharing the traffic across routes but it does not give the same result as an equilibrium assignment, or an incremental assignment that uses the speed of the marginal vehicle. What it does do is provide some consistency between assignments to networks that contain only relatively small differences between them. The difference between this form of ‘incremental’ assignment as used in TRIPS and the incremental assignment used in this system

(TRAMS) is explained in Appendix 3.4

Another method was to assign 100% of trips in each iteration and to successively average the volumes before revising the speeds (volume averaging). A third method was to assign 100% of trips in each iteration and to adjust the speed to halfway between the assumed speed and the desired speed. In all cases, the final volumes depended on the speeds assumed for the first assignment. Also, all the vehicles on a link were assumed to travel at the same speed. As pointed out by Sheffi (1985), each method produced traffic volumes that were different from every other method.

In the TRIPS traffic forecasting package, the assignment procedure and the process of speed adjustment were carried out in separate programs. Prior to 1971, the only choice for path building was the all-or-nothing process where all the traffic between each origin – destination pair was put on the same path. The assignment program could report turning volumes but these were not saved in the output computer file. As the design engineers wanted turning volumes, the choice of assignment methods was restricted to those where the total trips were assigned in the final run in order to provide the matching turning volumes.

The author’s early experience of using cycles of 100% assignment followed by partial speed adjustment, then 100% assignment etc was that it was almost impossible to achieve a satisfactory closure; with the level of closure after ten iterations being little better than after three iterations. The final result depended on the speeds assumed for the first iteration. When testing the effect of adding a project to the network, one had to be careful to use the same initial speeds in the base case and the project case. Otherwise the differences caused by the project could easily be swamped by the differences resulting from the different starting speeds.

The 1971 release of the TRIPS package contained Dial’s assignment program AVSTOCH which gave the user a choice of path building methods. In addition to the all-or-nothing path building routine, Dial had developed a procedure for sharing traffic across paths of almost equal length called multi-path assignment (Dial 1971). While the multi-path option made an improvement on the level of closure, the author still found it impossible to achieve a satisfactory closure between link speeds and traffic volumes. A detailed investigation was carried out into the reasons for the failure to close. The results of this investigation are described in the next sections.

2.4 Effect of using integers for link cost

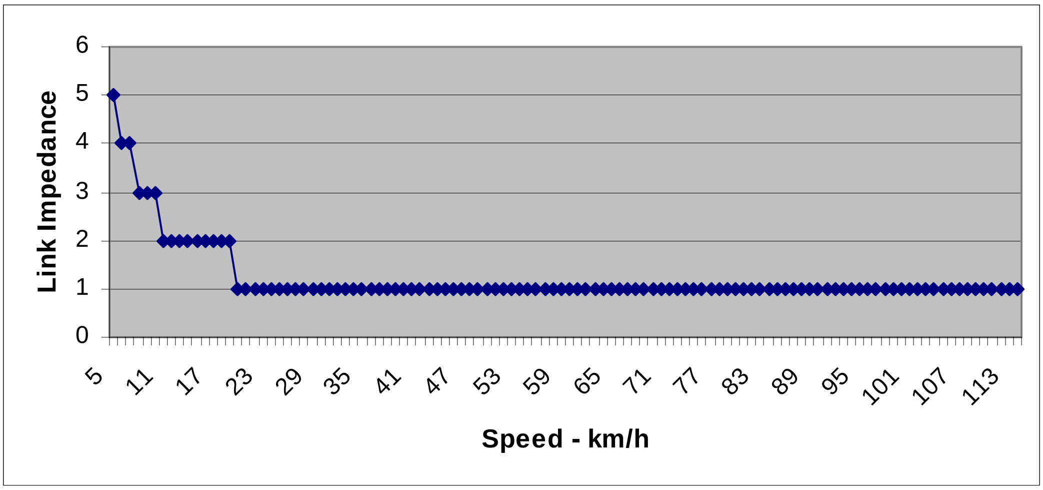
Link impedance (commonly referred to as link cost) is the behavioural length of a link used for building the choice of route through a network. Different modellers calculate it in different ways. Some use the out of pocket cost of fuel, some use travel time while others use a combination of travel time and distance. Perth is using a combination of time and distance based on the results published in Benesh (1972).

The first step in the investigation was to examine the number of links by cost value. It was found that 58% of all the links in the network had values in the range zero to three. When Dial first wrote his multi-path program AVSTOCH in 1970, access to a computer with more than 256 kbytes of memory was rare so programs had to fit in about 190 kbytes. In order to make his program able to process as large a network as possible, link cost values were saved as integers in one byte, thus limiting the maximum value to 255.

A second reason for choosing integers was that at certain stages in the path building process, the cost value is used as an index into arrays and only integers could be used as array indexes. AVSTOCH used the values in the network file header of maximum link distance and maximum link time, combined with the coefficients of time and distance to work out a suitable scale factor to limit the maximum link cost on the longest link to 255.

In consequence, on the shorter road links, changing speed over quite a wide range may have no effect, or a quite small change may affect the paths being built. This can be seen by reference to Figure 2.2 where speeds in the range of 20 to 114 km/h all produce the same link cost value for path building. Similarly, speeds from 11 to 20 all produce two units of cost. This made it extremely difficult to balance traffic volumes against link speeds.

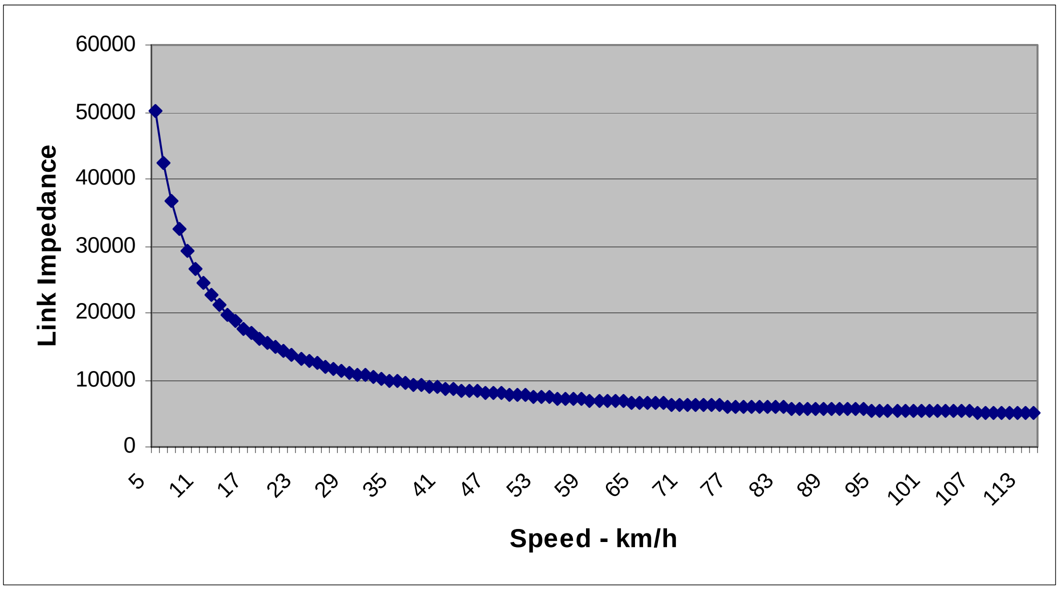
Figure 2.2 Link impedance versus speed for a link of 300 m when using Dial's AVSTOCH (1970 version)



Note: Impedance is a combination of time and distance, so has no meaningful units.

By the mid seventies computers with larger memories were available so the solution was to increase the size of the array variable used to store link costs to a half word with a practical limit of 32,000. This has since been increased to a whole word with a practical limit of 16,000,000. The maximum path length is also stored in one word integers; hence the limit on any one link must be considerably less than the maximum value that can be stored in one word. The effect on the relationship between link costs and speed can be seen in Figure 2.3. The ability to discriminate between routes has been vastly improved. These changes to the maximum possible link cost required some changes to the path building algorithm. These changes are described in section 2.8. The numbers used to construct these plots are listed in Table A2.1 in Appendix 2.1 The abrupt change in the slope of the speed flow curves at the specified data points was also contributing to the problem of trying to achieve closure.

Figure 2.3 Link impedance versus speed for a link of 300 m when using the modified program.



Note: Impedance is a combination of time and distance, so has no meaningful units.

2.5 Side effects of Dial's algorithm

The ability of traffic to spread across competing routes in Dial's multi path process is controlled by a parameter θ giving the probability of using route k from r to s as

$$P_{rs}^{k} = \frac{e^{-\theta C_{rs}^{k}}}{\sum_{k \in K_{rs}} e^{-\theta C_{rs}^{k}}} \qquad (1)$$

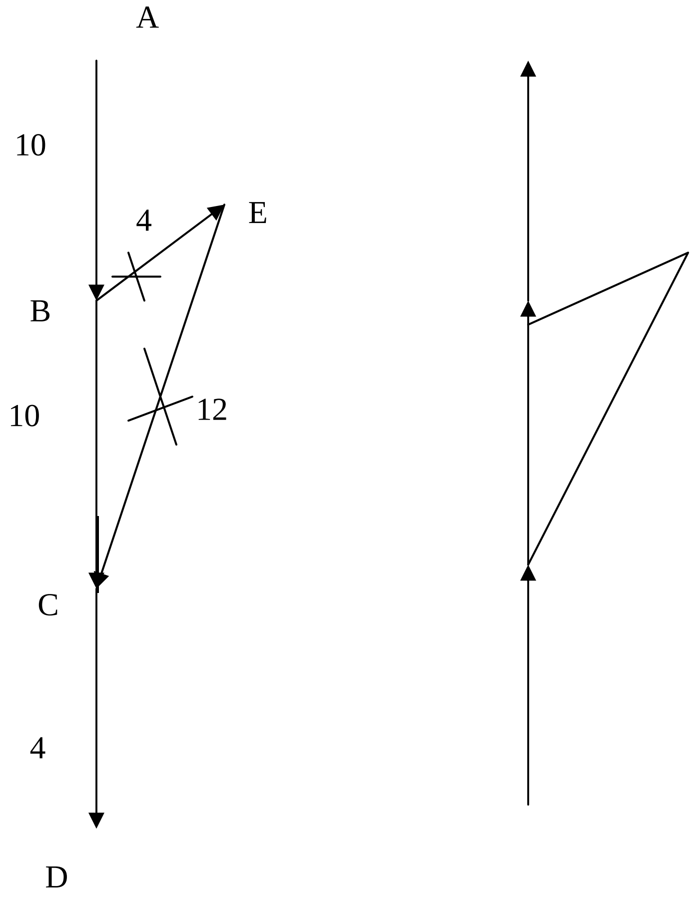
where C_{rs}^{k} is the impedance taken on route k from r to s.

The author had been using Dial's recommended value of 0.1 for θ. During examination of link volumes to determine the reason for the failure to close, it was observed that in portions of the network, the traffic volumes in each direction were vastly different. Given that there were no one way roads anywhere in the vicinity, this was unacceptable as the traffic counts were very similar in each direction. This was happening in areas where the network links formed a triangle rather than a rectangle. What appeared to be happening was that a path was being accepted when travelling in one direction but not in the opposite direction.

Dial first defined an acceptable path as one where the exit from a link was both further from the origin and closer to the destination than the entry to the link. This definition would have required paths to be built from the origin to all destinations, and from all destinations to the origin to determine if the path using that link met the requirement to be acceptable. This would have required a very large amount of computer time on production networks. So he defined a relaxed version that the exit from a link had to be further from the origin than the entry to the link. The effect of this can be seen by reference to Figure 2.4.

Traffic travelling from A to D can use either path ABCD (the minimum path) or path ABECD as E is further from the origin than B and C is further from the origin than both B and E. However traffic travelling from D to A is prevented from using path DCEBA because node E is further from the origin than the node B, thus preventing the use of link EB because this would involve back tracking. This will then result in flows on links BC, BE and ED being unequal in each direction. In this case, travel in both directions along the route BEC should be regarded as highly unlikely. With the first definition of an acceptable path, route BEC would have been rejected because node E is further from the destination than node B. This choice of unacceptable paths can be minimized by using a relatively high value of θ to restrict alternate paths to those that are only

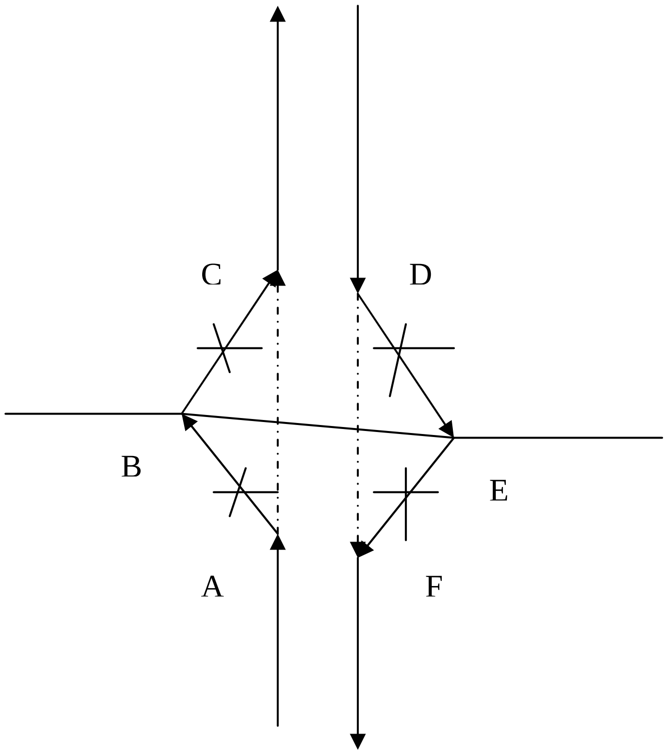
Figure 2.4 Unacceptable paths in Dial's algorithm



just longer than the minimum path. In this way, the benefits of using the multi path assignment to improve the sharing of traffic across competing routes of almost equal length is obtained while keeping the unwanted side effects under control. A method of choosing a suitable value of θ is described in section 2.6.

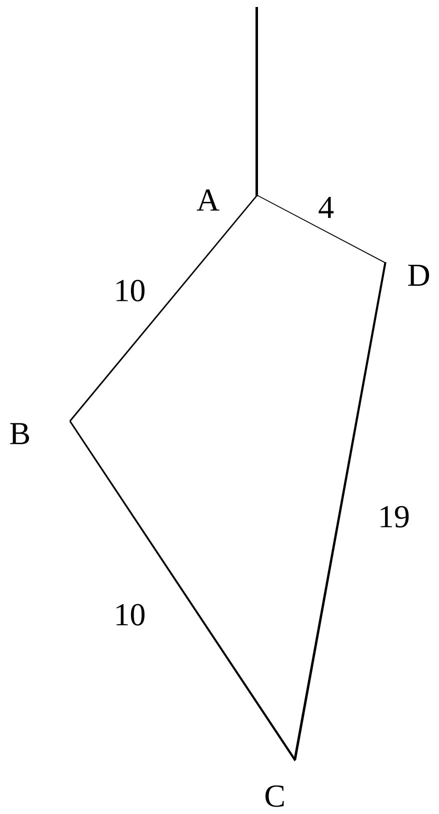
Other areas in a network that need to be watched for unacceptable routes are the freeway ramps in diamond interchanges. If the freeway volumes are very heavy, and freeway speeds become slow, the computer may start assigning traffic down an off ramp, through the intersection and then via the on ramp back to the freeway again as an alternative to staying on the freeway i.e. route ABC instead of route AC in Figure 2.5. The lower the value of θ, the more likely this will happen.

Figure 2.5 Illustrating improper use of freeway ramps



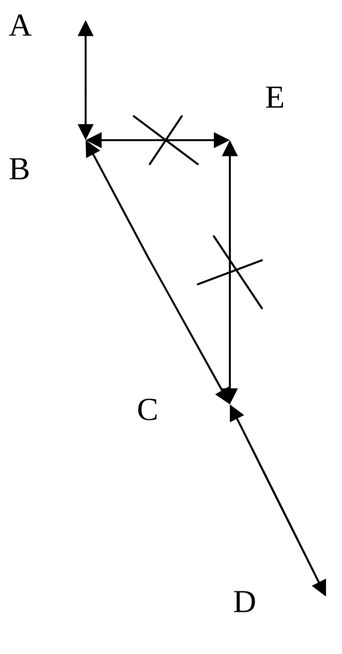
There is a second undesirable feature of Dial's multi path algorithm. This is described with reference to Figure 2.6**.** In Figure 2.6, both paths are possible when travelling from A to C and from C to A as in each case the terminating node (C or A) is further from the origin than the intermediate nodes B and D. If one were now to split the link AD in half by adding a node (E) in the middle, both paths from A to C are still possible. However, for traffic travelling from C to A, the path CDEA is now no longer possible because the path from CDE is longer than the path CBA, thus implying that to use the link EA would be back tracking when in fact it isn't. The only answer to this problem is to be aware of the possibility, and, if in doubt, if a link requires dividing in order to add a project, also make sure that the division of the link is made in the base network. In this way the addition of a node number changing the paths used between base and project is avoided.

Figure 2.6 Effect on acceptable paths of adding a node



Road designers expect to be able to get turning traffic volumes so as to be able to design intersections. In Figure 2.7, through traffic from A to D and from D to A would be expected to use the link BC and not to use the links BE and EC. As the value of θ is reduced, the amount of traffic using the route BEC (and CEB) grows as a percentage of the traffic on link BC. Values of θ that are too low would cause unreasonably high estimates of the traffic making the turns BEC and CEB.

Figure 2.7 Undesirable paths



2.6 Choosing a value for θ

The advantage of using a multi path type assignment is that it spreads the volume of traffic across paths that are either equal in length or almost equal in length. This assists greatly in achieving a suitable spread of traffic where there are parallel routes. Too low a value of θ allows paths that leave and rejoin the most direct route in ways that are unrealistic, while too high a value reduces the effectiveness of the path spreading.

In the Perth highway network, there are a number of cases of right-angled triangles where the most direct route for through traffic is along the hypotenuse. If θ is set too low, traffic will be assigned around the other two sides of the triangle which is not only longer but also includes travelling through an additional intersection. Six such triangles were selected for examination. Experiments were conducted with different values of θ. In each case the amount of traffic diverted via the right angle was expressed as a percentage of the total flow on the hypotenuse. The lowest value of θ that gave no more than about 4% of turning traffic in any of the cases was selected for everyday use.

θ depends on the level of link cost scaling used, the relationship being

$$\theta * \text{scale factor} = \text{constant}. \qquad (2)$$

For ease of comparison with Dial's work, all values for θ quoted in this work have been converted to the same scale factor used by Dial. The value chosen from the testing described above corresponds to a value of 1.97 in Dial's (1971) paper. These experiments were done many years ago and the results have been lost so it is not possible to tabulate them here. The road network has moved on, with four of the triangles no longer in existence. However, there are now over 20 diamond interchanges in the network where this problem can occur.

2.7 Further comments on the value of θ

For a number of reasons, vehicle drivers do not always select the shortest path between their origin and destination. This has the effect of slightly increasing the total vehicle-km and vehicle-hours of travel compared to the amount of travel if all drivers selected the shortest path. In a recent paper, Dial (2001) has used this to derive a method of calibrating the value of θ to give the best match between the assignment and traffic counts. In the stylised networks that he uses in his paper, the opportunities for the undesirable side effects are very limited and do not seem to occur, so that he can get an increase in the total travel with lower values of θ without any apparent problems. In real networks, there are nearly always some cases where triangles exist, where the undesirable side effects will show up if θ is set too low. In the sensitivity testing of θ it was noted that when this started to occur corresponded to an increase in the total travel on the network of about 0.05%.

2.8 Path building process

The path building process in the 1971 release of AVSTOCH uses Moore's algorithm in which a sequence table is used to keep track of which node should be visited next in the path building chain. In AVSTOCH, the length of this array was made equal to the largest possible link cost plus 1 i.e. 256. Indexation into the array is the modulus (cumulative path length to the node, 256) +1. Using a chain structure, more than one node maybe entered into the same position in the sequence table. The original computer code retrieved the last entered node where there was more than one entry. In order to increase the link scaling and still achieve the shortest path, it was necessary to modify the algorithm. To explain why this is necessary, the path building algorithm is described in detail.

During the course of building the paths from a single origin zone to all destination zones, the number of entries in the table rises from zero to a maximum before declining to zero on completion. In the Perth networks, (containing 937 zones and over 12,000 one-way links) the maximum number of entries was a little over 200. When the maximum possible link cost was increased, one option was simply to increase the length of the array to suit. This not only would have used up valuable computer memory but also added measurably to the computer run time as the search for the next available node through an almost empty array took place.

A simplistic option would be to leave the length of the table at 256, to use the modulus (cumulative path length to the node, 32768), divide this result by 128 and add one. In this approach, nodes in the same entry position now may have different accumulated cost values from the origin node. Once a node has been removed from the sequence table, it is not possible to enter it again if a shorter path to it is found. Simply removing the last entered node may result in instances of the shortest path being missed by up to the length of the divisor – one, in this case 127 units. More than one instance may occur on the longer paths. It is thus necessary to modify the path building algorithm to search along the chain of all nodes in the one index position to find the one with the lowest accumulated path cost. By adopting a compromise of extending the sequence table to 512 and searching along the chain to find the next node, the increase in computer time is much less than if the table was simply extended. This change also meant it was possible later to increase the maximum possible link cost to a very high value, when computer memories increased sufficiently to use whole word integers to store the link costs.

Although these changes produced an improvement in the ability to match link speeds to link volumes, in many cases there were still large differences between modelled flows and traffic counts. Possible reasons for this are examined in the next section.

2.9 Traffic volumes and traffic speeds

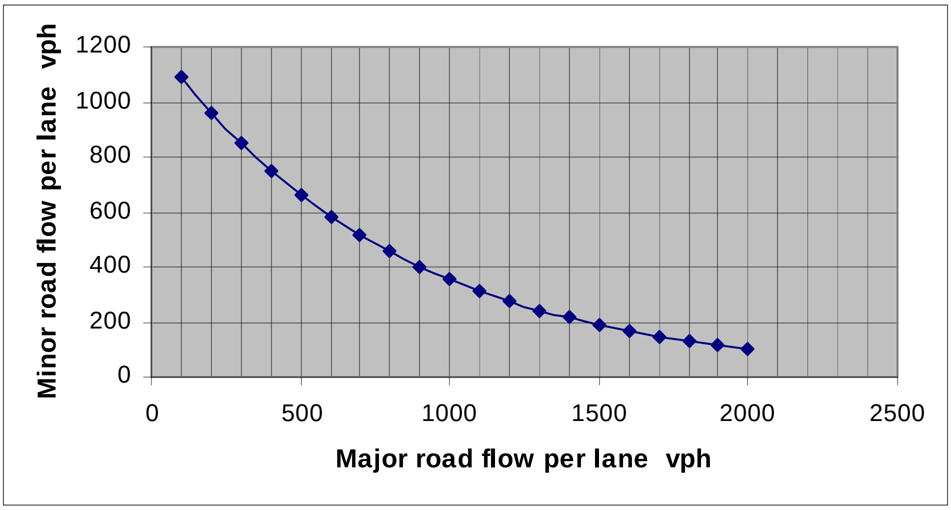
The speed of travel along a link between intersections depends on the speed limit, and the ratio of traffic volume to the capacity of the link. On most roads, the speed is very close to the speed limit, even in peak periods. The time required to negotiate an intersection depends on the type of control, the speed limit, the rate of deceleration and acceleration, the traffic volume and the capacity of that approach to the intersection, which in turn depends on the volume of traffic on the intersecting road. This time to negotiate the intersection is independent of the length of the link.

The capacity of a road or intersection is commonly expressed in terms of vehicles per hour (or vehicles per day). However, vehicle size also affects capacity. For detailed capacity calculations, large vehicles are expressed in terms of the equivalent number of cars that could pass a point in the same time. For rigid trucks, the usual factor is two and it is higher for combination vehicles. Capacity is then expressed in passenger car units per hour (pcus/hr). The volume of traffic on a link then has to be converted to pcus by taking into account the traffic composition. When capacity is expressed in terms of vehicles per hour, most times what is meant is pcus per hour. For an average urban road with about 10% heavy vehicles, the difference is about 10%.

The capacity per lane of roads at intersections varies quite widely, depending on the type of intersection control and the volumes of traffic on all the other approaches. Uninterrupted (or mid-block) per lane link capacities vary from around 1800 vehicles per hour for controlled access roads down to about 900 vehicles per hour in the median lane of undivided roads where the traffic is delayed by vehicles turning right into property entrances. The per lane capacity of roads approaching traffic signals can vary from as little as 150 vehicles per hour up to 1300-1400 vehicles per hour depending on the number of phases in the traffic signals and on how much of the available green time is allocated to that approach. These flows are being measured at a set of signals in Perth where the signals operate with two phases and the ratio of the demand flows on the two roads is one to ten. The share of the green times is running at about one to nine.

For intersections controlled by signs, the per lane capacity on the priority road can be as low as 800 vehicles per hour for the median lane with no protected turning lane and up to 1800 vehicles per hour for through lanes. For the minor approach, the capacity can vary between about 1100 vehicles per hour for very low flows on the priority road, down to as low as 50 vehicles per hour for very high per lane flows on the priority road (see Figure 2.8)**.**

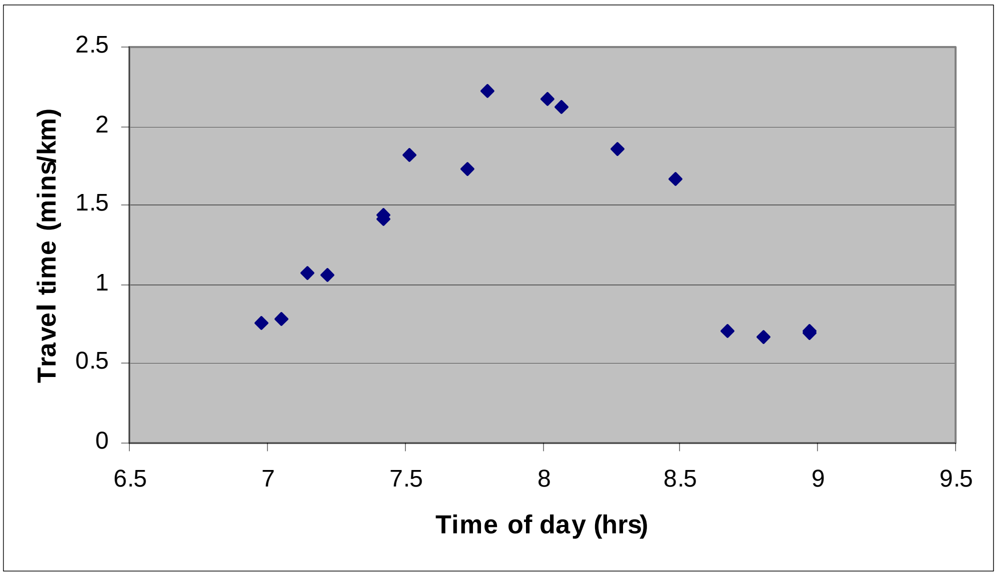
Figure 2.8 Capacity per hour per lane for a road controlled by a stop sign relative to the per lane traffic volume on the priority road



Source: Flows calculated from gap theory using the work of Tanner.

Trying to estimate travel times including the intersection delays using only a single capacity to represent both the intersection and the road between intersections is clearly not likely to produce good estimates of traffic volumes and speeds on all links in a network. Another issue is that many computation methods being used assume that all the vehicles on a link travel at the same speed. There is a marked change in average journey times on most links between peak periods and the remainder of the day. There can also be a marked change in journey times over quite small time periods in the peak period. This is illustrated in Figure 2.9, which shows the travel time per km on a four km section of the Mitchell Freeway in the morning peak period. These changes in time affect different routes differently, so that for drivers, the choice of route may depend on the time of day. Where all vehicles are assigned the same speed, this shift in proportions choosing each route dependent on the time of day cannot be accommodated, thus increasing the difficulty of trying to get the modelled volumes to agree with traffic counts. This analysis leads to the specification of the desirable features of an assignment method. This is discussed in the next chapter.

Figure 2.9 Travel Time in morning peak period on a 4 km length of freeway.



2.10 Further developments in commercial packages

Nguyen (1974) described a method of equilibrium assignment which is a modification of the volume averaging method. Instead of using equal portions of each iteration to determine the link flows to use to adjust the speeds, the method is attempting to find those volumes on each link which will provide equal path costs between the points of choice of two or more minimum paths. To do this, the process calculates an objective function which is measuring the difference between the product of link volume and link cost of paths actually used and the product of volumes and link cost assuming all vehicles were on the minimum path. The aim is to find that combination of the most recent assignment and the combined total from the previous iteration that minimizes the objective function. This proportion (lambda) of the latest iteration is combined with (1 - lambda) times the previous total to arrive at the link volumes.

Ruiter (1974) reported on the planned addition of this assignment option to the UMTA Transportation Planning System which is supported by the American Federal Government. This assignment option (equilibrium assignment) has since become available in other traffic forecasting packages such as EMME/2 (Florian 1984) and TRIPS. While providing some improvement in the degree of closure between link speeds and link volumes, the method is based on the assumption that all vehicles on a link travel at the same average journey speed.

The rate of traffic flow on a network varies considerably throughout the day (U.S. Highway Capacity Manual 1965 Figure 3.15 p38. This figure does not appear in the 2000 U. S. Highway Capacity Manual). The highest flows in an urban area generally occur at the time when most people are either going to or from work or education. Selecting a suitable flow rate to use for road design is a balancing act between having enough capacity so as not to excessively delay peak period travel and having too much capacity which is under utilised during the rest of the day and hence uneconomic. The Austroads Urban Road Design Manual suggests the 30th highest hour for rural roads and the average weekday peak period flow for urban roads. This corresponds to somewhere between the two hundredth and five hundredth highest hourly flow rate. When combined with the inability of traffic forecasting packages to address the problem of speed differences between peak and off-peak, most modellers choose to model a peak hour or peak period. For accurate economic analysis, road user benefits need to be estimated for the whole day as is done in the system described in chapters 4 to 7.

With most modelling being for a peak hour, it should not come as any surprise therefore that when the commercial software suppliers decided to make provision for intersection delays to be estimated separately and then added to the time to travel between the intersections, they chose the detailed methods used for intersection design to do the intersection delay calculation. This calculation requires the user to supply for each node at which calculations are to be performed, a table of entry, through and exit nodes for each possible movement for each approach to the intersection. In addition, for sites that have traffic control signals, the number of phases and the number of lanes on each approach that are allowed to move for each turn /through movement are also required. This assumes that the user has chosen the option of vehicle-actuated control. If not, then the user must also supply the total cycle time and the amount of green time allocated to each phase.

To supply all this intersection information requires considerable effort. If the information is not correct for the traffic demand, it can bias the results away from the correct demand, which is needed to correctly design the intersection. So at some stage during the matching of travel demand to network speeds, the intersection results need to be examined to see if they need changing. The delay calculation methods can only be used for time periods up to one hour and cannot be used for all-day modelling. Given the increased preparation work involved in having the system do separate intersection delay calculations, the advice in the TRIPS manual is to only do intersection delay modelling at the intersections of interest and to let the rest of the links in the network be analysed on a link only basis. In the TRIPS package, the user is advised not to use the equilibrium option when doing intersection delay modelling, as the calculation of the objective function does not include the intersection delays other than fixed time turning penalties.

With peak hour modelling, one soon runs up against the problem that on some links there are more trips in the trip table to be loaded on the network than there is available capacity to accept them. In real life, vehicles are being delayed beyond the first presentation of the green signal and have to queue and wait for the next cycle. The build up of queues can prevent vehicles exiting from an upstream intersection as there is nowhere for them to go. This results in a reduced capacity for the upstream intersection. The queues clear a short time later when the demand reduces.

The requirement to model this phenomenon led to the development of dynamic assignment modelling. The time period to be modelled is divided into slices with a varying percentage rate of the trips being assigned in each increment. A profile of the demand is input for each zone. As the vehicles move through the network, the profile may become modified if an intersection is reached where the demand exceeds the capacity. This modified profile is then continued through the system until the trips reach their destination. In theory, the total length of the modelling period should be at least twice as long as the longest possible journey time plus a lead in period to initially put some traffic onto the network. In this way there is time for trips that may be delayed at an intersection to eventually reach their destination within the modelled period. In the example given in the TRIPS User Manual, it appears that the modelling period is still retained at one hour plus lead in time.

Where the number of trips exceeds the link / intersection capacity, they are removed from the trip table and saved in a separate matrix, the theory being that the people making these trips would either not make the journey or make it by some other mode. The reality that they may choose to make the journey slightly earlier or later and hence extend the peak period is denied to the modeller. When additional capacity is added to the network, these journeys in reality would then retime their trip to once again be in the peak hour. However, the modeller is now using a trip table with these trips removed and hence underestimates the peak hour flow. The consequences of this were quite serious in the forecasts made for the completion of the M25 ring road in London in the early 1990s (Wood 1994). The completed 6 lane motorway was full in the peak period from the day that it was opened to traffic despite the modelling predicting that it should have had spare capacity equivalent to one lane each way. There were other contributing reasons as well and these are listed in the Wood (1994) report on the subsequent inquiry into what went wrong.

2.11 Summary

The capacity of a link excluding intersections is close to the saturation flow rate of 1800 vehicles per hour per lane. While the saturation flow rate per lane per hour of green time at signals is also about 1800 vehicles per hour, the actual average capacity per lane depends on the number of signal phases, on the number of additional turning lanes at the stop line, and on the flow on each of the other road approaches. Even at the same intersection, the throughput capacity can vary from a low of 150 vehicles per hour to a high of 1350 vehicles per hour. For intersections controlled by signs where one road has to give way to the other, the flow per hour per lane can vary from a high of 1800 vehicles per hour to a low of a little as 50 vehicles per hour.

The delays caused by intersections is independent of the length of the road between intersections, so the effect of the intersection delay can cause the average journey speed on a link to vary from as little as one or two km/h to a high of almost equalling the speed limit on the road. Trying to estimate travel times including the intersection delays using only a single capacity to represent both the intersection and the road between intersections is never going to produce an accurate estimate of vehicle speeds and volumes over the whole network. Even though traffic forecasting packages now have the ability for the user to model intersection delays separately to the time to travel between intersections, the amount of work involved in inputting the intersection information and checking the results is so onerous that users seldom use intersection delay modelling at more than a very small percentage of the intersections in a network. This means that the majority of flows and speeds are still being estimated from link only type calculations. In addition, intersection delay modelling is not available for all-day modelling and the systems are not capable of allowing for the variation in speed throughout the day.

Realistic economic assessments of road projects in an urban environment require an integrated system that can provide accurate estimates of traffic volumes and travel speeds. To do this the system must have the following features.

- It must allow for the fact that over a whole day not all vehicles on any one link travel at the same speed.

- Intersection capacity must depend on the relative flow rates on each approach.
- Network coding should not be too onerous, preferably being limited to the physical characteristics, such as type of road, number of lanes mid block, speed limit, number of turning lanes, and type of intersection control.
- Intersection analysis should be carried out at all intersections.
- It should be possible to vary traffic composition across a network.

An assignment system that meets the above criteria is the subject of the next chapter.

Appendix 2.1 Effect of link scaling on impedance for a link 300 m long

The effect of the limited range of link impedance values in Dial's AVSTOCH for a link length of 300 m is that speeds in the range of 20 to 114 km/h all produce the same link value for path building. Similarly speeds from 11 to 20 all produce two units of impedance. The effect of the current link scaling is also tabulated in Table 3.1 under 'New' where it can be seen that a speed change from 84.00 to 84.03 km/h will produce a change of 1 unit in the values used for path building. Even at a speed of 110 km/h it only requires a speed change of 0.05 km/h to produce a change in the link impedance.

Table A2.1 Effect of internal scaling on impedance for a link length of 0.3 km

Comparison between Dial's AVSTOCH and the MRWA Program

Speed	Time	Impedance	
km/h	min	Old	New
1	18.0000	24	238800
2	9.0000	12	120900
3	6.0000	8	81600
4	4.5000	6	61950
5	3.6000	5	50160
6	3.0000	4	42300
7	2.5714	4	36686
8	2.2500	3	32475
9	2.0000	3	29200
10	1.8000	3	26580
11	1.6364	2	24436
12	1.5000	2	22650
13	1.3846	2	21138
14	1.2857	2	19843
15	1.2000	2	18720
16	1.1250	2	17738
17	1.0588	2	16871
18	1.0000	2	16100
19	0.9474	2	15411
20	0.9000	1	14790
21	0.8571	1	14229
22	0.8182	1	13718
23	0.7826	1	13252
24	0.7500	1	12825
25	0.7200	1	12432
....	....	....	....
....	....	....	....
73	0.2466	1	6230
74	0.2432	1	6186
75	0.2400	1	6144
76	0.2368	1	6103
77	0.2338	1	6062
78	0.2308	1	6023
79	0.2278	1	5985
80	0.2250	1	5948
81	0.2222	1	5911
82	0.2195	1	5876
83	0.2169	1	5841
84	0.2143	1	5807
85	0.2118	1	5774

Note: Impedance is a combination of time and distance, so has no meaningful units.

CHAPTER 3

A NEW METHOD OF ASSIGNMENT

3.1 Introduction

Chapter 2 has discussed the difficulties of achieving realistic traffic volumes and vehicle speeds on all links in a network when using any of the commercial traffic forecasting packages such as TRIPS, EMME/2 or TRANPLAN. This chapter specifies the desirable features of an assignment process that will produce suitable estimates of traffic volumes and speeds. The implementation of a system that satisfies these criteria is outlined in sections 3.3 and 3.4. The results of detailed sensitivity testing on various aspects to show that the system responds in the correct direction to quite small network changes are discussed in sections 3.5 to 3.7. Section 3.8 comments on the ability to do peak period assignment. Other applications of the assignment process that need to be able to produce the same volumes on the links of interest as the assignment are described in section 3.9.

3.2 Desirable features of an assignment process

It is desirable that the assignment process has the following features.

- It should make separate estimates of the time to traverse a link and the time to negotiate the intersection at the end of the link for all links.
- The effort to code intersection information should not be too onerous.
- Information for intersections under signal control should not require details of which lanes move on which phases.
- It should not require individual phase times.

- It should allow for the variability in the traffic flow rate throughout the day.
- It should respond to all the physical changes that may occur in a road network e.g. number of lanes, number of turning lanes, type of intersection control, number of signal phases, speed limit.
- It should consistently close to the point where the speeds corresponding to the resulting volumes match the speeds assumed for the assignment.
- It is preferable that the intersection information does not require a separate file for storage so as not to have to keep track of which intersection file goes with which network file.

Incremental loading using the speed of the marginal vehicle about to be added to the network is an appropriate way of dealing with the variability in the traffic flow rate throughout the day and the effect this has on vehicle speeds. To improve the accuracy of estimates of the time to traverse a link requires all intersections in the network to be analysed to estimate intersection delay times. For this to work effectively, the information needed has to be minimal and not depend on the results being known first, i.e. for signalised intersections it should be limited to the number of approach legs, and the number of lanes at the stop line on each approach.

By making the assumption that the phase times will be set correctly for the resulting traffic, the calculation of delay time can be expressed in terms of the volume to capacity ratio instead of in terms of flows from one link to another, the amount of green time and what movements take place on each phase. By a suitable choice of formula for calculating this delay, it becomes possible to change to an incremental method of assigning traffic to the network using the speed (or delays) of the marginal vehicle being added to the system. This removes the need for a trial and error process as the speeds and delays being used for the path building are correct for the vehicles about to be added to the system.

As the assignment proceeds and the volumes on each link increase, the speeds get slower. The beginning of the assignment represents the conditions in the early hours of the morning. The middle period represents the flow conditions in the middle of the day. Near the end of the assignment represents peak period conditions. The process works best for all day assignments but can also be used for peak periods, provided that the period being modelled exceeds the time when queues exist, say two or three hour periods. Total all day travel times can be calculated by integrating the curve to obtain the area under the marginal speed flow curve, (see Figure 3.1). Dividing this value by the volume capacity ratio on the link will give the average vehicle speed. Similarly, estimates of peak period time can be obtained by integrating the curve between say 90% and 100% of the volume on the link, (see Figure 3.2). The conversion of formulae from average all day time of travel to the time of the marginal vehicle and vice versa is detailed in Appendix 3.1.

Figure 3.1 Total delay to all vehicles all-day

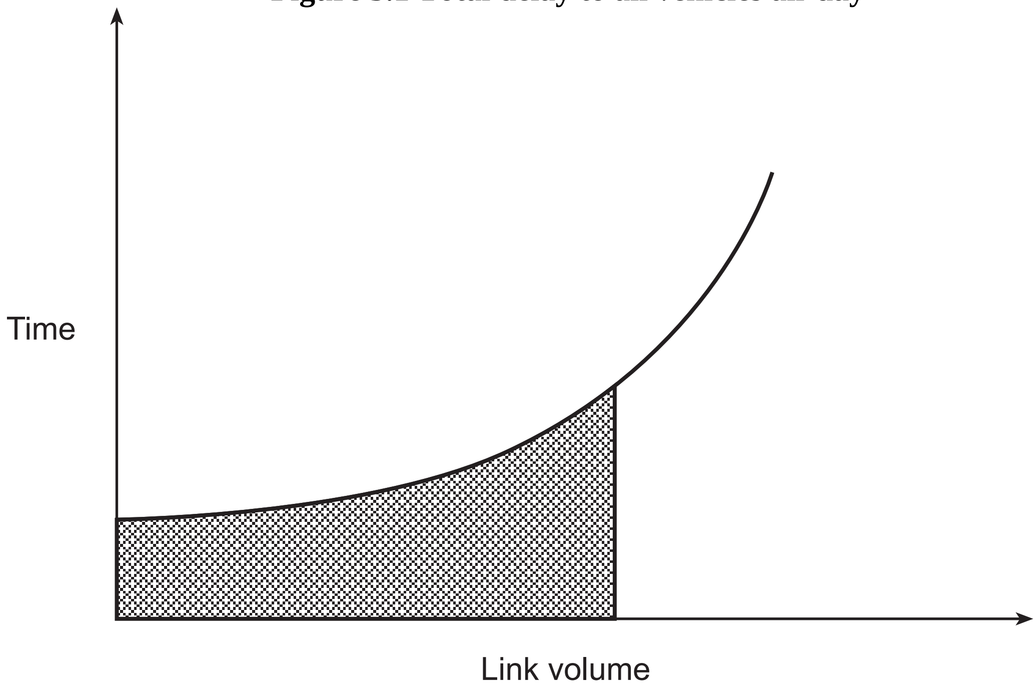
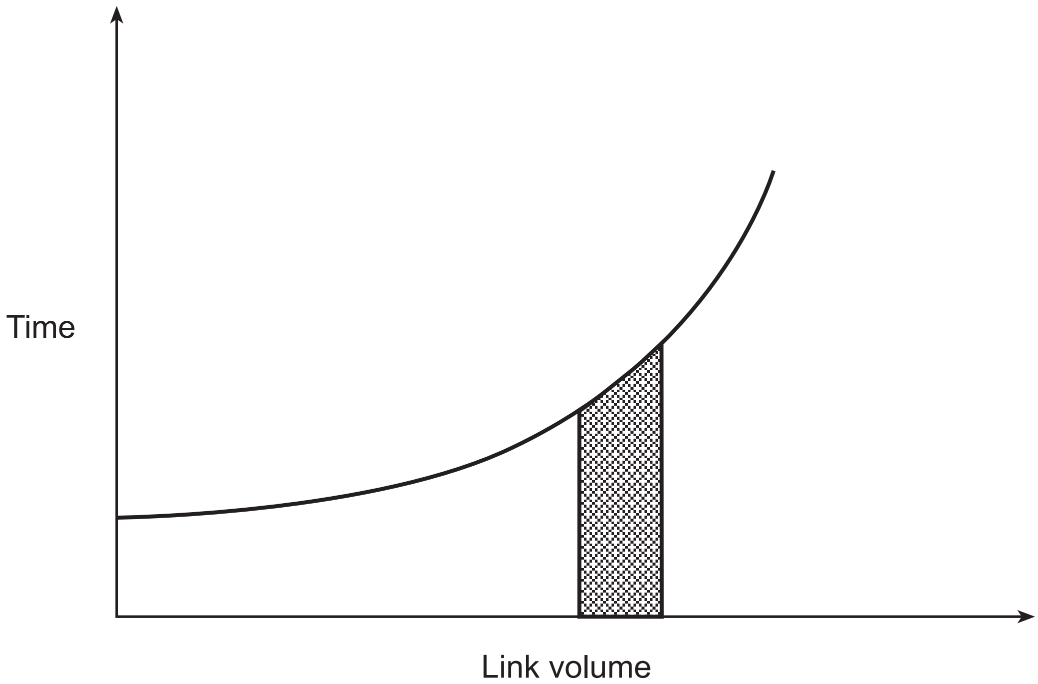


Figure 3.2 Total delay to all vehicles in the peak period



3.3 Incremental assignment using the speed of the marginal vehicle

Common methods of assigning traffic to a network assume that for any link, all vehicles on that link during the period of analysis will travel at the same speed. This is clearly not the case for all day travel and it can be argued that it also does not even apply in peak periods (see Figure 2.9). Incremental assignment is a method that has some of the travel at speeds attained during very low flows (night time), some at medium flow levels (middle of the day) and others at the speeds prevailing in the peaks, all modelled as a continuum. This is achieved with a process that sets the link times for no volumes, and then assigns a small percentage of the daily traffic to the network. The times are then recalculated to be the speed of the next increment of vehicles to be added to the network and the next increment of vehicles is then added. This process is repeated until all trips have been assigned.

The incremental method described here is not to be confused with the incremental assignment available in the TRIPS package. The differences are described in Appendix 3.4.

The average time of travel for all traffic is now required for the trip distribution phase of the four step forecasting process, also an estimate of the time of travel in the peak for the mode choice step. These can be calculated by integrating the marginal speed flow curve between no flow and the predicted flow for all-day, and between say ninety percent and one hundred percent of assigned flow for the peak period. This puts a condition on the type of formulae used to describe the speed flow relationship, that they need to be of a form that can easily be integrated to give the all day average time of all vehicles and the average peak time. The conversion of the formulae for the all day average time of travel to the speed of the marginal vehicle is described in Appendix 3.1. A flow chart of the program appears in Figure 3.3.

Figure 3.3. Flow Chart of Assignment Program

Read and edit run parameters
Build network in memory
Calculate selected data while copying trip table to a random access file
Calculate order in which zones will be selected for assignment
Set all link speeds to speeds at no load

Calculate volume to be assigned before next revision of speeds
Select zone to be assigned
  Calculate portion of trips leaving zone to be assigned
  Build all paths
  Assign selected trips
  If volume increment not reached, select next zone and repeat
End of zone loop
Revise speeds to that of the marginal vehicle that will be added next
Repeat from beginning of box

Integrate speed formulae to give average speed of all vehicles
Revise all link speeds
Write network information to output file
Report requested link and turn volumes
Report summary statistics

To achieve the effect of a reasonably continuous process, ie a small increment of trips on each link between capacity restraints, the total number of trips to be assigned needs to be broken into at least fifty steps. If this were done by taking the same percentage from all zones before revising the travel times, the amount of computer time would be excessive. The number of trips leaving a zone is not uniform over a whole network and can in fact vary quite widely. An alternate approach of loading all the trips from a small percentage of zones before revising travel times suffers from two problems. All the trips from the early zones would choose travel routes on an uncongested network, while the trips from the last few zones would choose routes from a congested network. A second problem would arise where the total number of trips from one zone exceeded the number to be loaded in that increment. Consider districts to be groupings of, say, ten neighbouring zones. What is needed is some way of ensuring that some trips from each district are assigned to the network between each revision of travel speeds. How this is achieved is described in the following paragraphs.

A list of zones with trips is constructed in such a way that on average the interval between zone numbers is one hundred if the high zone number is three digits, is ten if the high zone number is two digits and is sequential if the high zone number is one digit. The trip table is then processed to find and save the number of trips that leave each zone (the intrazonals are excluded from the table, as they are not assigned to the network). The largest number of trips leaving any zone is also obtained; call this value MAXVOL. One of the parameters that the user specifies is the largest number of trips leaving a zone that will be assigned in any one increment. (LVOL). The total number of trips that will be assigned before revising travel times is calculated from parameters and information in the trip matrix.

The process of selecting from which zones to load trips is more easily explained in steps. The process is:

- set a variable to zero (SUM)

- select the first zone on the sequence list
- add the trips leaving that zone to SUM
- does SUM equal or exceed MAXVOL
- no, select next zone from list and repeat
- yes, subtract MAXVOL from SUM
- calculate the proportion LVOL/trips leaving zone
- calculate the proportionate number of trips going to each destination and save in a temporary array
- subtract from the saved trip table and save the revised row, also revise the row total
- build the paths from the origin to all destinations and assign the trips in the temporary array
- add the volume assigned to the progress total
- if the total has reached the number to be assigned before revising travel times or the sequence list has been processed once, revise the travel times
- to allow for the increasing rate of change of travel time with increasing flows, the number of trips to be loaded between time revisions is revised downwards using supplied parameters
- processing through the sequence list is then resumed.

When the end of the list is reached, LVOL is subtracted from MAXVOL, LVOL is revised downwards and the list is wrapped around to continue processing. Using this process on networks with over nine hundred zones and with travel times being revised sixty to seventy times requires on average about five paths per zone to be built (depending on the number of trips leaving the zone, some zones may have as many as ten to twenty paths built, while others will be as low as one).

3.4 Expressing intersection delays in terms of volume capacity ratio

3.4.1 Signalised intersections

The method used for the detailed design of signalised intersections uses the volume of traffic on each individual movement from approach leg to exit leg, the number of lanes available for each movement, the phases on which the movement takes place, the total number of phases, and the cycle time. The user tries a number of combinations of these variables to find the one that is best able to carry the demand volume of traffic. If the wrong combination is tried, the capacity of the intersection will be less than for the best combination. During a traffic assignment, if the capacity available on any movement is less than the demand, the resulting traffic volumes will be less than the true demand and the excess will find an alternative route, i.e. the intersection design has to be supplied in order to find out the demand to do the design. Also the calculation methods do not lend themselves to doing an incremental assignment or for doing anything other than a peak one-hour assignment.

By making the assumption that the signal settings in a future year will be optimised to suit the actual traffic demand volumes, whatever they may be, it is possible to move away from needing all the detail about phasings and the proportion of green times for each phase. All that is needed is the total number of lanes at the stop lines on all approaches and the number of phases. The intersection capacity is then the total number of lanes at the stop lines multiplied by the capacity per lane per hour of green divided by the number of phases. The demand is the total volume entering the intersection.

The relationship between delays and hourly volume capacity ratio is built up by using the detailed calculation methods used for intersection design in Akcelik (1980a and 1980b) with the traffic flows being determined from progressively increasing volume capacity ratios. This is done with a specially written computer program that requires the capacity per hour of green time, the number of phases, the lost time per phase, the total cycle time and the proportion of the green time allocated to each phase. It also uses the speed limit on the approaching roads with nominated acceleration and deceleration rates to estimate the extra time to traverse the distance during deceleration and acceleration compared to travelling the same distance at the speed limit. For those vehicles that stop, this excess time has to be added to the time calculated for the delays at the intersection, which is only the stationary time.

The program also calculates the probability of stopping at the intersection (Akcelik 1980a). This is used to calculate the excess time averaged over all vehicles. It is also used in the economic analysis with the cost of stopping at the intersection. The derived formula for the intersection delay is for the average delay to all vehicles, assuming that the flow rates are hourly and that the time period involved is relatively short, say 30 mins. How this delay versus hourly volume capacity curve is converted into an all day average delay per vehicle versus daily volume capacity ratio and hence into the delay of the marginal vehicle is described in Chapter 5. In use, the form of the equation used is

$$Time = A * VC^{B} + C \quad (3)$$

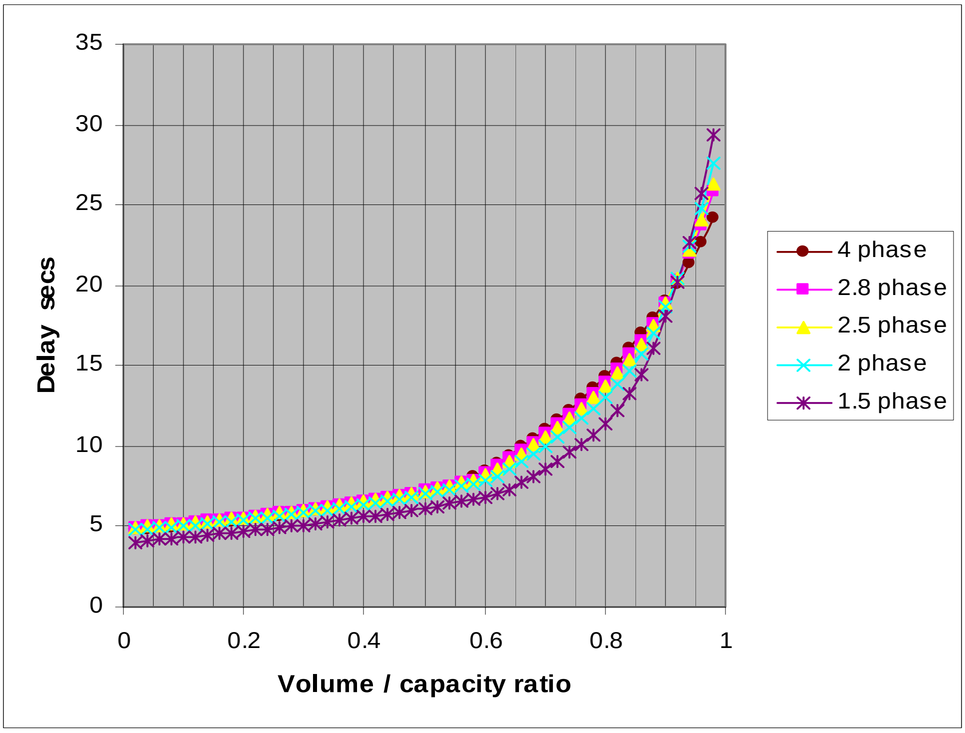
where A, B and C are coefficients

VC = volume to capacity ratio

The derivation of the formula for the excess time to change speed is reported in Appendix 3.2.

By running the program with different phases, it was found that the average delay per vehicle was almost directly proportional to the number of phases. This is shown in Figure 3.4 where the delay per phase is plotted against the volume to capacity ratio. Thus, by introducing the number of phases into the delay formula, it is possible to use only one set of coefficients for each speed limit.

Figure 3.4 Compare delay per phase across the number of phases



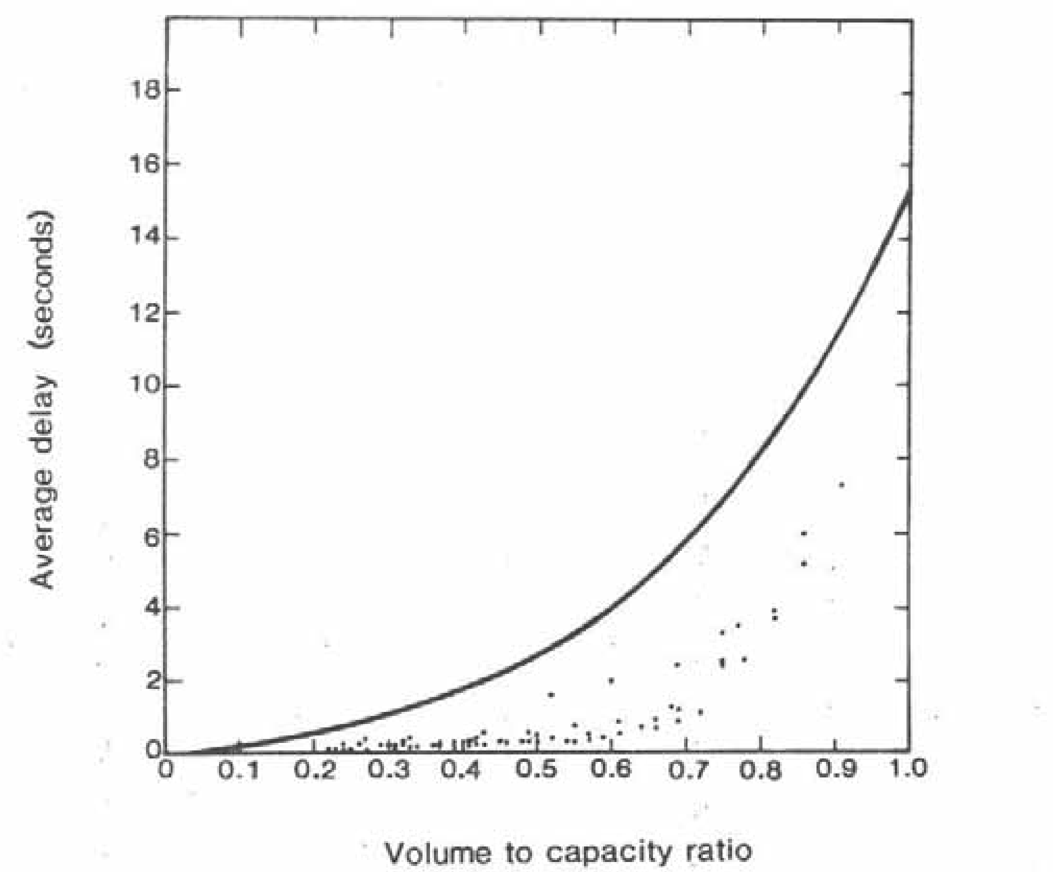
The formula for delays to vehicles at traffic signals is for the average delay to all vehicles assuming an equal share of the green time between phases. The delay on an individual approach will vary from this if the approach share of the available green time is not strictly in proportion to the number of phases. The program for estimating the delays at traffic signals as a function of the volume to capacity ratio also calculates delays and the probability of stopping for an assumption of the available green time being only 40% and 160% of its pro rata share (i.e. for a three phase set of signals, the initial delays are for an approach with 33% of the available green time. The alternate delays are for 0.4 x 33% and 1.6 x 33% = 13% and 53.3% of the available green time). These percentages give a reasonable spread of the likely lower and upper bounds of the phase share of available green times. By comparing these results against the mean delay to all vehicles, a correction procedure was developed to estimate the delays on each approach that assumes that the green times will be shared between approaches so as to give equal approach volume to capacity ratios. This procedure is described in Appendix 3.3.

During the early stages of an assignment, the relativity between volumes on each approach can change rapidly. If the full adjustment to the approach delay for unequal green times is made, the resulting delays favour the approach with the largest share of the green time as the preferred route in the following increments of traffic. This can distort the final link volumes. As the greatest need for the correction is in the later stages of the assignment when the delays are larger, the procedure incorporates a progressive increase in the proportion of the adjustment correction to be applied. This is also detailed in Appendix 3.3.

3.4.2 Freeway merges

Delays at the merge point for freeway on ramps are calculated by assuming a one phase set of signals with a zero constant. The accuracy of this assumption was later verified by researchers from Murdoch University and reported in Lyons, Rainford, Kenworthy and Newman (1984) (see Figure 3.5).

Figure 3.5 Measured and computed delays at a freeway merge



Source: Lyons T. J, Rainford H., Kenworthy J. R., and Newman P. W. G. (1984).

3.4.3 Major approaches to a major / minor intersection

On the two approaches to a sign controlled intersection that have priority, some delay to other vehicles is imposed by vehicles wishing to turn right having to wait for a gap in the opposing traffic flow. This delay to through vehicles can be reduced by the presence of an exclusive turning lane. To make the assignment sensitive to the addition of a turning lane, delays to the volumes on the two major approaches are calculated as for a one phase set of signals with no constant term, similar to a freeway merge. The accuracy of this assumption has not been checked but at least it means that the addition of an exclusive turning lane will produce an increase in the traffic flows as traffic changes route. The alternative is that the presence or absence of a turning lane does not affect either the assignment or the economic analysis, which is definitely not correct.

3.4.4 Approaches controlled by a stop or give-way sign

For approaches to stop or give way signs, the delays to the minor road traffic are calculated using gap theory based on the work of Tanner (1962), dealing with the simple case of one lane crossing one lane. No suitable formulae were found to deal with the case of one (or two) lanes crossing multiple lanes. The approach adopted is to use the split of traffic between directions and the usual spread of traffic across multiple lanes to arrive at suitable divisors to convert the total two-way flow on the major road into an equivalent one lane flow. The same approach is used for the minor road flows. The formula for one lane crossing one lane is then used to estimate the average delay per vehicle to the minor road traffic. Again the excess time for acceleration and deceleration is added to the dwell delay.

3.4.5 Rotaries

Large rotaries (or roundabouts) require eight nodes to represent them. Each approach to the rotary is treated the same as for a freeway merge ie as a one phase set of traffic signals with no delay at zero flow. The exit from the rotary has to be at a different node to the next entry point for the program to recognise that the entering traffic only has to give way to that traffic continuing around the rotary and not the traffic that is leaving the rotary. Traffic on the rotary is assumed to have no intersection delay. By choosing a suitable capacity indicator, a user can have the delays on the approach to the rotary calculated as a stop and give way.

3.5 Changes that have been made to AVSTOCH

As a result of the investigation into the failure of successive iterations of assignment and capacity restraint (AVSTOCH / AVCAP) to close adequately, a number of changes were made to the assignment program AVSTOCH as follows:

- The storage of link costs was changed to a half word.
- The program was changed to incremental assignment with capacity restraint.

Sensitivity testing was then carried out to

a) test the effect of changing the link cost scaling

b) test the difference between using all-or-nothing path selection and multi-path path selection.

This was done by selecting a congested link in the inner / middle suburbs and progressively increasing the link length by 10 m at a time through five increments (giving six networks). The link in question was a subway under the railway where quite long queues exist every morning with traffic attempting to cross the railway. The three nearest alternative crossings of the railway were also partly congested. The location of the link is shown in Figure 3.5. The tests were:

- all-or-nothing with the original link cost scaling
- multi-path with the original link cost scaling
- multi-path with the new link cost scaling.

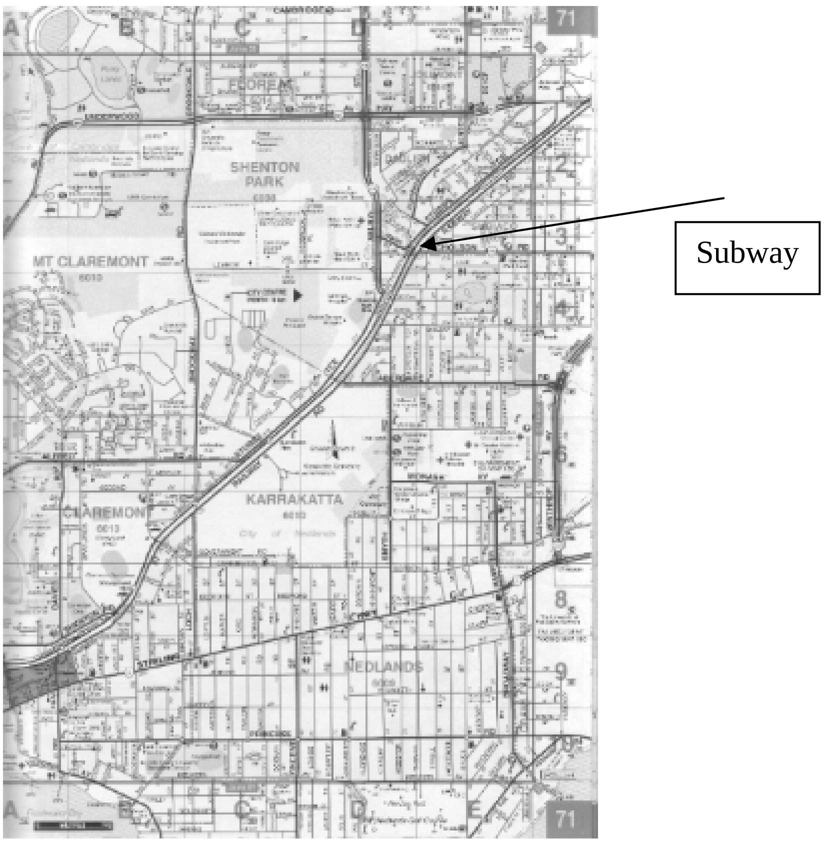
The tests were done a long time ago and have been lost but they did show

a) a considerable improvement with the change in link cost scaling

b) multi-path assignment produced a more even change in volume with changes in link length than did all-or-nothing assignment

c) while there was an improvement in responding to small changes in link length, there was still a long way to go to improve the ability of the model to match traffic counts across the whole network.

Figure 3.5 Location of subway chosen for sensitivity testing



Since then a number of improvements have been implemented over a period of years. These include:

- changing the storage of link costs from a half word to a whole word
- introduction of separate intersection delay calculations at all intersections
- changing the speed flow curve from a three point curve using straight line interpolation between points and representing the average speed of all vehicles to a continuous power curve representing the speed of the marginal vehicle about to be added to the system
- changing the increment of trips to be added from a single zone from being all the shortest trips leaving the zone to a percentage of trips going to all destinations
- reducing the size of the increment of trips from a zone to be loaded in one increment from 5,000 trips to a reducing value that starts at 960 trips at the beginning of the assignment and reduces to about half by the end of the assignment to allow for the increasing rate of change of travel time with increasing volume.

The program now in use as part of TRAMS uses whole integers to store link costs. The limiting value on scaling is now the requirement that the longest possible path in the network cannot exceed the largest integer value less the divisor used to rescale the index into the sequence table less one.

The effect of the current link scaling is also tabulated in Table A2.1 under ‘New’ where it can be seen (by interpolation) that a speed change from 84.00 to 84.03 km/h will produce a change of one unit in the values used for path building. Even at a speed of

110 km/h it only requires a speed change of 0.05 km/h to produce a change in the link cost. The original sensitivity tests have been repeated using the latest version of the program. These are reported in section 3.6.

3.6 Sensitivity tests on effect of small changes in link length

To test the programs ability to respond to small changes in network conditions, five additional networks were created by increasing a link across the railway by 10 metre increments. This is the same link as used for the earlier sensitivity testing but for a much later period (1996 instead of 1976). The chosen link has queues in the morning and evening peaks and the three alternative crossings of the railway are also partially congested. The all-or-nothing assignment method and the multi-path assignment method were each run using the original link cost scaling. The multi-path assignment method was also run using the current link cost scaling. The results are listed in Table 3.1 and illustrated in Figures 3.6 and 3.7. The incremental change in the link volume is now much more uniform as the length of the link increases.

Table 3.1 Directional link volumes by link length

	Direction A - B			Direction B - A		
	Link volume			Link volume		
Link Length (km)	AON (1)	MP (2)	MP (3)	AON (1)	MP (2)	MP (3)
0.08	16758	17131	16742	14976	15323	14945
0.09	16611	16963	16513	14858	15145	14852
0.10	16611	16699	16285	14712	15130	14760
0.11	16633	16501	16060	14568	15015	14669
0.12	16454	16432	15837	14439	14913	14579
0.13	16448	16364	15620	14355	14765	14488
	Direction A - B			Direction B - A		
Link Length (km)	Volume Decrements			Volume Decrements		
	AON	MP	MP	AON	MP	MP
0.09	147	168	229	118	178	93
0.10	0	264	228	146	15	92
0.11	-22	198	225	144	115	91
0.12	179	69	223	129	102	90
0.13	6	68	217	84	148	91
Average	62.0	153.4	224.4	124.2	111.6	91.4
Std Dvn	93.47	84.93	4.77	25.22	61.58	1.14

(1) All-or-nothing assignment with link scaling to 0.1 units
(2) Multi-path assignment with link scaling to 0.1 units
(3) Multi-path assignment with link scaling to 0.0003 units

Figure 3.6 Direction A – B Volume change versus link length (values from Table 3.1)

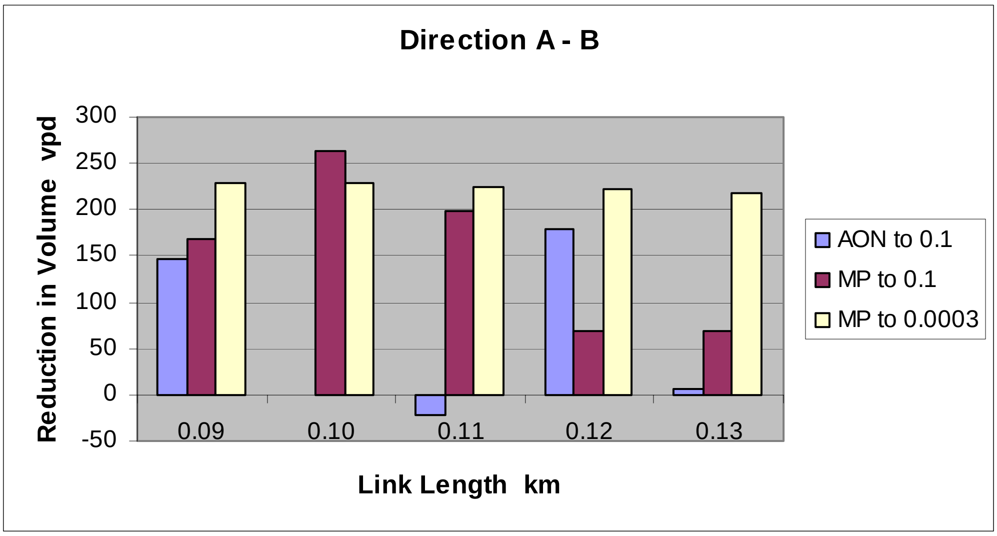
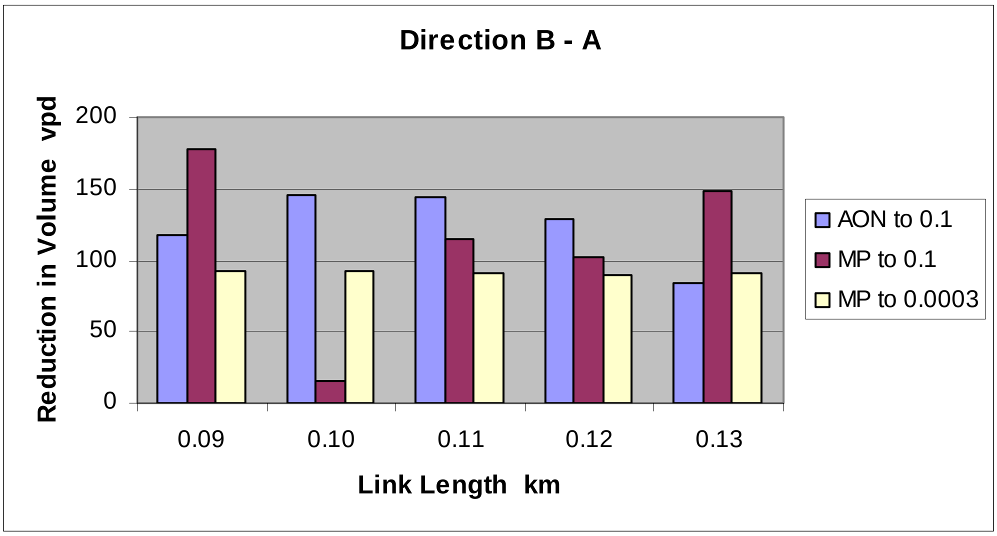


Figure 3.7 Direction B – A Volume change versus link length (values from Table 3.1)



3.7 Sensitivity tests on θ using the latest program

Only two of the original six triangles used to test the effect of changing values of θ still exist in current networks. These were used for the repeat tests. In one case, traffic signals exist at intersections B and E with the link lengths as shown in Figure 3.8. In the other case, signals exist at intersections C and E, with the link lengths as shown in Figure 3.9.

Figure 3.8 Vincent St, Fitzgerald St, Bulwer St Triangle

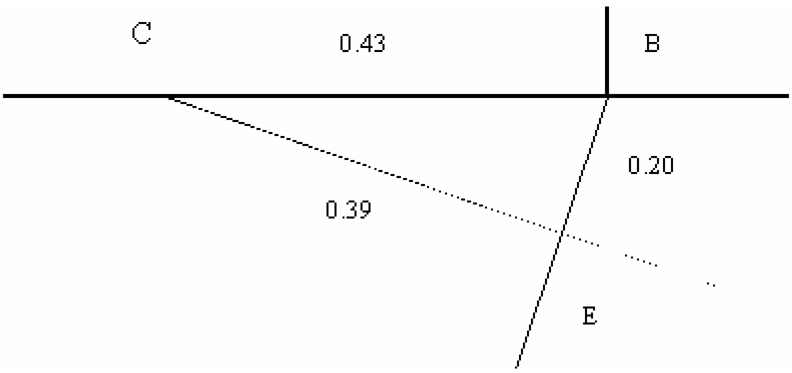
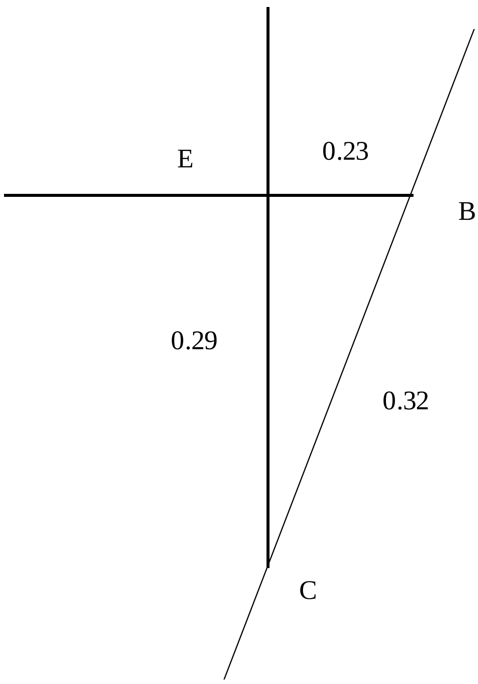


Figure 3.9 Thomas St, Nicholson Road, Rokeby Road Triangle



In both cases, the through route is the link BC. The volume making the turn BEC and CEB was recorded and expressed as a percentage of the total two-way traffic on link BC. θ was varied from the value in current use both up and down by powers of 10. θ is scaled internally in the program depending on the scale factor used to scale up the link costs. For the program to use only equal paths, the effective value of θ after the scaling is applied must be over 10.0. In the following tables, all values of θ are quoted in values corresponding to the scaling used in Dial's paper. The smallest value used was limited by the increasing size of an array used as a lookup table instead of continuously evaluating the exponential expression used for calculating the path weights. The turning and link values are listed in Table 3.2. The change in total travel is listed in Table 3.3.

In example 1, traffic travelling from B to C stayed on the through route and actually increased with reducing values of θ below 0.00197 while traffic in the reverse direction diverted in increasing numbers to the route CEB. In example 2, the main diversion occurred for traffic travelling from B to C with some diversion occurring in the reverse direction. The escalating growth in system travel starts to occur at the same values of θ where diversion of traffic to unacceptable routes begins to occur. Up to this point the percent increase in travel over that occurring when only minimum paths are permitted is 0.002% for vehicle-kilometres and 0.048% for vehicle-hours.

Table 3.2 Effect of θ on volumes using illogical route

	Bulwer St - Vincent St - Fitzgerald St						Nicholson Rd - Rokeby Rd - Thomas St					
	Vincent St Volumes						Thomas St Volumes					
θ	Turning Volume Bul-Fitz	Direction AB	Direction BA	Total	Percent Turning	% of base	Turning Volume Nich - Rok	Direction AB	Direction BA	Total	Percent Turning	% of base
11	0	11038	15706	26744	0.000	0.00	1	19887	18323	38210	0.003	0.00
1.97	0	11034	15696	26730	0.000	-0.06	1	19889	18322	38211	0.003	-0.01
0.197	1	11044	15702	26746	0.004	-0.03	1	19895	18324	38219	0.003	0.01
0.0197	3	11022	15739	26761	0.011	0.21	1	19907	18331	38238	0.003	0.04
0.00197	3	11229	15352	26581	0.011	-2.25	1	19840	18351	38191	0.003	0.15
0.000197	134	11591	13021	24612	0.544	-17.10	2981	19494	15768	35262	8.454	-13.94
0.0000197	1392	11579	12477	24056	5.786	-20.56	8808	19142	10391	29533	29.824	-43.29
0.000004	1794	11633	12152	23785	7.543	-22.63	9403	19378	9648	29026	32.395	-47.34

Bul-Fitz = Intersection of Bulwer St / Fitzgerald St
Nich – Rok = Intersection of Nicholson Road and Rokeby Road

Table 3.3 Effect of θ on total system vehicle-km and vehicle-hours of travel

θ	Veh-km	Veh-hr	Percent increase Veh-km	Percent increase Veh-hr
11	36290545.53	834563.36	0.000	0.000
1.97	36290566.44	834565.09	0.000	0.000
0.197	36290317.87	834563.01	-0.001	0.000
0.0197	36290989.68	834591.06	0.001	0.003
0.00197	36291368.99	834966.06	0.002	0.048
0.000197	36532517.58	847631.40	0.667	1.566
0.0000197	37739586.06	898176.25	3.993	7.622
0.000004	38100997.73	914342.39	4.989	9.559

Figure 3.10 Effect of varying the value of θ on proportion taking illogical route

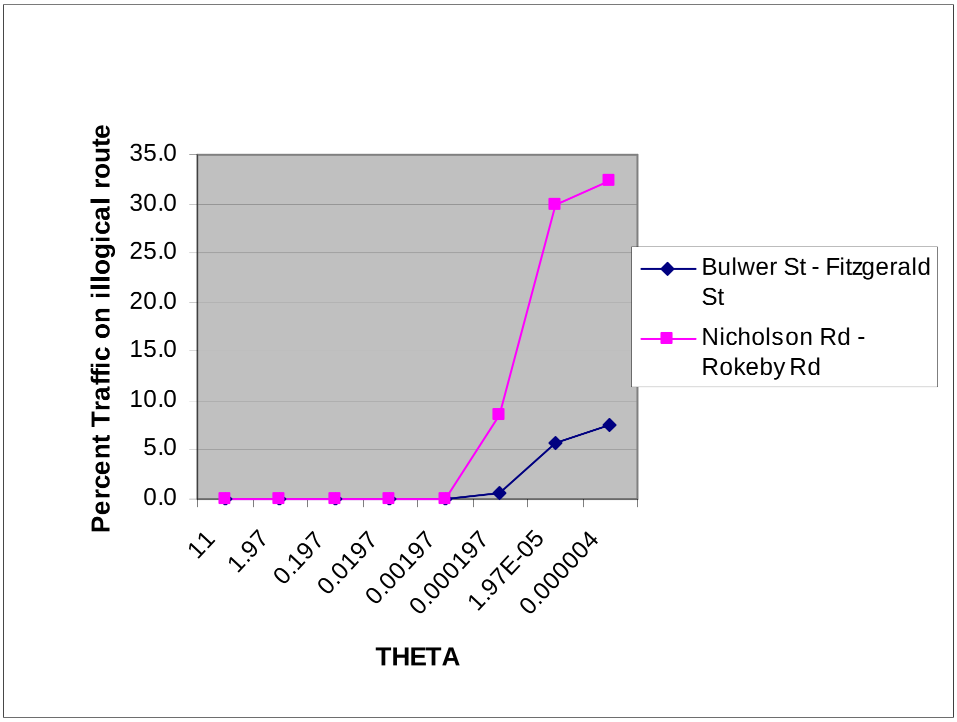


Figure 3.11 Percent increase in travel versus θ

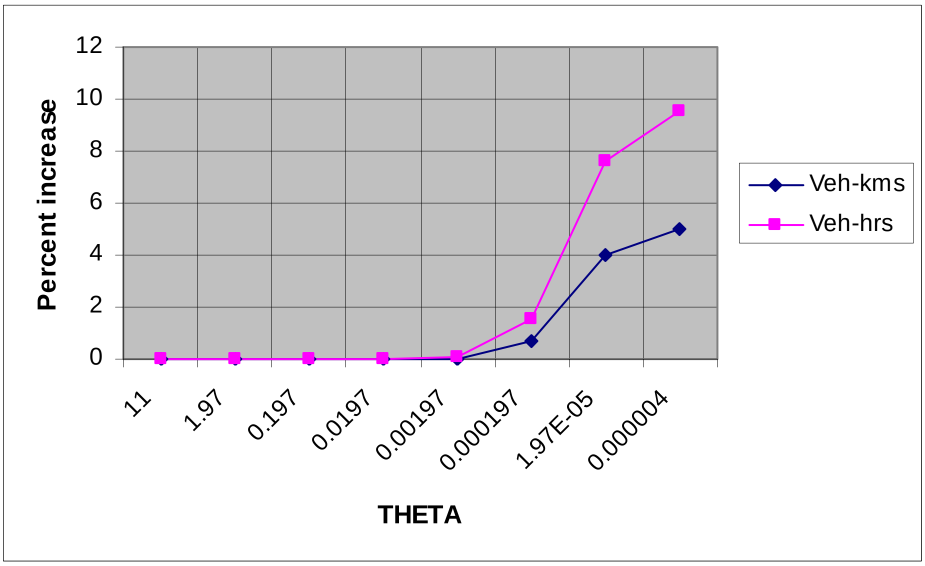
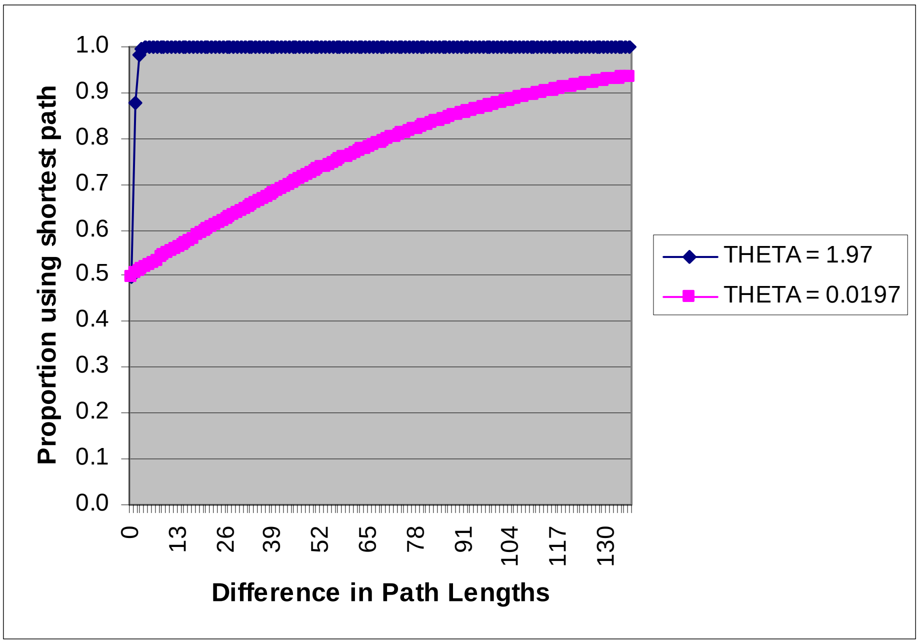


Figure 3.12 shows how lowering the value of θ allows the sharing of traffic to paths with a bigger difference in length from the minimum path. This shows the desirability of using as low a value of θ as possible consistent with not having traffic use unacceptable paths and acceptable paths being rejected when a new node number is added.

Figure 3.12 Spreading effect of lower values of θ



Note: Impedance used for path building is a combination of time and distance, so has no meaningful units.

3.8 Peak period assignment

The concept of incremental assignment using the speed of the marginal vehicle can be easily explained when used for all day assignment as it allows for the change in vehicle speeds as the flow rate changes. Other assignment methods assume that all vehicles on a link in the period of the assignment travel at the same speed. While this obviously does not apply to all day travel, it also does not apply in peak periods spanning one or more hours. So if a suitable speed flow curve is used, there is no reason not to use the incremental assignment method for extended peak periods. The peak period needs to be longer than the period over which queues exist, so that the process used to construct all-day and peak period speed flow curves can clear all vehicle queues. The process of constructing speed flow curves is described in chapter 5.

The assignment program MRDLOAD contains default speed flow curves to do either all day or two hour peak period assignments. Both sets of speed flow curves are contained in the program with the choice of which type of curves to use being controlled by an option. A parameter is also provided to convert all day capacities to peak period capacities. If a peak period trip table is available, it takes about an additional 4 secs when setting up a run to change from an all day assignment to a two-hour peak period assignment. This includes changing the speed flow curves and all network capacities as this is done by the program. It has been used on a number of occasions to extract a sub area am peak two hour trip table for the Perth City Council SATURN model.

3.9 Other features of the assignment program

In association with assigning traffic to a network, modellers also have a need to be able to identify only those trips that use a nominated link in the network and to be able to trace these trips along each link back to their origin and forward to their destination. This is referred to as selected link analysis. There is also a requirement to be able to extract a trip table of all those trips that cross a cordon around an area of interest – called ‘extract sub area trip table’. In commercial packages, both these processes use a set of all-or-nothing paths that may be built specially, or may be the last set used in the iterative process of assigning traffic to a network. In the first case, the selected link volumes will only match the assigned volumes if an additional final assignment is done at the end of the balancing process. If this is done, the speeds that would result if a capacity restraint was applied after the assignment will not match the assumed speeds used to build the paths. If the final set of paths from the assignment process is used (assuming the all-or-nothing path process was used), the selected link volumes will not match the assigned volumes. In commercial packages, Dial's multi-path process cannot be used for selected link analysis or 'extract sub area trip table'.

When using the incremental assignment method, selected link analysis and extract sub area trip table have to be made part of the assignment process as it would not be practical to use the multiple sets of paths involved. In MRDLOAD, the functions of selected link analysis and extract sub area trip table work with Dial's multi path algorithm. The selected link volumes match the assigned volumes but sometimes there is a maximum difference of one, resulting from differences in rounding numbers.

If the numbers of fixed route buses per day is known and is coded onto the network, the bus numbers can be preloaded onto the link before the assignment starts. This is entirely up to the user to decide if the effort is worthwhile. Currently this feature is being used by Main Roads in the Perth traffic model to represent delays to road vehicles at railway level crossings. The total length of time in the day that the crossing is closed to traffic is converted into an equivalent number of road vehicles that could pass a point in the same time. An additional link representing the rail track is added to the network and the estimated volume of road traffic coded onto it. This volume is preloaded before the assignment starts. The other end of the special link then has to be connected to the network with a long enough link so that it will not be chosen as a suitable path during the assignment. The junction of the special link with the network is coded as a signalised intersection. If a project grade separates the crossing, the special link is deleted to provide the project network. Project evaluation is carried out in the usual way.

Another feature of the assignment program is its ability to ‘carry along’ up to nine additional trip tables while it is assigning the primary trip table. With incremental assignment this is the only way of being able to assign a matrix of trips (such as trucks) that is less than total traffic, to the network. Given that trucks have an effect on road capacity that is greater than a car, this gives the user a choice of making a primary trip table of passenger car unit equivalents to use to do the capacity restraint and to ‘carry along’ either the total vehicle trip matrix or separate matrices of light private vehicles, light commercial vehicles and heavy commercial vehicles.

3.10 Example of the use of the carry along feature

In most assignment methods in commercial packages, it is not possible to assign a different trip matrix and have all the assigned trips use the same paths that they used in the full assignment. The exception is the equilibrium assignment, but in this case it is not possible to calculate intersection delays separately. In the program outlined here, separate trip matrices can be assigned to the same paths as used when assigning the full trip matrix at the same time as separate intersection delays are being carried out at all nodes in the network. An example of how this feature has been used is reported in the next paragraph.

In the early days of settlement of the Swan River Colony, a settlement developed at Guildford at the upper limit of the navigable section of the Swan River. Roads into the hinterland fanned out from this point. Consequently, with the advent of motor transport, the primary road network converged on the area like the spokes of a wheel. The development of the Perth Metropolitan Region Planning Scheme has provided a ring of controlled access roads around the area. Subsequent to the building of these roads, a move developed to downgrade the original primary roads and to discourage through traffic through the area. This resulted in a request to determine how much through traffic would be passing through the area under various road schemes. The planners also wanted to know what routes the local traffic would be using. Two supplementary trip tables were prepared. The first one deleted all the trips that either originated or terminated at zones inside the ring of controlled access roads. The second table was the complement of this, as it contained only those trips that originated or terminated inside the ring. A 'carry along' assignment was then done where the total trip matrix controlled the capacity restraint process and the other trip matrices were carried along using the same proportions and paths at each stage of the assignment. The results were then plotted for display to the planners.

3.11 Validation

This assignment and evaluation system started initially as a link only system with no separate calculations for intersections. At that time a new traffic model was being calibrated from the 1976 travel survey. Over 1500 one-way traffic counts were coded into the network to use for checking the accuracy of the traffic model. The best root mean square error between the model flows and traffic counts that was achieved was 4320 vpd. When this link node system was implemented about a year later, the root mean square error reduced to 2676 vpd. This shows the importance of doing separate intersection delay calculations at all intersections if one is to produce accurate estimates of traffic volumes.

During 1981/82, Kenworthy and Newman collected travel time information on 12 selected routes in the Metropolitan area as part of a NERDDC funded research project to develop a driving cycle for Perth. The routes were selected to give representative percentages of all road types and volume ranges in seven areas. All routes thus run partly in the peak direction of travel and partly in the off peak direction. Travel times were collected in the morning peak, through the middle of the day and in the evening peak. This data was made available for comparison with the travel times estimated from traffic assignments. A measured all day average speed was calculated by taking 10% of each of the morning and evening peak times and 80% of the middle of the day times. This was necessary as the assignments were daily. The results for each of the 12 routes are listed in Table 3.4. The results show very good agreement between estimated and measured speeds.

Table 3.4 Comparison of predicted and measured speeds on 12 selected routes

Route no.	Length km	All day average speed measured	All day average speed predicted	Ratio	Average link length km	Average intersection V/C	Stops/km
1B	11.90	33.66	36.84	1.094	0.124	0.50	2.40
1C	15.80	43.86	49.35	1.125	0.269	0.51	1.20
2A	16.80	41.48	38.90	0.938	0.354	0.67	1.40
2B	14.50	42.97	43.25	1.007	0.298	0.65	1.30
3A	17.80	46.95	46.28	0.986	0.382	0.61	1.00
3B	18.40	45.42	46.95	1.034	0.377	0.47	1.10
4A	19.40	44.31	47.12	1.063	0.368	0.53	1.00
4B	17.80	48.42	43.74	0.903	0.392	0.65	1.00
5A	18.40	52.96	57.57	1.087	0.636	0.43	0.70
5B	19.80	51.17	51.08	0.998	0.488	0.47	1.00
7A	19.40	46.67	43.59	0.934	0.416	0.67	1.20
7B	20.30	48.86	44.97	0.920	0.422	0.62	0.80
			Mean	1.007			
			Std Dvn	0.074			
			CV	0.074			

CV = Coefficient of variation

3.12 Summary

This chapter has listed the desirable features of a traffic assignment process. An incremental assignment program that meets the specification is described together with some of its additional features. The results of sensitivity testing are provided to show the effectiveness of the program and to show that the program responds positively to small network changes. In the next chapter, the desirable features of an urban evaluation system are outlined. Some of the choices that have to be made are reported, followed by the choices made in the design of this evaluation system.

Appendix 3.1 Derivation of formulae for marginal speed and peak speed

When deriving the all day average speed from hourly speed-flow curves, daily capacities and a histogram of hourly flow rates as percentages of the daily flow, formulae of the form of equation 4 are fitted to the data to give the average all day time of each vehicle.

$$Time = A * VC^{B} + C \qquad (4)$$

where A, B, and C are coefficients

VC = volume capacity ratio

During the assignment process, the delay to the marginal vehicle is required. This is obtained by converting to the total delay of all vehicles then differentiating. The total delay to all vehicles is then

$$Time1 = A * VC^{B+1} + C * \mathrm{VC} \qquad (5)$$

where Time1 = Total all day time of all vehicles

Differentiating this formula will give the time of the marginal vehicle, i.e.

$$Time2 = (B+1) * A * VC^{B} + C$$

$$= A' * VC^{B} + C \qquad (6)$$

where Time2 = Time of the marginal vehicle being added to the system.

When the assignment process is finished, this needs to be converted back to the total delay to all vehicles. Integrating between zero flow and actual flow and dividing by actual flow will provide an estimate of average all-day travel time per vehicle. Integrating the formula between actual volumes and (1.0 minus the proportion of travel in the peak) times the actual volume will provide an estimate of total peak period total vehicle time. Dividing this value by the interval in volume will convert it to an estimate of time of travel in the peak, i.e.

$$Time3 = (A' * VC^{B+1} / (B+1)) / VC + C$$

$$= A' * VC^{B} / (B+1) + C \qquad (7)$$

and

$$Time4 = A' / (B+1) * (VC^{B+1} - (1.0 - Peakpc) * VC^{B+1}) / (Peakpc * VC) + C \qquad (8)$$

Where Time3 = All-day average time of all vehicles

Time4 = Average time of all vehicles in the peak

Peakpc = Percentage of all day travel in the peak period

Similarly for vehicles that approach a stop, give-way situation, the corresponding formulae are

$$Time = \alpha * Min * \varepsilon^{\beta * Majf} + C \qquad (9)$$

$$Time2 = \alpha * Min * \varepsilon^{\beta * Majf} * (2.0 + \beta * Majf) + C \qquad (10)$$

$$Time3 = \alpha * Min * \varepsilon^{\beta * Majf} + C \qquad (11)$$

$$Time4 = \alpha * Min * (\varepsilon^{\beta * Majf} - \varepsilon^{\beta * Majf * VCPCNT} * VCPCNT^{2}) / (1.0 - VCPCNT) + C \qquad (12)$$

Where Time = Average all-day time per vehicle

Time2 = Time of the marginal vehicle being added to the system

Time3 = All-day average time of all vehicles

Time4 = Average time of all vehicles in the peak

Min = Volume approaching the stop, give-way situation

Majf = Two-way volume on the priority road

VCPCNT = Proportion of all-day travel outside of the peak period

α, β and C are constants

Appendix 3.2: Derivation of the equation for excess time

Excess time is defined as the time taken to decelerate to a stop, then accelerate back to cruising speed, less the time taken to travel the same distance at cruising speed.

First consider the acceleration component.

Consider a vehicle accelerating from rest to speed V_f over a period{0,T}, with its speed being governed by the formula

$V^2 = A + Bt$

At $t = 0$ $V = 0$ So $A = 0$

And $V^2 = Bt$

At $t = T$ $V = V_f$

So $V_f^{\,2} = BT$

Hence $T = V_f^{\,2} / B$ (13)

Distance covered during acceleration, x_1

$x_1 = \int_0^T V.dt$

$= \int_0^T \sqrt{Bt}.dt$

$= B^{1/2} \int_0^T t^{1/2}.dt$

$= B^{1/2}.2/3t^{3/2} \Big|_0^T$

$= B^{1/2}.2/3T^{3/2}$

But $T = V_f^{\,2} / B$ from (13)

So $x_1 = 2/3\, B^{1/2}.\, V_f^{\,3} / B^{3/2}$

$= 2/3\ V_f^{\,3} / B$ (14)

Now consider a vehicle decelerating from V_f to rest over a period {0,T}, and its speed being governed by the relationship

$$V = C - Dt$$

At $t = 0 \quad V = V_f = C$

So $\quad V = V_f - Dt$

At $t = T \quad V = 0$

So $\quad DT = V_f$

$$T = V_f / D \tag{15}$$

Consider the distance travelled, x_2

$$x_2 = \int_0^T v.dt$$

$$= \int_0^T (V_f - Dt)\, dt$$

$$= V_f t - ½ Dt^2 \Big|_0^T$$

$$= V_f T - ½ DT^2$$

But $T = V_f / D$ from (15)

So $x_2 = V_f^2 / D - ½ D (V_f / D)^2$

$$= ½ . V_f^2 / D \tag{16}$$

Total distance covered during deceleration and acceleration

$X = x_2 + x_1$

$= ½ . V_f^2 / D + 2/3 V_f^3 / B$

To cover X at cruising speed takes time

$t_c = X / V_f = V_f / 2D + 2 V_f^2 / 3B$

The time taken during acceleration and deceleration, t_s, is from (13) and (15) above

$$t_s = V_f^{\,2} / B + V_f / D$$

Nett increase in travel time

$$= t_s - t_c$$

$$= V_f^{\,2} / B + V_f / D - V_f / 2D - 2V_f^{\,2} / 3B$$

$$= V_f^{\,2} / 3B + V_f / 2D \qquad (17)$$

Now check for consistency of units.

V_f in $Km.hr^{-1}$

B in $Km^2.hr^{-2}.sec^{-1}$

D in $Km.hr^{-1}.sec^{-1}$

$$t = (Km.hr^{-1})^2 / Km^2.hr^{-2}.sec^{-1} + Km.hr^{-1} / Km.hr^{-1}.sec^{-1}$$

$$= Km^2.hr^{-2} / Km^2.hr^{-2}.sec^{-1} + Km.hr^{-1} / Km.hr^{-1}.sec^{-1}$$

$$= sec + sec$$

Incorporating values from Samuels and Jarvis (1978).

$$t = V_f^{\,2} / 960 + V_f / 14 \quad secs \;(V_f \text{ in } Km / hr)$$

Reasonableness check $\quad V_f = 60$ Km / hr

$$= 60 * 60 / 960 + 60 / 14$$

$$= 3.75 + 4.29$$

$$\cong 8 \text{ sec}$$

Appendix 3.3: Calculation of delays at signals by approach leg in lieu of average delay to all vehicles

NOTE: In the early stages of an assignment, the relative volumes on each approach fluctuate wildly but gradually settle down as the assignment proceeds. It is therefore necessary to minimize the leg green/ average green ratio adjustment early, gradually moving towards full adjustment near the end of the assignment.

The following is calculated once per capacity restraint

$$x = (a/b - 0.4) * 10.0 \qquad (18)$$

Where a = total trips loaded so far

b = total trips to be loaded

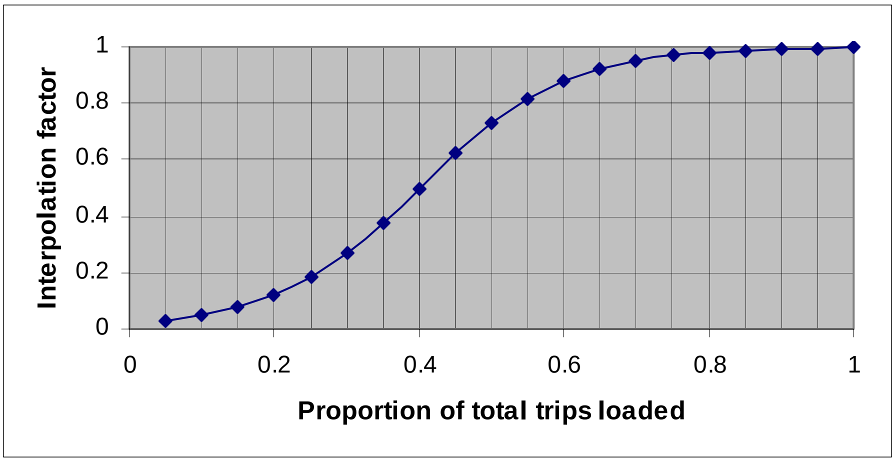
x = temporary variable

$$VCC = e^{x} / (1.0 + e^{x}) \qquad (19)$$

Where

VCC = adjustment based on proportion of total trips loaded so far

Figure A3.3.1 Interpolation factor between equal share of green time and actual share of green time



Calculate slope of curve relating leg/intersection ratio of VCs which depends on the intersection load.

The following is calculated once per signalized intersection when the intersection volume / capacity ratio is known

x = intersection volume / capacity ratio

If (x > 1) x = x * x (20)

x = 360 * x – 234 (21)

subject to x being not less than – 90 and not more than 270

x = Sine (x / 57.29578) (22)

$y = 1.01 + 0.85 * e^x /(1.0 + e^x)$ (23)

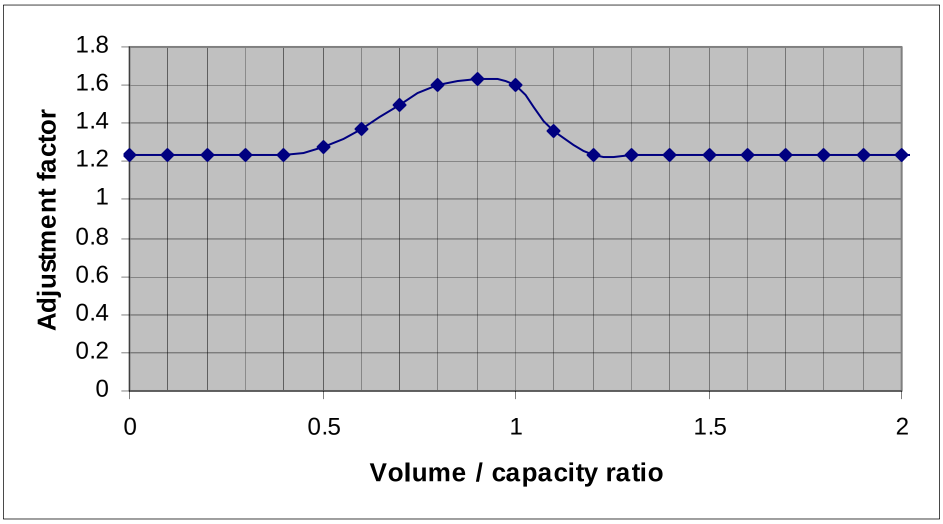
RAA = VCC * y + (1.0 - VCC) (24)

RAB = ALOG(1.0/RAA) (25)

Where

- x = intersection volume / capacity ratio
- y = working variable
- VCC = factor from equation 19 above
- RAA and RAB are required adjustment constants

Figure A3.3.2 Adjustment factor versus intersection volume/capacity



The following is calculated for each approach leg to a signalized intersection

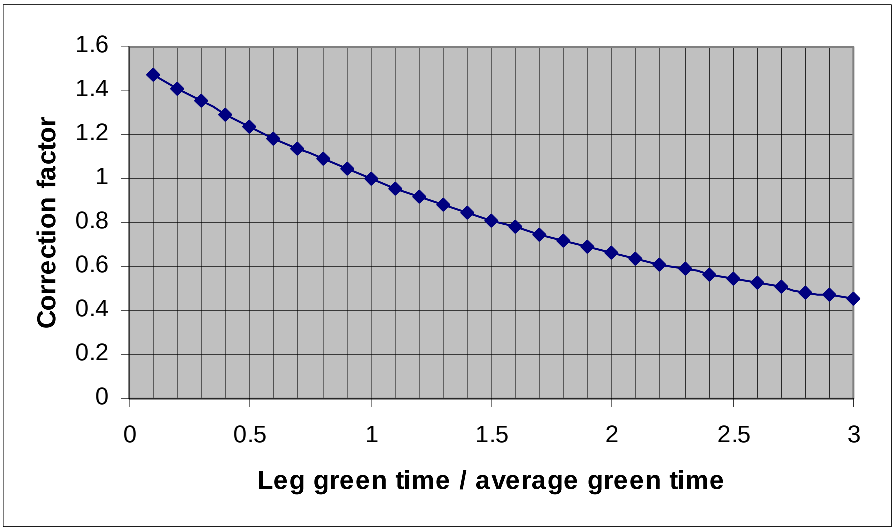
- a = link approach volume * number of phases / the approach uninterrupted capacity at the stop line (26)
- $c = RAA * e^{(RAB * a/x)}$ (27)
- VCL = c * VCC + 1.0 – VCC (28)

Where,

- VCL = delay correction factor to be applied to the average intersection delay
- a = approach leg volume/capacity ratio
- x = intersection volume/capacity ratio
- RAA and RAB are constants calculated in equations 24 and 25 above
- VCC from equation 19

c = working variable

Figure A3.3.3 Factor for leg delay times average delay



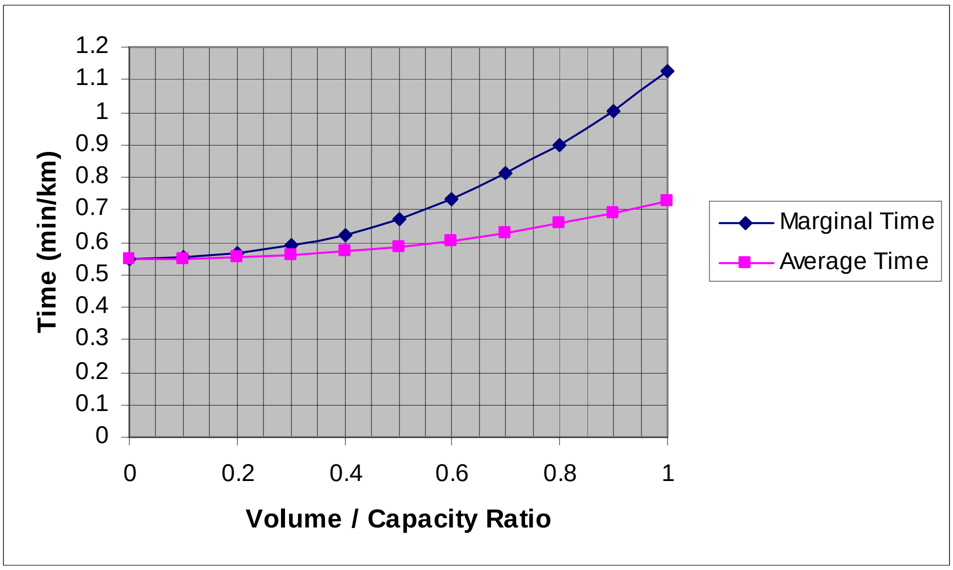
Appendix 3.4 Differences between 'incremental' assignment in TRIPS and incremental assignment in TRAMS

Both methods are assigning a portion of the total trips, then revising the speeds, followed by the addition of a further increment of trips being added to the trips already assigned. This process is repeated a number of times until all trips are assigned. However, there are two important differences between the 'incremental' assignment in TRIPS and the incremental assignment used in TRAMS. The first one is in the way the speeds are used during the assignment and at the end of the assignment. The second difference is in the way increments of trips to be assigned are chosen.

To illustrate the different way speeds are being used, as an example, let us assume that on a particular link, five equal increments of trips are used, each equivalent to 0.2 of capacity. In TRIPS, the initial travel time is set equal to the average time for a volume / capacity (V/C) of 0.0. (see Figure A3.4.1) The first increment of trips is then assigned. The travel time is then revised to the average travel time at a V/C of 0.2. The next increment is assigned and the travel time set equal to the average travel time at a V/C of 0.4. This is repeated with the time being set at the average speed at V/Cs of 0.6 and 0.8 with the final travel time equal to the average time at a V/C of 1.0. When calculating road user costs for an economic analysis, all vehicles are assumed to have travelled at the final average travel time equal to a V/C of 1.0 even though the trips were assigned using a lower average travel time (higher average speed).

In TRAMS, a small increment of V/C (about 0.01) is added to the V/C calculated from the link volumes, and the travel time is then calculated from the marginal time curve. So the initial time will be set from the marginal time curve at a V/C of 0.01. The first increment of trips is then assigned. The travel time is revised for a V/C of 0.21 using the marginal time and the next increment of trips added to those already assigned. This is repeated for V/Cs of 0.41, 0.61 and 0.81. At the end of the assignment, the final travel time saved in the network file is calculated from the average time curve. For the economic analysis, vehicles are assumed to have travelled at all the speeds on the marginal time curve between a V/C of 0.0 and 1.0, corresponding approximately to the speeds used during the assignment.

Figure A3.4.1 Marginal and average travel time per km



The second major difference between the methods is in the choice of trips to be assigned in each increment. In TRIPS, the same user specified proportion of trips leaving a zone is assigned for all zones before revising link times. To be effective, it is preferable to use at least eight to ten increments in an incremental assignment process so paths have to be built from all zones for each increment. With the smallest zones having less than 500 trip origins while the largest zones had more than 40,000 trip origins, it did not make sense to build up to 10 sets of paths for the small zones.

In TRAMS, some trips are assigned from selected zones, and then the travel times are revised. Some more trips are assigned from a further selection of zones and the travel times again revised. On average, travel times are revised anywhere between about 60 and 80 times, depending on both the number of zones being used and the number of trips in the trip table. The effect is that for those links that are used to access only one zone, the number of times that the link volumes will change when the times are revised will be somewhere between one and about eight depending on the number of trip origins. However, for links near the centre of the network, the number of times that the link volumes will change when the link times are revised can be as high as 25 to 35. In this case the effective increment in the volume / capacity ratio between successive link time revisions is about 0.02 (assuming a normal average upper volume / capacity ratio of about 0.7). So in the area of the network that is most likely to be overloaded with significant changes in the travel time with only small changes in link volumes, the effective volume increments are quite small. The total number of paths that are built is only about five times the number of zones even though the percentage of the total trips being loaded between travel time adjustments is less than 3% (In TRIPS, if 3% increments are used the number of paths to be built would be 34 times the number of zones).

Path building accounts for more than half of the computer time when doing an assignment so the saving in computer time is considerable. At the time of development, a single assignment was taking about 15 minutes of CPU time on a mainframe with the elapsed time being considerably more. With the speed of modern day PCs this does not have the same importance as it used to. However, the very small increment of trips being loaded between successive link time adjustments in TRAMS yields sensible link volume comparisons, even for very small network changes such as adding a turning lane at an intersection. Also for most of the links, the small increments in volume between successive restraints more closely approximates to the continuous time curve assumed in the economic evaluation.

CHAPTER 4

DESIGN OF THE URBAN EVALUATION SYSTEM

4.1 Introduction

The purpose of this chapter is to list the desired features in an urban road project evaluation system. All the variables that influence road user costs are described and any trade offs that have to be made between accuracy and the need to keep the workload and computer run times down to acceptable levels are discussed. This leads into a general outline of the economic evaluation system with the grouping of variables into those that are input at the individual link level, those that are input for each capacity indicator group and those that are input at the study area level.

Realistic economic assessments of road projects in an urban environment require an integrated system that can provide accurate estimates of traffic volumes and travel speeds. The system needs to allow for the fact that over a whole day not all vehicles on any one link travel at the same speed. Intersection capacity needs to depend on the relative flow rates on each approach. Network coding should not be too onerous, preferably being limited to the physical characteristics, such as the type of road, the number of lanes mid block, the speed limit, the number of turning lanes, and the type of intersection control. Intersection analysis should be carried out at all intersections. It should be possible to vary traffic composition across a network. Vehicle operating costs should be built up from the basic building blocks of fuel and oil consumption, tyre wear, depreciation and vehicle repair and maintenance.

Accurate estimates of travel time and vehicle operating costs can be made by splitting travel into three components:

- travel between intersections at approximately constant speed for the full distance
- the time spent stationary at intersections waiting for the signals to change or waiting for a gap in the traffic on the cross road
- the time and costs during deceleration to stop and acceleration back to cruising speed less the time and cost to travel the same distance at the cruising speed.

4.2 Variables that influence urban road user costs

From the measurement of road user costs in uninterrupted flow conditions, it has been found that the following variables have an influence on the total road user costs on a road link [Claffey(1971), Commonwealth Bureau of Roads (1972), Thoresen(2000)]. Total vehicle kilometres and total vehicle hours of travel have a major impact. Also important are the number of traffic lanes available for through traffic, the road surface type (i.e. whether it is sealed or unsealed), the speed of travel, the variation in the rate of flow of traffic throughout the day, and the traffic composition in terms of the proportions of each road vehicle type. Other variables include the gradient and curvature of the road, the width of the traffic lane, and the roughness of the road surface [Claffey(1971), Commonwealth Bureau of Roads (1972), Thoresen(2000)]. More recently it is becoming evident that the texture of a sealed road also has some effect on fuel consumption and road tyre wear but as yet no mathematical relationships have been published. For interrupted flow conditions, the need to stop at intersections adds considerably to road user costs, particularly in terms of increased time and increased fuel consumption. The presence of intersections also adds considerably to the number of road crashes.

4.3 Desirable features of an urban evaluation system

In an urban network, any change to the network almost always causes traffic to change route. For some projects the impact is spread over a very large portion of the network. This means that evaluation of urban road projects needs to consider all the links in the network in order to correctly estimate the change in road user costs. With the very large number of links in an urban forecasting network it is essential that the evaluation system be integral with the process used to forecast traffic.

If an evaluation system for urban road projects is going to produce credible results, it must have the following features.

- It must consider the effect of intersections on urban travel costs.
- It must consider the variation in hourly flow rates throughout the day. Given the very rapid change in speed with small changes in traffic volumes as flows reach capacity, the hourly flow increment as a percentage of the daily flow rate needs to be quite small.
- It must consider the network effects of traffic changing from one route to another as a result of the project being evaluated.
- It needs to consider traffic volume and speeds at the link level.
- It must consider traffic composition.
- It is desirable that the effect of road roughness, road gradient and road curvature on the base level of road user costs be taken into account.
- It should be possible to build all components of the system from basic data that is measurable.
- As there are of the order of 10,000 + links in an urban network, the evaluation component needs to be integral with the process used to forecast traffic.

4.4 Type of system – matrix driven system versus a link driven system

One approach to estimating road user costs is to work at the matrix level using matrices of zone to zone travel times, travel distances, the number of person trips, and the number of vehicle trips. This method can only use average journey speed to estimate vehicle operating costs. The cost of stopping at intersections will not be directly taken into account. It is not possible to use a profile of the variation in the traffic flow rate throughout the day. Also it is not possible to vary the crash rate by the type of road or to separately estimate crashes at intersections from crashes that occur between intersections. These approximations will not give a reliable estimate of the benefits of road projects. Traffic forecasts and hence matrices are usually only available for two points in time within the analysis period. Although the growth in traffic volumes on a link is normally linear, the change in the zone to zone travel times is not linear. The change in road user costs moves even further away from linear as the costs also vary with the speed of travel. As an assumption of a linear growth in road user costs is the only one that can be made using two points in the analysis period, this introduces another source of error.

The alternative is to calculate road user costs at the link and intersection level. This has the following advantages.

- Estimates of vehicle operating costs can be made as the sum of three components, cruising at constant speed, the acceleration/deceleration phase and time spent idling at intersections.
- For intermediate years, traffic volumes can be estimated by linear interpolation between the forecast years at the individual link level and then the vehicle operating costs calculated.

- Items like fuel consumption, which are speed dependent, can be calculated using the actual speed on the individual legs of the journey instead of the average journey speed.
- The costs of stopping at intersections can be correctly estimated.
- The variation in traffic flow rate throughout the day can be used.
- Road crash rates can be separated by road type and by intersection type.
- The proportions of each type of crash (fatality, injury and property damage only) can be varied by the speed limit on each road.
- It is reasonable to assume a linear growth of traffic on a link between forecast years but the change in road user costs will not be linear due to changing levels of congestion.
- Network changes such as adding a lane or installing traffic control signals will have a measurable effect on the altered links, which in turn affects costs, but the change at the average journey speed level will be minimal.
- The size of the study area used for economic analyses can be varied to suit the problem; eg. adding turning lanes at signals can be analysed by looking at one intersection only and assuming that traffic volumes do not change as a result of the project.

The disadvantages are:

- It is not possible to assign project benefits to people living in defined areas as distinct from trips travelling in those areas. This notional advantage of evaluation systems based on overall journey values is not accurate anyway as about one third of all journeys have neither end at home. As these are mostly the high value trips (heavy and light commercial vehicles plus people travelling on employers business) at least half the benefits would be incorrectly ascribed to the people living in the defined area.

- It is not possible to vary the value of private time savings above and below a threshold value; eg if a value is only placed on private time savings when the saving per trip is more than a threshold time period (in minutes).

Private travel time savings resulting from constructing a road project in an urban area are usually made up of quite small savings per person applied to a very large number of people. As such the total value can be quite large. Opinion is divided on whether pricing these savings is valid or not, with one argument in favour of only putting a price on private time savings if it exceeds a threshold value per trip. However this calculation can only be done when working at the matrix level. By doing the evaluation twice, once with a value on private time and once with a nil value on private time, the user can assess the effect of valuing private time on the results. There is seldom any change in the ranking order of projects using benefit cost ratios calculated with or without a value on private time. The absolute value of the benefit cost ratio changes, but for projects in an urban area, the values are high enough to justify the project without putting a value on private time.

The link / intersection approach was adopted for the above reasons. For a multi mode evaluation system, person travel on modes other than road vehicles would need to be calculated using the matrix approach and added to the values calculated for road vehicles at the link level. This is acceptable as for person travel, it is only the overall journey totals of time and out of pocket costs such as fares that make up system user costs.

4.5 Potential sources of data for an urban project evaluation

The traffic forecasting networks contain the length of each link, the capacity of the link, and the road type. The speed of travel is estimated from the traffic volume and the link capacity. In normal project evaluations, estimates of traffic at two points in time are available for each link. Any other variables have to be provided in some other way.

The speed limit is available on speed zoning plans. The number of turning lanes at an intersection is on road construction plans. The type of intersection control is on plans if signalised, or on sign inventory lists if control is by stop or give-way signs. Lane width, road roughness (if available), road curvature and road gradient are contained in a computerised inventory of the road system. Traffic forecast networks contain links representing future roads as yet unbuilt that do not exist in the inventory. Trying to link the inventory to a traffic forecasting network is time consuming, as there is not a one for one match between records in the inventory and the links in a forecasting network. To have to link the inventory to the forecasting networks every time a project is to be evaluated is not practical.

Until the advent of laser technology in the nineteen nineties, the equipment used to measure road roughness had to be driven at constant speed and gave a reading every 0.5 km. This limited its use in urban areas to grade separated roads and roads with very long lengths between controlled intersections. Road roughness measurements on Perth roads were collected only once and this was on less than ten percent of the total primary road length. With the extra strength built into urban pavements to save having to rebuild them later, there is very little relationship between pavement age and roughness (Potter 1982). Items such as service trenches, drainage gullies and tree roots have a far greater impact on road roughness than changes in pavement age. The method of resurfacing the pavement to keep it waterproof means that the roads rarely reach a level of roughness that affects vehicle operating costs sufficiently to generate much benefit if the roughness level is reduced by the project. These days, service trenching has been replaced with tunnelling but there is still little correlation between pavement age and roughness (Magnus 1998). In urban areas, lane widths tend to be a uniform standard width. This shifts the primary determinant of road user benefits from considerations of the road geometry and the condition of the pavement to network effects of changing route, changing levels of congestion and alterations in the likelihood of having to stop at intersections.

By assuming that roughness does not increase with increasing pavement age, it is possible to move considerations of roughness from the individual link to groupings of links defined by their index into the speed flow curves used in the traffic forecasting process. At the slower operating speeds on urban networks, road curvature has very little impact on vehicle operating costs. Very few roads have gradients in excess of 3%. For these roads it is unlikely that the addition of a road project to the network will alter the volume of traffic on the grades over 3%. Detailed traffic composition figures are known at only a very small number of sites in the urban area. Moving road roughness, road curvature and gradient from individual links to a grouping of links makes it unnecessary to connect the traffic forecasting networks to an inventory of the road system.

4.6 Where to introduce the variables into the system

In an open road situation, particularly with the relatively low traffic volumes experienced on Australian country roads, the largest portion of road user benefits come from items such as changing an unsealed road to a sealed road, widening the lanes on narrow sealed roads and reducing road roughness and road curvature. Only rarely does the hourly demand flow exceed hourly capacity, causing queues to develop so that increasing capacity rarely provides much benefit. There is a relationship between pavement age and road roughness. Pavements are rebuilt when the level of roughness reaches values that the travelling public will no longer accept. For the roads carrying larger volumes of traffic this may occur at a lower level of roughness when the benefits of reducing the road roughness exceed the cost and finances are available to fund the work.

The system for evaluating road projects in an open road situation is built around reading an inventory of the road system that contains details about the geometry of the road, details of the road surface type, age and roughness and details of the traffic volumes including composition and growth rates. It has been customary to ignore network effects and all traffic flow is treated as being uninterrupted. Usually no more than about twenty records are processed for any one project. The histogram of hourly flow rates throughout the day uses quite coarse increments of 4% of the daily flow (2% in the system used in Western Australia). For each record, the effect of every input variable on road user costs is considered.

In an urban environment, for the larger urban centres, anywhere between ten percent and fifty percent of daily travel occurs when the demand exceeds capacity and queues exist. Due to the very rapid change in speed as hourly flows reach hourly capacity, it has been found necessary to reduce the hourly flow increment as a percentage of the daily flow to as small as 0.05%. With the number of links in an urban network exceeding 10,000 (the number in the networks used in Perth is over 13,000), if the method of processing links in the urban system was the same as that used in the rural system, the relativity of the computer times would be:

Urban – 13,000 links times 300 flow increments times 14 vehicle types for 30 years

Rural - 20 links times 10 flow increments times 16 vehicle types times 16 years (the system evaluates every second year and then linearly interpolates for the intervening years).

This would give a computer run time for urban evaluations of about 32,000 times the run time for a rural project. With the run time for a rural project in excess of one second, the run time for an urban project could exceed nine hours, so some way had to be found to reduce the run times.

The speed limit and intersection information is required for every link. Given their minor effect on road user benefits for urban road projects, average values of road roughness, and road curvature by road type are suitable. There is some traffic composition data available for urban roads, but it is not possible to disaggregate this down too far, possibly as far as road type is about the limit.

If this additional information was added at the network building stage, the format of the network files would need to be changed. This would mean changing all the programs that read network files (between 10 and 15 programs). In addition, it would no longer be possible to read historical network files unless additional computer code was written. Each link already contained an index number that is used to access the appropriate speed flow function. By ascribing additional details to the speed flow function and increasing the number of speed flow curves, it is possible to include the intersection information and the speed limit without changing the network file formats.

4.7 How to provide the intersection information

The simplified method of analysis to calculate vehicle delays at intersections requires the following information:

- the type of intersection control
- which approaches have to give way for sign controlled intersections
- the number of through lanes on each approach
- the number of lanes at the stop line on each approach
- the number of phases for signal controlled intersections
- the speed limit on the approach roads.

To provide this information by adding to the details on each link record would not only add considerably to the work of network coding, but would also require the computer network files to be changed. The decision was therefore made to attach this additional information to the speed flow curves. This was achieved by adding five more fields to the speed flow function data. The first three are the coefficients in the formula for intersection delay, the next field is a code to indicate the type of intersection control on that approach, and the last field contains the number of added turning lanes on that approach. Rather than have a separate node or junction file in which to store the number of phases at signals, it was decided to let each program apply a formula to the number of approaches to the intersection with minimum and maximum overrides. In this way, adding a link to a network, or deleting an existing link from the network would not involve making changes to other links, or to intersection information, thereby reducing the amount of work and the risk of introducing errors into the network.

As a result of these changes the speed flow curves now provide the following additional information to a network for each individual approach:

- the coefficients for calculating the intersection delay times
- the type of intersection control
- the number of turning lanes.

The advantages of providing intersection information via the capacity indicator are threefold. The primary advantage is to minimise coding time when a network is being modified by adding new links or by deleting existing links. Each link contains only that information pertaining to it so that only the link being added or deleted needs to be changed. Intersection capacity and the number of phases are calculated internally by the assignment program from the information provided for each link. The second advantage is that the existing computer network file layout did not need to be changed or a new file of intersection data created and maintained with the obvious risk of mismatching network and intersection files during any production run. Thirdly, the system calculates intersection delays at every intersection in the coded network for practically no change in the workload of network coding from that involved when intersections are ignored.

4.8 Determination of the intersection type from link details.

There are three programs that need to know the intersection details, a network editing program, the assignment program and the project evaluation program. They determine the type of intersection from the intersection type code on the capacity indicator for all links approaching the intersection. Three intersection type codes are used on the capacity indicators

0 = approach to a merge (such as freeway on ramp)

1 = approach to signals or the major approach to a major / minor intersection

2 = minor approach to a major / minor intersection

From this information, the system distinguishes between freeway merges, signalised intersections, sign controlled intersections, and the special cases where there are insufficient approach links to a node for an intersection to exist. This happens at a small percentage of nodes in a network where a link providing the third leg to an intersection is deleted. If the number of approaches to a node is 2, the number of exits is 2 and the road is a two way road, the program assumes that an intersection does not exist. The node is referred to as a dummy node and no intersection time (or cost) calculation is performed. If the road is one way, the count is 1 approach and 1 exit to define a dummy node.

If all the approach codes are 1, the intersection is assumed to be controlled by signals. The number of phases is estimated from the number of approach legs with over rides from parameters for the minimum and the maximum number of phases. If there are no code 2s but one or more code 0s the intersection is treated as a merge and analysed as a one phase set of signals with no delay at minimum flow. If there are any code 2s the node is treated as a major / minor intersection. If there are two code 1s and the balance are code 2s, the program assumes that the intersection has been correctly coded. Approach links with code 1s will be analysed as if approaching a one phase set of signals with flows from the major road only and with no delay at minimum flow to mimic the delaying effect of right turning vehicles at higher flows. The links with code 2s will be analysed by assuming they are attempting to cross both major streams of traffic in a stop and give way situation. If the numbers of code 1s and code 2s are not correct, rather than cancel the run, the program searches for the two largest approach flows to use as the major road. All other approaches are treated as minor approaches. A mixture of one way and two-way roads complicates the decision process but the principle is the same.

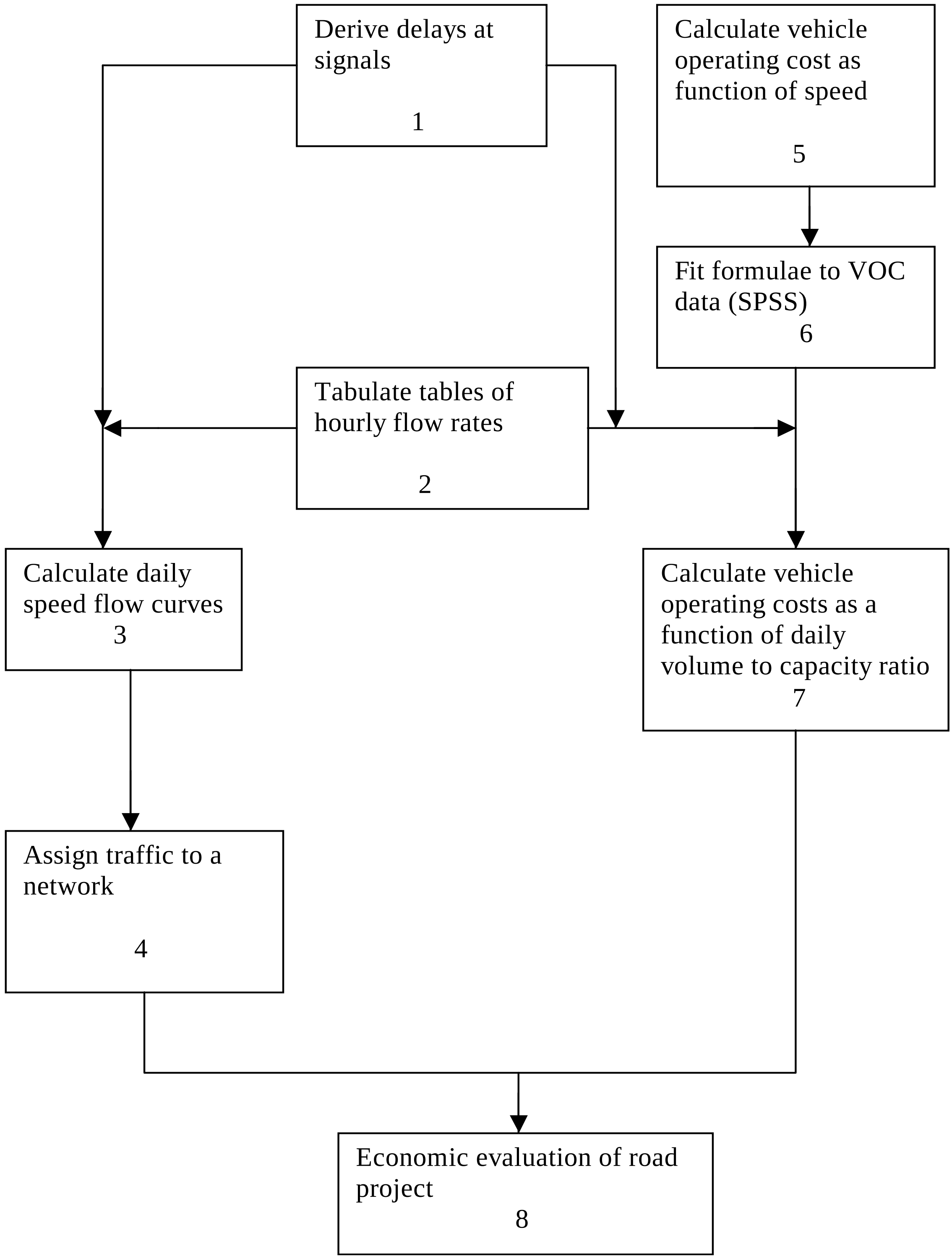
This same decision structure is used by a number of programs in the system, including the assignment program, the network editing program, the economic analysis program and the program for calculating emission data. For the assignment program, the estimation of the average delay time at signals is made proportional to the number of phases with the input times being taken as for the maximum number of phases. The delay on each approach is then obtained by modifying this average delay according to the proportion of the green time allocated to the individual approach. Similarly, in the economic analysis program, the intersection costs at signals are modified to allow for the number of phases at the intersection with the input unit costs being for the maximum number of phases. At this time, the question of the effect of unequal shares of the green time on costs has not been investigated. This is something that will be looked at in the near future.

4.9 Structure of the system

All the price variables are constant over the whole study area and change at relatively infrequent intervals of one or two years. Variables such as speed limit, traffic composition, crash rate and hourly capacity per lane can be tied to the index used to select the speed flow curve in the traffic forecasting process. This has made it possible to design the system so that the loops of flow rate and vehicle type are performed once for each speed flow curve number (maximum of 128) once each time the prices are changed to create a file that is used later when processing each link. This has brought the computer time down to something that is quite moderate and acceptable. The full system is a collection of seven programs counting the assignment program. The sequence of using them is shown in Figure 4.1.

The resulting system is modular, comprising seven separate programs counting the assignment program. Programs 1, 2 and 3 are used to calculate intersection delay at signals as a function of the volume / capacity ratio, to provide histograms of the distribution of hourly flow rate and to estimate the daily speed flow functions from hourly speed flow functions. They are likely to be used only once when first setting up the system. Steps 5, 6 and 7 which convert unit prices of vehicle consumption into the average cost per vehicle-km by volume / capacity ratio are executed once each time unit input prices are changed i.e. maybe once a year or once every two years. Steps 4 and 8, assignment and evaluation, are used to evaluate each project which may be almost a daily occurrence.

Figure 4.1 Components of the system



As a user, a big advantage of the system is that by splitting all calculations back to basics, the primary input of items such as hourly speed flow curves and hourly capacities is something that can be measured directly and does not require interpretation as it is not a compromise between two things such as having to adjust the speed flow curve to include an ‘average’ time allowance for delays at the ‘average’ intersection on a link of ‘average’ length. This means that an assistant coding updates to a network from the correct ‘raw’ information of the type of road, the number of lanes, the speed limit and the type of intersection control will get the same coding as the person who originally coded the network years earlier, i.e. he does not have to decide what capacity to use or if a link falls in the middle suburb or the outer suburb, because this ‘grey area’ type data is not used in TRAMS.

4.10 Summary

The aim with TRAMS is to build it on to the traffic forecasting system used to forecast traffic in a network without adding unduly to the workload required to operate it and without changing the format of the network computer files. At the link level, this makes it necessary to restrict it to only those items that are contained in the files used to forecast urban traffic. Other data can be brought in at the macro level. To try to link it to an inventory of the road details over a whole network would involve considerable work each time an evaluation was to be run. The choice was made not to link it to a road inventory but to use the variables from the inventory as group averages. The variables used by the system and the point at which they are introduced are listed in **Table 4.1.**

Table 4.1 Point of entry of variables into the system

Variable	Where introduced
Distance Daily capacity Traffic volume Inferred growth rate	Link
Road type Speed limit Hourly capacity Type of intersection control Number of additional turning lanes Inferred lane width Traffic composition * Road roughness * Road gradient and curvature * Road crash rates (between intersections and at intersections) Cost per crash (between intersections and at intersections)	Group of links
Price per litre of fuel Price per litre of oil Purchase price of vehicles Repairs and maintenance price of vehicles Purchase price of tyres and retread tyres Time value of freight per vehicle Commercial time value per vehicle Private time value per vehicle	Study area

* These values are currently introduced at the study area level but the system allows the user to introduce them at the grouping of links level if one is prepared to do the extra work entailed each time the prices are changed.

As the road characteristics are thought to have little impact on urban projects aimed at increasing lane-kilometres of capacity, the system is written to allow the user to input average values of roughness, grade and curvature for the study area, and at the link level to use only the network information contained in the files created by the traffic forecasting process which do not contain roughness, grade and curvature. This allows calculations to be divided into groups based on frequency of use, and then programmed as stand alone programs, resulting in a set of six different programs, excluding the assignment program. The sequence of using the programs is shown in Figure 4.1. Different values of roughness, grade and curvature for each road type/speed limit/intersection type combination, can be used if the user is prepared to do extra work by running two programs for each combination, but in each case roughness is not allowed to change with time. A second consideration in the design of the system is to limit the demands on staff time necessary to develop and maintain all the network and related computer files needed for production work. This has been achieved by limiting the required intersection information to needing the intersection control type, and the number of added turning lanes on each approach.

The method of deriving daily speed flow relations from hourly flows, hourly capacities and the distribution of flow throughout the day is outlined in the next chapter.

CHAPTER 5

DERIVATION OF DAILY SPEED FLOW RELATIONS

5.1 Introduction

The assignment process requires daily speed flow formulae that can be derived from hourly speed flow relations and the distribution of the hourly flow rate throughout the day. Further, it must be possible to differentiate the average speed flow function to give the speed of the marginal vehicle during the assignment. These functions then have to be integrated to give the all day average speed and the average speed during the peak period. This imposes a limit on the form of the functions that can be used to express travel times and delays as a function of the traffic flow. How these relations are derived is explained in the following sections.

5.2 Components of travel time

Travel time is estimated as the sum of three components:

- the time to travel the full distance between intersections at cruising speed
- the time spent stationary at intersections waiting for a gap in the traffic on the cross road or for the lights at traffic signals to change
- the excess time to slow to a stop and accelerate back to cruising speed less the time to travel the same distance at cruising speed.

For convenience, the excess time is added to the intersection stationary time to give total intersection delay time. At the operational level, this reduces the calculation of travel time to two components:

- cruising time between intersections
- total intersection delay time.

The Bureau of Public Roads formula for estimating travel time on a link is

$$T = T_0 (1 + 0.15 (V/C)^4) \qquad (29)$$

Where

T = time to traverse the link

T_0 = time to traverse the link at very low traffic flows

V/C = volume to capacity ratio

This can be simplified to

$$T = (a + b (V/C)^c) D \qquad (30)$$

Where

T = time to traverse the link

V/C = volume to capacity ratio

D = length of link (km)

a, *b*, and *c* are coefficients

In this form, the function can easily be differentiated and then the resulting form can be integrated. The coefficient *a* is the travel time per km at the free or no load speed. It can be calculated directly from the average free speed of travel at each speed limit. The coefficients *b* and *c* for hourly speed flow curves on freeways were derived from the plotted points on the speed flow curve in Figure 3.44 of the U. S. Highway Capacity Manual, taking care to separate the points according to the speed limit on the road on which the data were collected.

Freeways are usually multi lane roads in each direction and vehicle speeds at medium flows tend to be very close to free speeds as the faster vehicles can easily pass any slower vehicles. On the lower road classes, roads are mostly a single lane in each direction and the faster vehicles find it more difficult to pass. Vehicle speeds fall away more quickly from the free flow speed as flows increase, i.e. the coefficient c reduces. For the lower road classes, the coefficients b and c were adjusted to give acceptable speeds at forced flow with a faster loss of speed at medium flows. Care was taken to ensure that the derived all day average speeds at daily volume to capacity ratios of three for the lower roads was less than for freeways at the same volume to capacity ratio.

5.3 Excess time

The excess time for those vehicles that have to stop at an intersection is the total time taken to decelerate to stop, and to accelerate back to cruising speed, less the time taken to travel the same distance at constant cruising speed. This time is a function of the speed limit, the average deceleration rate and the average acceleration rate of vehicles. At stop signs, all vehicles have to stop, but at signals, not all vehicles have to stop, the proportion stopping being dependent on the signal cycle time, the share of the available green time, the speed limit and the load on the intersection. The derivation of the formula for excess time for stopping vehicles is in Appendix 3.2. The rate of deceleration and acceleration were obtained from Samuels and Jarvis (1978).

5.4 Stationary time at traffic signals

The most accurate method of estimating the average delay to all vehicles on any approach to traffic signals uses:

- the cycle time
- the number of phases
- the 'all-red' time
- the traffic flow turning left, passing straight through, and turning right from each approach

- the number of lanes available to each traffic movement
- the movements that are possible on each phase.

The disadvantages are:

- in a forecast situation, much of this information is unknown
- the calculation method is only applicable for time periods up to one hour
- it does not give the delays of the marginal vehicle about to be added to the system.

However, by making the assumption that in the future, the signals will be correctly set to maximise the total throughput of vehicles through the intersection, and redefining the load on the intersection in terms of the volume to capacity ratio, it is possible to use the detailed methods to calculate the average delay per vehicle including the excess time and to express the delay as a function of the volume to capacity ratio.

The capacity of the intersection is the capacity per lane per hour of green time multiplied by the number of lanes at the stop line and divided by the number of phases. The ratio, total volume entering divided by the capacity is a measure of the load on the intersection. The average delay to all vehicles entering can be related to this measure. This process will then provide a realistic estimate of delays at signals.

For vehicle-actuated signals, the length of each phase depends on the demand for that phase. If no demand exists, a phase will be omitted so that over an extended period, the average number of phases used per cycle will be less than the number of programmed phases. The effective average number of phases for three-way and four-way intersections will thus be something less than three and four respectively.

A computer program that reads user values for the number of phases, the lost time or all-red time per phase, the minimum and maximum green times per phase, and acceleration and deceleration constants is used to calculate average hourly rate delay as a function of volume capacity ratio. It uses the algorithm described in Akcelik (1980a) as modified in Akcelik (1980b) on the assumption of equal green times per phase. Data points are calculated in volume capacity ratio increments of 0.02 up to 0.98. The probability of stopping is also calculated. Allowance is made for those vehicles that have to partially slow down but not stop by converting them to an equivalent number of vehicles that have to fully stop.

The formulae in Akcelik (1980) calculate the idling time averaged over all vehicles. To this must be added the excess time times the probability of stopping to give the total average intersection delay to all vehicles. Equivalent sets of data points are also calculated assuming that the green time is forty percent and one hundred and sixty percent of the mean value. A best-fit curve of the form

$$DELAY = A*(VC^{D} + VC)^{B} + C \qquad (31)$$

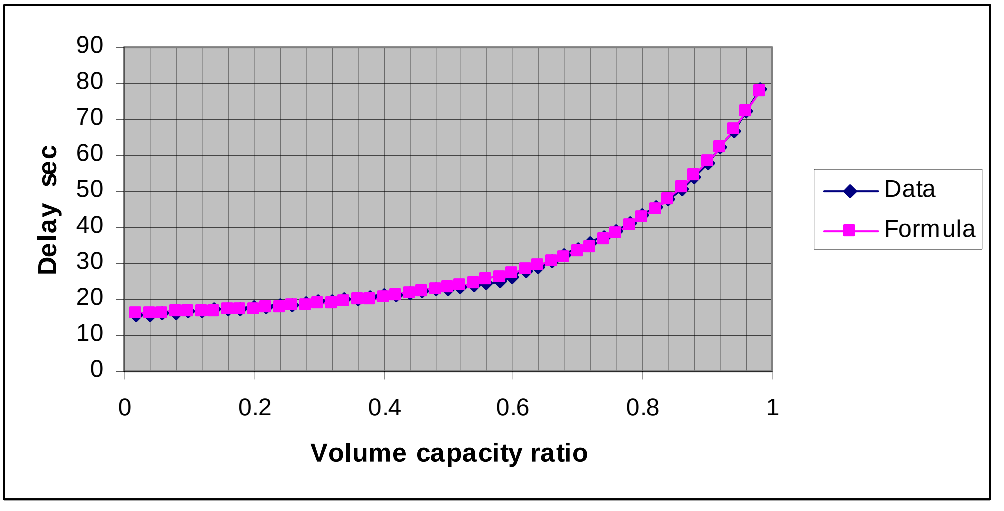
is fitted to the primary data output where

VC = volume capacity ratio

A, B, C and D are constants.

A normal power curve does not turn upwards sharply enough as the volume / capacity ratio rises above 0.7, while an exponential curve rises too sharply. It was found that a curve of the above form fitted the data best. Curves are derived for the maximum number of phases for each approach speed limit. A typical fit of formula to data is shown in Figure 5.1. A quadratic formula to predict the probability of stopping is fitted to the stop data. The alternate sets of data are used to derive an approximate correction formula to convert the average delay to all vehicles approaching the intersection into the average delay by approach by assuming that the green times will be allocated to give equal volume capacity ratios by approach. It was found that the magnitude of the adjustment for unequal shares of the green time depends on the volume capacity ratio of the intersection, as well as on the relative shares of the green time. The application of the adjustment is described in Appendix 3.3.

Figure 5.1 Average delay to all vehicles at signals on an 80 km/h road.



Delay equations for different values of the maximum number of phases were compared. It was found that the delays were almost linear with the maximum number of phases (see Figure 3.4). This made it possible to provide only one set of delay equations for the maximum number of phases and the assignment program could then calculate delays for a reduced number of phases by proportionality.

5.5 Delays at other intersection types

At freeway merges, traffic can enter continuously from both approach links, with the limiting capacity being the exit link. At low flows vehicles do not have to slow down, but as the flows increase, there is some loss of speed. This was considered to be equivalent to a one phase set of traffic signals with a zero constant term so this was adopted. Some years after implementing the approach, it was confirmed when Murdoch University researchers had access to an instrumented car that accurately measured time, distance and fuel consumption. They measured the delays in a merge situation over a range of traffic flows and compared the delays to the model results. These are reported in Lyons et al (1984) (see Figure 3.5).

For traffic on a priority road approaching a sign controlled intersection, through traffic in the median lane may or may not be delayed by vehicles waiting to turn right off the priority road, depending on the absence or presence of a separate turning lane. As this is one of the options available to road designers, it is desirable that the assignment process be influenced by the presence or absence of separate right turning lanes on the priority road. At low flows there is little or no delay, but this increases as the flows increase towards capacity. These delays are modelled by considering the traffic on the priority road only to be passing through a one phase set of traffic signals with no delay at zero or minimum flow, i.e. the constant is set to zero.

Delays to minor road vehicles facing a stop or give-way sign are estimated using Tanner’s (1962) formula for one lane crossing one lane of traffic. This formula can be differentiated and is

$$Time = \alpha * Min * \exp(\beta * Majf) + C \qquad (32)$$

Where

Time = Average delay per vehicle

Min = flow per lane on minor approach

Majf = flow per lane on priority road

α, β and C are coefficients

No suitable formulae were found for traffic crossing multiple lanes. The problem was solved by using the percentages of traffic travelling in each direction at peak periods and the percentages of traffic in each lane of a multi-lane road to derive divisors to convert the total two-way flow on the priority road into the flow on the busiest lane. Similarly for multiple lanes on the minor road approach, the approach flow was converted to the flow on one lane. Capacity is taken to be that combination of flows that gives an average delay of 100 secs on the minor road. At the time of development, this was about the point at which the general public started to complain about delays.

5.6 Derivation of hourly flow histograms

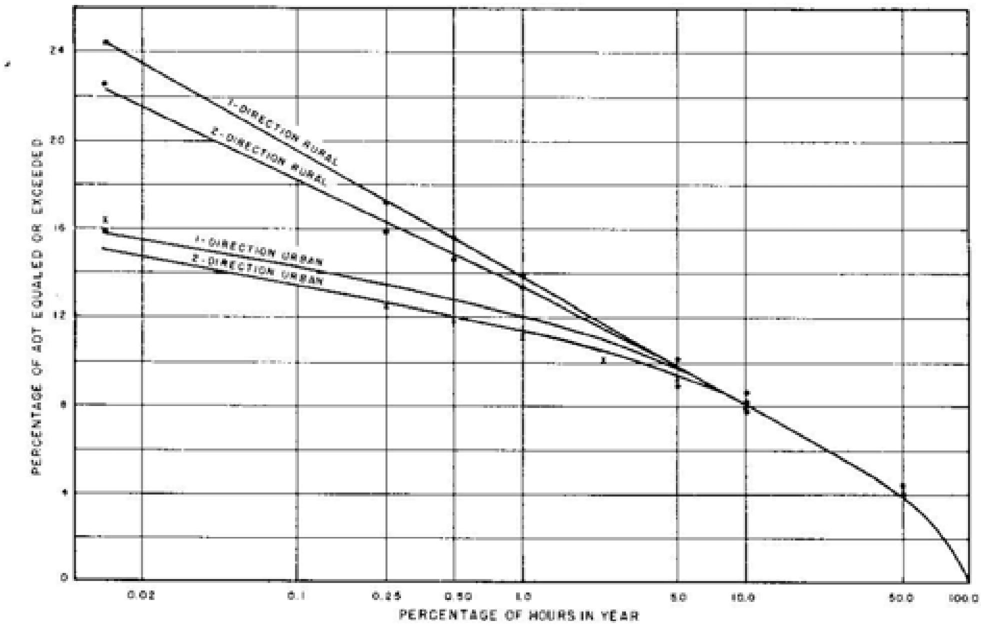
To convert the hourly speed flow curves that have been derived into daily speed flow curves requires a histogram of the number of hours per year with hourly flow rates at each percentage of the daily flow. During analysis, all two-way roads are treated as two independent oneway links. During peak periods, the total flow in one direction is usually more than in the reverse direction. However, at intersections (say four way), two of the approaches will be carrying traffic in the peak direction and two will be carrying traffic in the off peak direction. This means that the peak hour flow as a percentage of daily traffic entering intersections is lower than the peak hour percentage on the links.

In consequence, two histograms of the number of hours in the year with hourly flow rates as percentages of the daily flow are required. These histograms must meet two requirements:

- the total number of hours must add to 8760 +/- a tolerance (= number of hours in a year)
- the product of hours by percentage of the daily flow must add to 36500 +/- a tolerance (= 365 days x 100%).

The only data available was a graph of the number of hours in a year with the hourly flow rate equal to or exceeding percentages of the daily flow in the U.S. Highway Capacity Manual (1965) (see Figure 5.1).

Figure 5.1 Cumulative hours in the year with flows equal or exceeding given percentages of daily flows

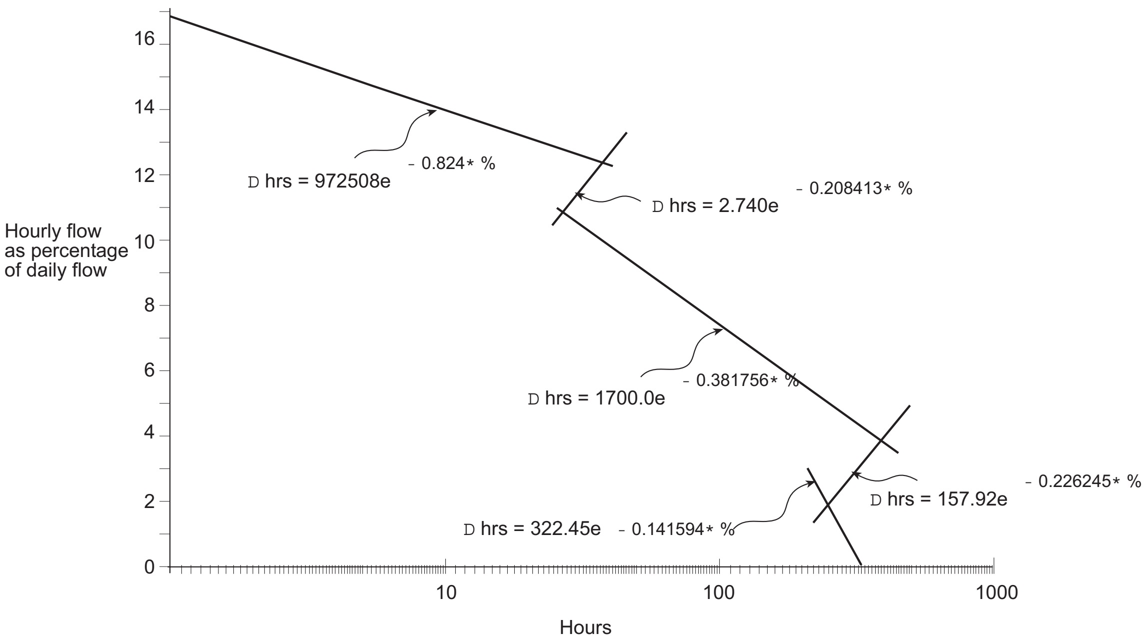


Percentage of ADT recorded during all hours of the year on 113 selected urban and rural roads, 1959-1960. (Source: BPR).

Source: U. S. Highway Capacity Manual 1965

This had to be converted into the number of hours in each flow increment of 0.25% of the daily flow, at the same time satisfying stringent criteria for the total hours and the product of hours and flow rate. The first step was to read off data points from the graph to convert cumulative values into values in the discreet intervals. On plotting these incremental values on log-linear paper it was found that the graph could be very closely approximated with five straight lines (see Figure 5.2).

Figure 5.2 The number of hours in the year where the hourly flow rate falls in each 0.25% increment of daily flow.



Source: Derived from Figure 5.1

The histogram represented by Figure 5.2 approximates to the demand flow profile for the one way flow of traffic along roads in moderate sized cities. An alternate histogram of hourly flow for the total volume entering intersections is generated by reducing the number of hours at high flows (peak period) and very low flows (late night, early morning), and increasing the number of hours in the middle range representing flows in the middle of the day. The method of processing in the system expects the histogram to be a demand profile which it then modifies when hourly demand exceeds hourly capacity. One needs to remember that measured profiles in large cities will already be modified by the capacity of the system because the outflow rate from bottlenecks is less than the inflow rate of traffic. The system has since been modified to reduce the flow increment to 0.05% by dividing the appropriate coefficients by five and making some slight adjustment so that the basic equations give the correct total hours and total flow.

5.7 Derivation of daily speed flow curves

For daily assignments, the system requires daily speed flow curves for each capacity indicator for cruising between intersections and for the delay time to negotiate intersections. These are constructed from hourly speed flow data for each of five vehicle types, hourly and daily capacities, traffic composition and passenger car unit equivalents for twelve vehicle types, and a histogram of hourly flow rates as a percentage of the daily flow. Hourly capacities are assumed to be in equivalent passenger car units, and the input values should be the best estimate of the true capacities. Daily capacities are assumed to be in vehicles and are the figures that are coded in the networks. As the daily capacity is used as both a multiplier and a divider at different stages in the process, the coded capacity does not have to be the true capacity, so that any figure reasonably close, that is convenient for network coding, will do. This makes it convenient to use the same per lane daily capacity for different road types (divided, undivided) for network coding. If at a later stage, a road changes from say four lanes undivided to four lanes divided, only the capacity indicator will need to be changed, thus reducing the amount of work and the risk of coding errors.

The histograms supplied have all the flows arranged in high to low order. The profile of traffic throughout the day (say starting around 4.00 am) rises to the morning peak, falls to about half through the middle of the day, rises again for the evening peak, then falls away to very low flows around 4.00 am. Mostly the two peak flows are about the same but in some cases, one peak flow is larger than the other. In some cities, there can even be a smaller third peak around midday. When daily flows are sufficiently large for hourly demand to exceed capacity, the flow in excess of capacity is stored and added to the next time period. This means that in order to ensure that no vehicles are left in queues at the end of processing, the last time through the histogram must be in high to low order of hourly flow rate. If the histogram is processed once only, the carryover of vehicles will take longer to clear when demand falls below capacity. To account for a single rise and fall to one peak, the histogram needs to be split into two. For one peak larger than the other, the histogram is split into three. For equal peaks, it is split into four. In the case of a smaller midday peak, it should be split into five. The user nominates the number of intervals that the histogram should be split into. The program ensures that the last pass is always with hourly flows in high to low order.

The process of building a set of data points of average vehicle delay at different daily volume to capacity ratios is as follows.

- The program reads in mid block and intersection data for one capacity indicator. After editing, the values are reported.
- Using the traffic composition and the passenger car unit equivalents, the hourly capacities are converted to vehicle capacities.
- A daily volume to capacity ratio of 0.025 is selected and the daily flow calculated.
- For each entry in the histogram, calculate the hourly flow and the hourly volume to capacity ratio.
- For each vehicle type, calculate its travel time (to travel one kilometre if mid block, or to negotiate the intersection) and add to the total.
- Calculate the average time per vehicle and save.
- Increment the daily volume capacity ratio by 0.025 and repeat.

This process continues until the total hours in the day in which queues exist reaches a user specified value. (A workable value is somewhere in the range one to two hours per day).

When hourly demand exceeds hourly capacity, the number of vehicles equal to capacity is serviced; the remainder are stored and added to the next time slice. The number of hours in the time slice is added to the total time that queues exist. The total vehicle time spent waiting is calculated and a proportion of the waiting time is added to the total daily time.

In peak period travel, time spent waiting in a queue at a congestion point modifies the histogram of demand at the next congestion point, and so on until the destination is reached. This modification of the demand histogram reduces the total queue delay that will occur at the next point. In the evaluation process, the assumption is that the histogram is unmodified; that the vehicles are getting through when they wish to. Counting the total queue delay for each congestion point will result in a degree of multiple counting. To counter this, the queue delay is spread over a user nominated number of congestion points. Also the user can nominate what proportion of vehicles are assumed to change their time of travel to avoid congestion and the proportion of counted delays to be allocated to intersections with the remainder being allocated to mid block travel (to give some increase in delays on freeways when demand exceeds capacity).

A curve of the form

$$TIME = A * VC^{B} + C \qquad (3)$$

is fitted to the output data.

Where A, B and C are coefficients and

VC is the volume to capacity ratio.

This formula is for the average all day time of all vehicles. For assignment of traffic to a network, the speed of the marginal vehicle being added to the system is required. This is obtained by multiplying the above formula by VC to give the total all day delay for all vehicles, then differentiating with respect to VC to give a formula for the time of the marginal vehicle (see Appendix 3.1). The value of the coefficients A, B and C are reported, then the program looks for another set of data. These coefficients are input as parameters to the assignment program.

5.8 Peak period speed flow curves

The same program can also be used to prepare peak period speed flow curves. In this case the user nominates the length of the peak period in hours. As the peak period has to be longer than the period over which queues exist, the program checks that the nominated number of hours for queues to exist is less than the length of the peak period and if necessary resets it. Starting with the highest hourly flow rates, the program will sum the number of hours until the length of the peak is reached. The number of hours at each percentage increment for the remainder of the day is then set to zero. The program then proceeds as for the daily speed flow curves to calculate the total vehicle time in the peak period and hence the average time per vehicle and hence the peak period speed flow curve.

5.9 Summary

The estimation of travel time on a link is divided into two stages:

- the time to travel the full distance between intersections at cruising speed
- the time spent stationary at intersections plus the time to decelerate to a stop and accelerate back to cruising speed less the time to travel the same distance at cruising speed.

Delays at traffic signals have been derived from the detailed methods of analysis used for intersection design and then expressed in terms of the volume to capacity ratio. In a forecast situation, this avoids having to supply signal design data which is still unknown at that stage with very little loss of accuracy. The conversion of hourly travel time functions into daily travel time functions requires two histograms of the number of hours in a year with hourly flow rates as percentages of the daily flow. The derivation of these histograms is described together with the process of constructing the daily travel time functions and peak period travel time functions.

Chapter 6 describes the estimation of vehicle operating costs from its components of fuel and oil consumption, tyre wear, depreciation and repair and maintenance as a function of vehicle speed and the conversion of this data to a function of the daily volume to capacity ratio.

CHAPTER 6

DERIVATION OF ROAD USER COST DATA

6.1 Introduction

The cost per kilometre of owning and operating a vehicle varies with changes in many variables including the need to stop at intersections. For travel between intersections and for stopping at intersections, road user costs are built up in three stages. The first stage applies prices to estimates of consumption by components to construct a set of data points at eight km/h intervals (the metric equivalent of the old five mph intervals). The second stage fits a polynomial in speed to the data points. The third stage uses the cost per vehicle kilometre as a function of speed, the traffic composition and the distribution of the flow rate throughout the day to estimate the average vehicle cost per kilometre (and cost per vehicle entering) as a function of the daily volume to capacity ratio for each speed flow curve number used in the network. The output file created is the one used in the final program in the set to perform the project economic evaluation. These three stages only have to be performed each time the unit prices change. It is the final stage that is used continuously.

6.2 General

Road user costs are the sum of the costs of owning and operating a vehicle, the time costs of people and freight in the vehicle and the costs of vehicle crashes. The proportion of crashes in each of the severity categories of fatal, injury and property damage varies with the prevailing speed of traffic. The incidence of crashes in terms of vehicle kilometres of travel varies with the type of road (controlled access, divided road and undivided road) and with the type of development along the road frontage. The number of crashes at intersections per vehicle entering varies with the type of control at the intersection and with the degree of saturation of demand over capacity. It is possible that both of these rates of crashes vary with the prevailing speed but as yet the author has not seen any reports of analysis of crash data along these lines.

Vehicle operating costs are estimated by applying prices to estimates of consumption of the individual components that make up the cost of operating a vehicle. The individual components are fuel, oil, tyre wear, repair and maintenance and depreciation. Factors that influence vehicle operating costs are the speed of travel, the roughness of the road surface, the road gradient and degree of horizontal road curvature, and extent of vehicle utilization per annum. In addition, in urban driving conditions, the need to stop at intersections adds additional costs for time, fuel consumption, additional wear and tear on brakes, the engine and transmission system and additional tyre wear.

Bowyer et al (1986) have described a four level set of models for estimating fuel consumption. These are:

- an ‘instantaneous’ model
- an ‘elemental’ model
- a ‘running-speed’ model
- an ‘average travel-speed’ model.

These four model types form a set of interrelated modelling steps traversing from a ‘highly disaggregated’ to a ‘highly aggregated’ level. In this process a ‘coarser’ model is derived from a more detailed model. For example, the elemental model is derived from aggregating the instantaneous model.

The **instantaneous model** provides a second by second estimation of fuel consumption as a vehicle traverses a road. The road data information required is extensive, being a full longitudinal profile including the road roughness plus any constraints on speed such as intersections, speed limit, road curvature and the presence of other traffic. This model is the most detailed of the set and is also the most accurate.

The **elemental model** splits driving into the three (or four) primary components, cruising at relatively constant speed, idling, and the speed change component representing deceleration to stop and accelerating back to cruising speed. Road data requirements are less demanding, being restricted to those items that determine average cruising speed, the presence of controlled intersections and the time spent idling. The loss of accuracy over the instantaneous model is about +/- 1 percent but there is a huge saving of computer time.

The **running speed** model combines the cruising and speed change components into one to give running speed and idle time. Running speed varies with the cruising speed, the spacing of controlled intersections and with the type of control at the intersection. Fuel consumption is estimated as a function of the running speed plus a separate estimate for the time spent idling.

The **average travel speed** model estimates the average speed from the total journey time and total journey distance with fuel consumption again being a function of the speed. As there are many permutations of combining cruising speed with the number of stops for the same average travel speed, the loss of accuracy of these latter two models is considerable. The economic evaluation model being described here is a three stage elemental model.

6.3 Vehicle operating cost model for the cruising component

For travel between intersections, a model for predicting vehicle operating costs was already available from work done on contract to the Commonwealth Bureau of Roads. The model is described in Commonwealth Bureau of Roads (1972). To estimate fuel consumption, the model uses a basic equation to estimate consumption of a petrol engine on a flat, straight, smooth sealed road. Adjustments for gradient, road curvature and road roughness are by means of percentage additions to the basic fuel consumption, using factors from look-up tables that are vehicle and speed dependent. For diesel powered vehicles, the basic equation is factored by a diesel efficiency factor, which again is by a look-up table that depends on speed, gradient and vehicle type. The equation is of the following form

$$\begin{matrix}\text{Fuel}\\\text{Consumption}\\\text{(litres/1000km)}\end{matrix} = \begin{matrix}\text{Basic}\\\text{Fuel/Speed}\\\text{Relationship}\end{matrix} \times \left[1 + \begin{matrix}\text{Engine}\\\text{Efficiency}\\\text{Adjustment}\end{matrix} + \begin{matrix}\text{Gradient}\\\text{Adjustment}\end{matrix} + \begin{matrix}\text{Curvature}\\\text{Adjustment}\end{matrix} + \begin{matrix}\text{Road}\\\text{Roughness}\\\text{Adjustment}\end{matrix} + \begin{matrix}\text{Traffic}\\\text{Congestion}\\\text{Adjustment}\end{matrix}\right] \tag{33}$$

Apart from an ill defined road train, the heaviest vehicle in the original model is a five axle articulated vehicle with a gross vehicle mass of 38 tonnes. Modern day vehicles now include up to a seven axle articulated vehicle with a gross vehicle mass of 46.5 tonnes plus any number of combination vehicles with gross combination masses from 60 tonnes for a B double up to 143.5 tonnes for a triple road train. Extension of the model to include these heavier vehicle types raised two issues; how to estimate coefficients for the basic equation, and how to extend the look-up tables for fuel consumption on grades and the corresponding diesel efficiency factors.

In the original model, the approach taken was to derive a relationship between fuel consumption and average gross vehicle mass for vehicles in the range from two axle rigid trucks to five axle articulated vehicles. Fuel consumption for the heavier vehicles was then calculated as the fuel consumption for the five axle articulated vehicle, corrected for the increased vehicle mass.

In the mid eighties, different vehicles were fitted with equipment to measure and record instantaneous fuel consumption and speed. From these measurements, a computer model (ARFCOM) was set up to estimate fuel consumption second by second for a vehicle with nominated characteristics when traversing a supplied road profile. This model was used in recent work done by ARRB Transport Research for Austroads to provide new values of the coefficients for the base fuel consumption using the formulae

$$\text{Fuel Consumption (vehicle type)} = A\,(B + C / \text{Speed} + D \times \text{Speed}^2)\ \text{(litres / 1000 km)} \tag{34}$$

Where

A = Factor to allow for average vehicle tune

B, C and D are coefficients

e.g. for a small car at 60 km/h the fuel consumption is

1.071 (30.9 + 1236.2 / 60.0 + 0.0051 x 60.0 x 60.0) = 74.82 litres per 1000 km

These coefficients are listed in Table 6.1. (Thoresen 2000). Values for three axle vehicles and above are for diesel engines, while the values for the lighter vehicles are for petrol engines. New values were also provided for the grade adjustment factor by vehicle type. These values are listed in Table 6.2.

Close examination of the coefficient D in Table 6.1 shows no consistency across vehicle types. This term is basically overcoming wind resistance, so one would expect the term to rise as the frontal area of the vehicle increased. Similarly, there is a problem with the numbers in Figure 6.2. Comparison of the grade correction factor for a large bus with a triple road train shows that the bus is using nearly five times the amount of fuel per gross tonne to climb gradients.

Table 6.1 Basic speed fuel consumption equations

(fuel consumption = litres per 1000 km)

Vehicle Stereotype	Fuel Type P = petrol D = Diesel	State of Tune factor A	Equation Coefficients		
			B	C	D
Passenger Vehicles					
Small Car	P	1.071	30.9	1236.2	0.0051
Medium Car	P	1.071	36.8	1453.2	0.0053
Large Car	P	1.071	44.2	1889.1	0.0057
Large Bus	D	1.1	69.4	5451.1	0.0131
Freight Vehicles – Rigid & Articulated Trucks					
Utility/light commercial (2 axle 4 tyre)	P	1.071	59.9	1915.3	0.0087
Light Truck (2 axle 6 tyre)	P	1.071	42.1	2596.7	0.0234
Light Truck (2 axle 6 tyre)	D	1.1	42.0	1948.0	0.0143
Medium Truck (2 axle 6 tyre)	D	1.1	43.3	3543.3	0.0159
Rigid or Articulated 3 Axle Truck	D	1.1	65.1	5408.3	0.0168
Articulated truck - 4 Axle	D	1.1	106.5	6779.7	0.0169
Articulated Truck - 5 Axle	D	1.1	118.1	10126.1	0.0158
Articulated Truck - 6 Axle	D	1.1	131.1	11957.5	0.0148
Combination Freight Vehicles					
Rigid (3 axle) + 5 Axle Dog Trailer	D	1.1	129.11	15209.82	0.018
Twin steer + 4 Axle Dog Trailer	D	1.1	132.20	17012.87	0.018
Twin steer + 5 Axle Dog Trailer	D	1.1	140.97	18085.63	0.019
B Double Combination	D	1.1	172.7	14720.4	0.0160
Road Train (double)	D	1.1	223.6	17201.8	0.0148
A B Combination	D	1.1	254.94	23765.82	0.017
B Triple Combination	D	1.1	235.82	20512.58	0.018
Road Train (triple)	D	1.1	312.1	26646.9	0.0150
Double B Double Combination	D	1.1	282.40	28144.99	0.017

Source Thoresen, T (2000)

Table 6.2 Harmonised fuel consumption gradient correction factors

NIMPAC Vehicle Stereotypes

Correction Factor by Grade and Speed Category

Vehicle Category	Gradient Category	Speed (km/h) 8	16	24	32	40	48	56	64	72	80	88	96	104
Small Car	4%	0.03	0.07	0.07	0.07	0.08	0.09	0.09	0.10	0.09	0.06	0.05	0.04	0.03
	6%	0.04	0.11	0.10	0.11	0.13	0.15	0.17	0.20	0.17	0.14	0.13	0.12	0.09
	8 %	0.05	0.18	0.16	0.16	0.18	0.21	0.24	0.29	0.26	0.23	0.20	0.18	0.14
	10%	0.06	0.28	0.27	0.28	0.30	0.34	0.39	0.44	0.40	0.33	0.30	0.26	0.21
Medium Car	4%	0.03	0.07	0.07	0.07	0.08	0.09	0.10	0.11	0.10	0.07	0.05	0.04	0.04
	6%	0.04	0.11	0.10	0.12	0.13	0.15	0.18	0.21	0.19	0.16	0.15	0.14	0.12
	8 %	0.05	0.20	0.18	0.18	0.19	0.21	0.26	0.31	0.29	0.26	0.23	0.21	0.17
	10%	0.06	0.29	0.28	0.29	0.32	0.36	0.43	0.49	0.46	0.39	0.33	0.31	0.26
Large Car	4%	0.02	0.06	0.06	0.06	0.07	0.08	0.09	0.09	0.04	0.03	0.03	0.03	0.02
	6%	0.04	0.10	0.09	0.10	0.11	0.13	0.16	0.16	0.11	0.05	0.05	0.04	0.04
	8 %	0.05	0.19	0.16	0.16	0.16	0.20	0.27	0.29	0.19	0.13	0.10	0.07	0.06
	10%	0.06	0.28	0.26	0.26	0.29	0.34	0.43	0.49	0.40	0.30	0.21	0.18	0.16
Large Bus	4%	0.08	0.11	0.10	0.13	0.20	0.26	0.39	0.52	0.42	0.29	0.19	0.10	0.00
	6%	0.15	0.24	0.32	0.42	0.54	0.65	0.83	0.98	0.84	0.70	0.57	0.45	0.32
	8 %	0.26	0.50	0.62	0.76	0.91	1.05	1.25	1.42	1.25	1.08	0.92	0.78	0.62
	10%	0.39	0.76	0.93	1.11	1.28	1.45	1.69	1.90	1.69	1.49	1.31	1.13	0.95
Utility / Light Commercial (2 axle 4 tyre)	4%	0.04	0.11	0.10	0.10	0.11	0.13	0.14	0.16	0.13	0.10	0.09	0.05	0.02
	6%	0.06	0.20	0.16	0.17	0.19	0.21	0.24	0.28	0.24	0.20	0.16	0.11	0.07
	8 %	0.08	0.34	0.32	0.33	0.35	0.39	0.44	0.49	0.44	0.37	0.27	0.21	0.15
	10%	0.10	0.50	0.47	0.50	0.54	0.60	0.66	0.72	0.65	0.56	0.45	0.32	0.18
Light truck (petrol) 2 axle 6 tyre	4%	0.02	0.04	0.04	0.03	0.02	0.01	0.01	0.01	0.01	0.02	0.02	0.01	0.02
	6%	0.03	0.07	0.07	0.06	0.06	0.04	0.03	0.03	0.03	0.03	0.03	0.03	0.03
	8 %	0.06	0.13	0.15	0.16	0.16	0.13	0.06	0.06	0.05	0.05	0.05	0.05	0.05
	10%	0.09	0.23	0.27	0.30	0.30	0.27	0.22	0.22	0.22	0.22	0.22	0.22	0.22
Light truck (diesel) 2 axle 6 tyre	4%	0.04	0.08	0.07	0.06	0.05	0.07	0.12	0.08	0.06	0.05	0.05	0.05	0.05
	6%	0.07	0.13	0.15	0.18	0.20	0.26	0.34	0.24	0.20	0.20	0.20	0.20	0.20
	8 %	0.12	0.31	0.36	0.41	0.44	0.51	0.61	0.45	0.45	0.45	0.45	0.45	0.45
	10%	0.21	0.50	0.58	0.64	0.67	0.77	0.86	0.86	0.86	0.86	0.86	0.86	0.86
Medium Truck (2 axle 6 tyre)	4%	0.06	0.10	0.09	0.09	0.11	0.19	0.32	0.24	0.13	0.13	0.13	0.13	0.13
	6%	0.12	0.21	0.27	0.33	0.40	0.52	0.69	0.64	0.64	0.64	0.64	0.64	0.64
	8 %	0.23	0.45	0.55	0.64	0.73	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87
	10%	0.34	0.70	0.83	0.95	1.05	1.12	1.12	1.12	1.12	1.12	1.12	1.12	1.12
Rigid or Articulated Truck (3 axle)	4%	0.09	0.12	0.11	0.13	0.20	0.28	0.43	0.46	0.42	0.42	0.42	0.42	0.42
	6%	0.16	0.26	0.33	0.44	0.55	0.69	0.76	0.76	0.76	0.76	0.76	0.76	0.76
	8 %	0.29	0.53	0.65	0.79	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
	10%	0.44	0.82	0.98	1.15	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23
Articulated Truck (4 axle)	4%	0.12	0.14	0.13	0.16	0.24	0.32	0.44	0.44	0.44	0.44	0.44	0.44	0.44
	6%	0.20	0.28	0.38	0.50	0.61	0.67	0.67	0.67	0.67	0.67	0.67	0.67	0.67
	8 %	0.36	0.59	0.73	0.86	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
	10%	0.56	0.90	1.06	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14	1.14
Articulated Truck (5 axle)	4%	0.01	0.14	0.13	0.19	0.28	0.38	0.47	0.47	0.47	0.47	0.47	0.47	0.47
	6%	0.25	0.29	0.40	0.53	0.66	0.73	0.73	0.73	0.73	0.73	0.73	0.73	0.73
	8 %	0.45	0.60	0.75	0.89	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
	10%	0.60	0.90	1.08	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17
Articulated Truck (6 axle)	4%	0.06	0.14	0.14	0.21	0.31	0.42	0.48	0.48	0.48	0.48	0.48	0.48	0.48
	6%	0.09	0.29	0.41	0.54	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
	8 %	0.17	0.61	0.76	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
	10%	0.25	0.91	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
Rigid Truck (3 axle) + 5 Axle Dog Trailer	4%	0.05	0.20	0.24	0.28	0.34	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41
	6%	0.10	0.30	0.41	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
	8 %	0.19	0.61	0.75	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
	10%	0.27	0.92	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
Twin steer Truck+ 4 Axle Dog Trailer	4%	0.05	0.20	0.25	0.29	0.35	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
	6%	0.10	0.32	0.43	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57
	8 %	0.20	0.64	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79
	10%	0.29	0.97	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17
Twin steer Truck + 5 Axle Dog Trailer	4%	0.05	0.20	0.25	0.29	0.35	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44
	6%	0.10	0.32	0.43	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57	0.57
	8 %	0.19	0.64	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78
	10%	0.29	0.97	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17	1.17

Table 6.2 (Continued)

Vehicle Category	Gradient Category	Speed (km/h) 8	16	24	32	40	48	56	64	72	80	88	96	104
B-double	4%	0.06	0.15	0.15	0.22	0.31	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43
	6%	0.10	0.30	0.42	0.54	0.63	0.63	0.63	0.63	0.63	0.63	0.63	0.63	0.63
	8 %	0.18	0.62	0.76	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
	10%	0.27	0.93	1.12	1.12	1.12	1.12	1.12	1.12	1.12	1.12	1.12	1.12	1.12
Road Train (double)	4%	0.07	0.16	0.15	0.19	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29
	6%	0.11	0.29	0.39	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	8 %	0.21	0.61	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
	10%	0.30	0.91	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
A B Combination	4%	0.06	0.21	0.25	0.28	0.32	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39
	6%	0.13	0.30	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
	8 %	0.24	0.62	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79	0.79
	10%	0.35	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
B Triple Combination	4%	0.03	0.20	0.24	0.28	0.32	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37
	6%	0.11	0.28	0.37	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52
	8 %	0.21	0.60	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74
	10%	0.31	0.89	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
Road Train) (triple)	4%	0.16	0.17	0.13	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
	6%	0.39	0.29	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34
	8 %	0.60	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
	10%	0.75	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Double B-Double Combination	4%	0.16	0.21	0.25	0.27	0.32	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
	6%	0.40	0.30	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41
	8 %	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62
	10%	0.78	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96

Source Thoresen, T (2000)

TRAMS and the system used to evaluate rural road projects (WARES) used vehicle types that were not included in these tables. This raised problems in two areas, the first being what coefficients to use for the basic fuel consumption equation and the second being what values to use for the grade adjustment factors. There is a third problem that will arise the next time that the allowable axle masses are raised at some future date in that the fuel consumption coefficients and grade correction factors will need to be changed. Given these problems and that there was some inconsistency across vehicle types in the data provided, further analysis of the data was performed.

Equation 34 was further disaggregated as follows

$$\begin{aligned}\text{Fuel Consumption (vehicle type)} &= \text{a} * \text{function(MASS)} + \text{b} * \text{POWER / Speed} \\ \text{(litres / 1000 km)} &\qquad + c\ * \text{FRONTAL AREA} * \text{DRAG x Speed}^2\end{aligned} \tag{35}$$

For three axle trucks and above the basic fuel consumption at speed increments of 10 km per hr was estimated using the supplied coefficients. The complete data set was then analysed as a single set to obtain values for a, b and c in the above equation. It was found necessary to raise MASS to the power of 1.04 to obtain the best fit. A possible reason for this is that the power to weight ratio is decreasing as the mass increases. For these vehicles, the engine will operate closer to or at full power for a higher proportion of time than for the lighter vehicles. The efficiency of the engine in converting fuel to energy decreases towards full power but these vehicles are also likely to have more gears than lighter vehicles, complicating the situation.

For the grade adjustment factors, for each of the single unit vehicles, the grade excess fuel in litres per 1000 km was calculated at each speed and grade. When this amount was divided by the vehicle mass, at each value of gradient, the values were almost constant across the complete speed range and across the vehicle types for all vehicles up to about 50 gross tonnes. At higher values of mass, the value dropped away considerably with increasing mass to the extent that the heaviest vehicle was using about 20% of the fuel per tonne of mass to move the vehicle on grades compared to a three axle large bus suggesting that there has been a problem in setting up the table. As these vehicles were not part of the fleet at the time of the fuel consumption measurements were taken that were used to set up ARFCOM, it is possible that it has trouble estimating consumption for very large vehicles. For each grade, the values for the single unit vehicles were averaged. The grade correction in the general formula was then made part of the basic equation as follows

$$\begin{aligned}\underset{(\text{litres}/1000\text{ km})}{\text{Fuel Consumption (vehicle type)}} &= a * \text{function(MASS)} + b * \text{POWER} / \text{Speed} \\ &\quad + c * \text{FRONTAL AREA} * \text{DRAG x Speed}^2 \\ &\quad + \text{coeff(grade)} * \text{MASS} \qquad (36)\end{aligned}$$

For all vehicles of three axles and above the program uses the basic coefficients with the vehicle characteristics to estimate the fuel consumption. Not only does this provide for vehicle types not in the basic list but also has the advantage that when the axle masses are next increased, the values of average gross vehicle mass can be adjusted and this will then automatically flow through into the fuel calculations. For the light vehicles, the supplied coefficients were used. The equation for these vehicles assumes a petrol engine and the vehicles do not load up to the full legal axle mass limit.

6.4 Source of intersection data

A literature search provided one source of information and data for the effect of decelerating to a stop and then accelerating back to the cruising speed. This was Claffey (1971). This data was limited to only four vehicle types and the data was expressed in cents per stop. The data was already about nine years old and in a foreign currency (American) so updating and conversion to Australian costs would be extremely difficult. Cost data for cruising at constant speed in cents per km (American) was also available for the same vehicles for the same time period. Two transformations of the intersection cost data were made. The first transformation was to express the amount consumed of each component during the stop cycle in terms of travelling an equivalent length at the appropriate cruising speed so that any updating of the cruising speed model would automatically flow through to the intersection model. This provided a set of data points at selected fixed cruising speeds. A polynomial formula as a function of speed was then fitted to the data for fuel consumption for the four vehicle types to estimate the equivalent distance at cruising speed to equal the cost of stopping.

In order to derive data for additional vehicle types, each coefficient in the speed formulae was then derived as a function of vehicle characteristics of mass, frontal area and engine cubic capacity. A power curve in speed was fitted to tyre wear, oil consumption and repair and maintenance costs. Expressing fuel consumption as a function of vehicle characteristics made it possible to add additional vehicle types (eg lighter cars and heavier articulated vehicles).

In the recent work to update the model, no new test data was available. However, for the fuel component, the ARRB Transport Research model ARFCOM was used to estimate the excess fuel consumed during a stop-start cycle from a range of cruising speeds for most of the vehicle types in current use. This model also provided updated data for fuel consumption while idling. Using the same approach as when the model was first derived, for each speed, the consumption was expressed in terms of travelling an equivalent distance. For each vehicle type, a polynomial in speed was then fitted to the length data. Each speed coefficient was then derived in terms of vehicle characteristics. However engine power was substituted for engine capacity, as the power was known but not the capacity.

The formulae for estimating tyre wear in terms of mms of tread per 1000 km were updated to reflect the reduced wear of modern tyres. This has been brought about by improvements in tyre technology, the change from crossply tyres to radial tyres and the reduction in the level of overloading of tyres. There has also been a marked reduction in vehicle repair and maintenance costs, again largely from a reduction in the level of vehicle overloading but also due to improvements in vehicle design.

6.5 Converting unit prices into vehicle operating cost data

The user first decides on the vehicle types that he/she wishes to use. He/she then constructs a file containing prices for each of the vehicle consumables of fuel (petrol and diesel powered), oil, new tyres, retreaded tyres, purchase price of new vehicle, vehicle repair and maintenance, person private time cost per vehicle, commercial time cost per vehicle, and cost per hour of freight carried per vehicle, with a separate price for each vehicle for each item. Average values of road roughness, road curvature and road gradient are also provided.

The characteristics of fifteen vehicle types are contained within the program. Over time these may need to be changed. They consist of the number of tyres on the vehicle, the number of spares, the number of times that tyres are retreaded, the tread depth of new and retreaded tyres, the number of hours in the year that the vehicle is used, the average gross vehicle mass, the frontal area and engine power, the amount of oil consumed per 1000 km, and the idling fuel consumption.

From these inputs, two data sets are calculated, one containing the estimated cost per kilometre for each vehicle while cruising at set speeds in increments of eight km/hr up to 104 km/hr, the second containing the excess cost of stopping from set speeds for each vehicle type, also the cost of idling in cents/min for each vehicle. For each vehicle, a polynomial in speed is then fitted to each of these data sets. These formulae will then provide the operating cost of each vehicle type as a function of travel speed.

6.6 Deriving vehicle operating costs by volume capacity ratio

The next stage is to combine the vehicle operating cost data with traffic composition, road type, speed limit, hourly speed flow curves by primary vehicle type, hourly and daily lane capacities, and a percentage distribution of the hourly flow rate throughout the day to calculate the average cost per kilometre of travel as a function of the daily volume to capacity ratio. This is performed for each of the speed-flow curves used in the traffic forecasting process. Similarly the average intersection cost per vehicle is calculated as a function of the daily volume to capacity ratio. The cost of a crash per vehicle is calculated by combining the cost per crash with user supplied crash rates. The output is a road user cost file, which is used by the final program in the system to perform the economic analysis.

The procedure used to construct the all-day costs is the same as that used to calculate the all-day speed flow curves up to the point where the number of hours in the day when queues exist reaches a user specified value. The cost data is used as a series of data points with linear interpolation between the points. To cater for the process of estimating user costs for up to thirty years forward from the date of the opening of a project to traffic, with no additional increase in corridor road capacities to cater for the continual increase in traffic, it is necessary to extend the cost data in some way for increasing daily volume to capacity ratios up to a value of three, knowing that in reality, some of that increase in traffic will be carried on additional projects that will be added later.

The extended curve still needs to give some benefit if the project reduces the traffic flow as some of the reduction will result from the project under consideration. One extreme is to assume that the cost per vehicle does not increase for flows above the point where queues exist for about two hours per day. The other extreme is to continue using the same process until queues exist for the whole day. The first option will provide no benefits in some future year if the volume capacity ratio is reduced from say 1.6 to 1.4 while the second option will exaggerate the benefits because it assumes that all that future traffic is not being shared on other as yet unbuilt links in the corridor.

The assumption adopted is to assume that the excess traffic is being carried on as yet unbuilt projects and that the reduction in flows has to be shared between the existing link and the unbuilt projects. To illustrate, assume the final point calculated above was for a daily volume to capacity ratio of 1.1 and the volume being considered was at a volume to capacity ratio of 2.6. The assumption is that a reduction of x vehicles per day would be shared between the current project and other as yet unknown projects and the proportion for the current project in this case would be 1.1/2.6. The other 1.5/2.6 vehicles are not to derive any benefit from this project although the benefits are being calculated as a reduction in the road user costs of the total traffic flow. This is achieved by first calculating the marginal rate of change in the road user costs between the last two data points. The total value of this increment as the product of the per vehicle cost times the number of vehicles is held constant and the following increments calculated by dividing this total by the increasing volume of vehicles. The increment is then added to the previous data point. In this way potential over counting of benefits on links carrying exceptionally high volumes in the later years of the analysis is minimized.

6.7 Summary

The process of constructing a file of costs per vehicle kilometre by volume to capacity ratio by working up from primary costs of the individual components that add up to the cost of owning and operating a vehicle has been described. The functions and formulae that are used to estimate the quantities of each component consumed for each vehicle type are inbuilt in the system. Periodically, these functions will need to be updated as technology changes. Price changes can be worked through the system in an orderly and logical manner. The steps described so far are used to set up a file of vehicle operating cost data as a function of volume to capacity ratio for each of the road types and speed limits. The final step in the economic evaluation of urban road projects is described in the next chapter.

CHAPTER 7

ECONOMIC ANALYSIS

7.1 Introduction

The previous chapters have described the processes used to set up all the data necessary to perform an economic analysis. This chapter deals with the actual economic evaluation itself. There are a number of measures that can be used to measure economic worth. Each has advantages and disadvantages. It is also possible that all of the data customarily supplied is not available in the network files or that the area of influence of the project is not very big. The evaluation program has flexibility to deal with different levels of availability of data (two traffic forecasts or one forecast with growth rates and one or two networks) and the area of the network to be used in the analysis. In any extended mathematical calculation procedures, computation errors are introduced by the limited accuracy of storing numbers in computers and from rounding real numbers to integers. The program makes an estimate of the possible size of this error and expresses it as a range in the benefit cost ratio (BCR). Alternate BCRs are calculated to show the sensitivity of the analysis to discount rate and to delaying the project by two years.

7.2 General

Economic analysis is the process of comparing a time stream of benefits over a number of years to the time stream of the costs of creating the project. (Sugden and Williams 1978). A number of methods can be used, depending on the purpose of the analysis. The methods are:

- the net present value (NPV)
- the net present value per dollar of investment (NPV/$)
- the benefit cost ratio (BCR)

- the first year rate of return (FYRR)
- the internal rate of return (IRR)
- the pay back period (PBP).

In the **net present value** method, the time stream of benefits and costs are separately discounted to a common point of time and then summed over all the years of the analysis. The net present value is then the total discounted benefits minus the total discounted costs. A positive value indicates a project where the benefits exceed the costs. The magnitude of the net benefits depends on the project size, the discount rate, the number of years of analysis, the number of years of benefit, the economic worth of the project and also on the time reference point chosen for discounting.

The **net present value per dollar of investment** is the net present value calculated as above, divided by the capital cost to give a ratio. If the capital costs have been discounted, the result will be 1.0 less than the benefit cost ratio. A ratio greater than zero indicates a project where the benefits exceed the costs. The ratio depends on the number of years of benefit, the discount rate and the economic worth of the project. If the capital costs are not discounted, it also depends on the time reference point chosen for discounting the benefits.

In the **benefit cost ratio method**, the total discounted benefits are divided by the total discounted costs to give a ratio. This ratio depends only on the economic worth of the project, the discount rate and the number of years of benefit. It is independent of the time reference point chosen for discounting and independent of the size of the project. A ratio greater than 1.0 indicates that the benefits exceed the costs.

In the **first year rate of return**, the benefits in the first year are expressed as a percentage of the total discounted capital costs. The value depends on the economic worth of the project in the first year of benefits. A favourable project is when the ratio exceeds the discount rate. Sometimes, the **payback period** is calculated. This is the number of years of discounted benefits needed to pay back the capital cost.

The **internal rate of return** method finds the discount rate that will equalise the total discounted benefits with the total discounted costs. It depends on the number of years of benefit, the time reference point chosen for discounting and the economic worth of the project. A favourable project is when the internal rate of return exceeds the discount rate.

Any one of the methods can be used where the decision is simply "is this project worthwhile?" However, if the aim is to select projects from a list to make up a construction program, each method will produce a different works program, as each tends to favour one type of project over another. The net present value method favours large projects over small projects. The internal rate of return method and the first year rate of return method favour projects with a higher percentage of early benefits such as projects where the traffic growth rate is low (or even negative). The benefit cost ratio method and the net present value per dollar of investment method are the best for selecting projects to make up a program where the object is to maximise the total benefits over the whole program.

Recently, it has been recognised that where growth rates are high, it is possible to construct a project too early when it is selected based on benefit cost ratio alone. To reduce this possibility, it is also necessary to calculate the first year rate of return, and to delay the project until this becomes favourable. The system described here uses the benefit cost ratio method. It does not currently calculate the first year rate of return, but it is intended to add this to the system in the near future.

Historically, maintenance costs have been treated as negative benefits when evaluating capital projects. With the modern trend to calculate project life cycle costs which are a combination of capital and discounted maintenance costs, some people prefer to count discounted maintenance costs as part of the capital cost when doing economic evaluations. The difference in the benefit to cost ratio between the two methods is usually quite small. The system described here uses the original idea of counting increased maintenance costs as negative benefits.

The number of years of analysis is the total time from the beginning of capital expenditure to the last year for which benefits are calculated. The number of years of benefits is measured from the first year that the project generates benefits till the last year for which benefits are calculated. When comparing projects of vastly different sizes, the results of the comparisons will be different, depending on which one is kept constant. The number of years of benefit should not exceed the life of the project. Most analysts attempt to keep the number of years of benefit constant across all projects, but where projects have different lives, it is necessary to vary the number of years of benefit so that it is not greater than the project life.

All benefits and costs are discounted to the base year. The main issues involved in discounting and selection of the discount rate are presented in Marglin (1963), Price

(1993) and Stiglitz (1994). In simple terms there are three main reasons for discounting. One is that a dollar next year is worth less than a dollar now. People prefer to pay more to have something now rather than save up for it. This is also known as the social time preference. The second explanation is that it is the opportunity cost of capital after negating the effect of inflation. The third is that it is to allow for the uncertainty of future estimates. Whatever the explanation, Government Treasuries usually decide the rate to be used for projects funded by the Government.

7.3 Analysis program

The evaluation program reads the binary network files created by the traffic forecasting process, combined with project cost data, three discount rates, and times of forecast volumes relative to the date of opening the project to traffic. In addition, optionally the user can supply selected link data values that will override the values read from the network(s). The reference point for discounting is taken as the date of opening the project to traffic. The system is flexible in that it can read:

- two networks, each containing two estimates of future traffic volumes
- two networks, each containing one estimate of traffic volumes plus a growth rate which can apply to less than the number of years of analysis
- one network with two estimates of future traffic volumes plus records changing the base and/or project values on selected links
- one network with one estimate of traffic plus a growth rate plus records changing the base and /or project values on selected links
- The area evaluated can be the whole network or an area defined by listing all the contained nodes.

The normal usage is two networks, each containing two estimated future traffic volumes with the evaluation applying to the whole network. The only requirement between the two networks is that they must contain the same number of zones and that the assignment years must be the same. Each network usually contains traffic volumes for two points in time, one close to the date of opening the project to traffic and the second about ten years later (the interval is not a constraint). Optionally the two networks may only contain one forecast in which case a user supplied growth rate is uniformly applied to all links. The user also has the option of using the whole network, or a portion selected by specifying the node numbers to be included in the study area. In this case, intersection analysis is done on all nominated nodes and only those links that have both nodes in the study area. The flow chart for the program is illustrated in Figure 7.1.

Figure 7.1 Flow Chart For Economic Analysis Program

Read Parameters, edit and report
Read network(s)
Read unit cost file
Read project data

Select first node
Is it in study area
Copy data for all connecting links into work area
Estimate first year traffic and increment
Determine intersection type and capacity
For each analysis year
  Calculate intersection traffic
  Calculate road user costs and add to total
  For each approach link
    Is it in study area
    Calculate traffic
    Calculate road user costs and add to total
    Calculate increase in lane kms and add to total
End of loops

For each discount rate
  For each analysis year
    Calculate road user benefit
    Discount benefits and costs and add to totals
  End of loop
  Calculate Benefit Cost Ratio
  Delay project by two years and calculate BCR

For first discount rate only
  Calculate alternate road user benefits
  Discount benefits and total
  Calculate BCR
  Calculate error range

Report all values

7.4 Calculation of annual benefits

The process that is used is as follows. The program first reads the user parametric data, edits and reports. It then reads the networks into memory, followed by the unit road user cost file. These costs are then factored by a user supplied factor to convert the per vehicle unit costs into annual costs per daily vehicle.

The process of estimating the total road user costs in the area of interest for each year of analysis for the base and project cases now follows. Select the first node and check that it falls within the area. Copy the base and project information for each link approaching or leaving the node to a temporary work area. The rate of annual linear increment in traffic and the traffic in the first year of analysis are calculated from the assigned volumes for each link and saved. The type of intersection control is determined and the volumes entering and the increments entering determined. The total road user costs for the traffic entering the intersection are calculated for each year and added to the totals. For each approach link, the node at the opposite end is checked to see if it is in the study area. If yes, the mid-block road user costs for each year are calculated and added to the totals. The difference in lane kilometres of roadway between project and base is calculated and added to the total. The process is repeated until all nodes and links have been processed.

The increase in road maintenance costs for each year can be entered by the user or it can be calculated from the increased lane kilometres in the project network and parametric data of annual cost per lane kilometre, reseal costs and reseal interval. The increase in road maintenance costs is treated as a negative benefit. For each of three discount rates, the benefits are discounted back to the date of opening and the capital costs are discounted forwards to the date of opening. The benefit cost ratios for each discount rate are then calculated as the sum of the discounted benefits divided by the sum of the discounted capital costs.

7.5 Estimation of the Error Range

During the assignment of a trip matrix to a network, random arithmetic rounding errors occur causing a small change in the total road user cost. At the same stage of the assignment, trips for a particular origin – destination pair may be assigned to a different route in the project network compared to the route chosen in the base network. This gives slightly different link times in the later stages of the assignment, causing small unexpected changes in link volumes well away from the influence of the project. These unintentional changes will have some effect on the total road user costs over the whole network. The benefits of adding a project to an urban network usually result from a very small percentage change in a very large total of road user costs. As this total is subject to both arithmetic rounding errors and to errors in the magnitude of the link volumes, it is possible that these errors can be larger than the estimated benefits, particularly for the smaller projects.

To provide the user with some idea of the magnitude of the errors, numerical methods are employed to remind the user that the BCR is a best estimate within a possible range. Experimentation was done varying the starting zone in the zone loading order to find the change in the total road user costs. Other tests were done by reducing the size of the number of trips to be assigned between each application of capacity restraint. Based on these tests, a percentage value was chosen for the possible random error in the total road user costs.

A second possible source of error in the BCR is caused by the fact that the model is not 100% accurate in predicting the link volumes in the vicinity of the project. Assuming a linear relationship between errors in the corridor volumes in the vicinity of the project and the estimated benefits provides another estimate of potential errors in the BCR. These two error estimates are then combined to provide an estimate of the possible error in the benefits which is then expressed as a range in the BCR.

7.6 Calculation of alternate BCRs

The assumption of using up to thirty years of traffic growth on a network, which is not changing, can produce very high volume to capacity ratios, so high in some cases that it is obvious that the road would not be able to carry the volume. The issue of how to estimate road user benefits in these circumstances is contentious. One major Australian road study in the late sixties (Commonwealth Bureau of Roads 1973) assumed that there would be no increase in the unit road user costs once the flows exceeded hourly capacities. This had the effect of underestimating future benefits on the affected projects. How this has been treated in more recent studies has not been described in the published reports. This program assumes that only that portion of the traffic on a link that is realistic will achieve a benefit if the volume on a link is reduced.

An alternative method that circumvents this issue is to calculate the benefits in the first year, then express these as a benefit per vehicle kilometre of travel in the base network. Future year benefits are then estimated from the amount of travel. The program calculates an alternate BCR using this method for comparison with the reported results. The program also calculates alternate BCRs by assuming that the timing of the whole project is delayed two years including funding and the reference point for discounting.

This provides an indication of the sensitivity to the timing of the project as not every project is going to be inserted on the construction program in the year assumed for analysis. Table 7.1 is an example of the information that is reported for each project analysis.

Table 7.1 A set of results for a road extension project

BCR for a road Extension to Junction Road A

YEAR	CONST $pa	MAINT $pa	OTHER $pa	DNCOST $pa	DN VEH-KM pa	PRCOST $pa	PR VEH-KM pa	RUBFT $pa
-10	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-9	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-8	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-7	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-6	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-5	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-4	0.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-3	48333333.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-2	48333333.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
-1	48333333.	0.	0.	0.00000000D+00	0.00000000E+00	0.00000000D+00	0.00000000E+00	0.00000000E+00
1	0.	198534.	0.	0.12919059D+11	0.50967522E+08	0.12849771D+11	0.50894375E+08	0.69287510E+08
2	0.	198534.	0.	0.13207475D+11	0.51904422E+08	0.13131914D+11	0.51827975E+08	0.75560966E+08
3	0.	198534.	0.	0.13504297D+11	0.52841321E+08	0.13421310D+11	0.52761575E+08	0.82986552E+08
4	0.	198534.	0.	0.13810329D+11	0.53778221E+08	0.13718917D+11	0.53695175E+08	0.91411358E+08
5	0.	198534.	0.	0.14125771D+11	0.54715121E+08	0.14024914D+11	0.54628775E+08	0.10085672E+09
6	0.	198534.	0.	0.14450741D+11	0.55652021E+08	0.14339671D+11	0.55562375E+08	0.11107004E+09
7	0.	198534.	0.	0.14785624D+11	0.56588920E+08	0.14663791D+11	0.56495974E+08	0.12183298E+09
8	0.	198534.	0.	0.15130420D+11	0.57525820E+08	0.14997362D+11	0.57429574E+08	0.13305866E+09
9	0.	198534.	0.	0.15484556D+11	0.58462720E+08	0.15339851D+11	0.58363174E+08	0.14470574E+09
10	0.	198534.	0.	0.15847861D+11	0.59399620E+08	0.15691682D+11	0.59296774E+08	0.15617913E+09
11	0.	198534.	0.	0.16219679D+11	0.60336520E+08	0.16051539D+11	0.60230374E+08	0.16814006E+09
12	0.	198534.	0.	0.16600038D+11	0.61273419E+08	0.16420322D+11	0.61163974E+08	0.17971576E+09
13	0.	198534.	0.	0.16987382D+11	0.62210324E+08	0.16795821D+11	0.62097573E+08	0.19156097E+09
14	0.	198534.	0.	0.17382975D+11	0.63147233E+08	0.17178897D+11	0.63031174E+08	0.20407867E+09
15	0.	1191204.	0.	0.17785923D+11	0.64084143E+08	0.17568883D+11	0.63964774E+08	0.21704044E+09
16	0.	198534.	0.	0.18197431D+11	0.65021053E+08	0.17966695D+11	0.64898374E+08	0.23073690E+09
17	0.	198534.	0.	0.18616644D+11	0.65957962E+08	0.18372050D+11	0.65831974E+08	0.24459437E+09
18	0.	198534.	0.	0.19043730D+11	0.66894872E+08	0.18784835D+11	0.66765575E+08	0.25889482E+09
19	0.	198534.	0.	0.19477561D+11	0.67831788E+08	0.19203823D+11	0.67699181E+08	0.27373789E+09
20	0.	198534.	0.	0.19918861D+11	0.68768727E+08	0.19630092D+11	0.68632811E+08	0.28876929E+09
21	0.	198534.	0.	0.20367225D+11	0.69705666E+08	0.20063146D+11	0.69566441E+08	0.30407946E+09
22	0.	198534.	0.	0.20822834D+11	0.70642607E+08	0.20503574D+11	0.70500072E+08	0.31926005E+09
23	0.	198534.	0.	0.21285494D+11	0.71579548E+08	0.20950158D+11	0.71433704E+08	0.33533591E+09
24	0.	198534.	0.	0.21755179D+11	0.72516520E+08	0.21404047D+11	0.72367364E+08	0.35113207E+09

Table 7.1 (cont)

25	0.	198534.	0.	0.22231700D+11	0.73453514E+08	0.21864242D+11	0.73301048E+08	0.36745762E+09
26	0.	198534.	0.	0.22714781D+11	0.74390508E+08	0.22330617D+11	0.74234733E+08	0.38416339E+09
27	0.	198534.	0.	0.23203641D+11	0.75327503E+08	0.22802717D+11	0.75168421E+08	0.40092388E+09
28	0.	198534.	0.	0.23698976D+11	0.76264498E+08	0.23281552D+11	0.76102110E+08	0.41742412E+09
29	0.	198534.	0.	0.24200693D+11	0.77201496E+08	0.23766562D+11	0.77035800E+08	0.43413044E+09
30	0.	1191204.	0.	0.24708915D+11	0.78138504E+08	0.24257746D+11	0.77969497E+08	0.45116915E+09

DISCOUNT RATE = 10.00 BCR1 = 9.12 BCR2 = 10.45 ALT BCR = 4.67 RANGE = 1.02
DISCOUNT RATE = 7.00 BCR1 = 13.94 BCR2 = 15.84
DISCOUNT RATE = 4.00 BCR1 = 22.79 BCR2 = 25.65

	10% dr	7% dr	4% dr
DISCOUNTED COSTS $M =	167.982492	160.825374	153.894871
DISCOUNTED ACCIDENT BENEFITS $M =	111.418647	152.810436	223.474098
DISCOUNTED BASE ACCIDENT COSTS $M =	18540.431739	24665.653776	34857.051590
DISCOUNTED PROJECT ACCIDENT COSTS $M =	18429.013092	24512.843340	34633.577492

Where
CONST = Annual Construction cost
MAINT = Annual additional road maintenance cost resulting from project construction
OTHER = Annual other costs such as environmental costs
BCR1 = Benefit cost ratios assuming project opened to traffic at nominated time
BCR2 = Benefit cost ratios assuming that the project was delayed 2 years
ALT BCR = Alternate benefit cost ratio – see report for method of calculation
RANGE = Estimated error in the benefit cost ratio caused by rounding errors in calculations

7.7 Updating coefficients in the average speed equations for calculating road user costs

Urban traffic forecasting around Australia is done using a number of different commercial traffic forecasting packages. The system described here works with one package only at the present time. It takes considerable effort to change from one system to another. It was recognised that there would be a continuing need for road user costs to be calculated from average speeds. Therefore part of the Austroads work was to find some way of updating the average speed coefficients that allowed for urban driving conditions and used the updated vehicle consumption equations and the changes in vehicle types and traffic composition. This was achieved by adding additional computer code to the evaluation program. Now, each time the unit prices are updated, base and future year traffic assignments to the Perth regional network are used to summarise the updated road user costs in increments of average link speed. This information is written to a data file and linear regression is then used to derive new coefficients. These coefficients are then published with the updated unit prices.

7.8 Summary

Normal use of the program is to read two networks, one for the base case and one for the project, each containing estimates of traffic flows at two points in time with the study area being the whole networks. However, at the other extreme, it can also be used for a single intersection where say a signalised intersection is having additional lanes at the stop lines added. In this case one network with two estimates of traffic would be used with the assumption that the project volumes were the same as the base case volumes. The increased intersection capacity in the project case would be read from the project file. The program is versatile in its ability to accept various levels of traffic data availability and provides the user with results from three discount rates, plus sensitivity to timing changes and the likely range from calculation errors.

In Chapter 8, comments are made on how the system can be further developed. It also reports on the results of a literature search for a suitable urban economic analysis method that was carried out by ARRB Transport Research on behalf of Austroads.

Appendix 7.1 Estimating the error range

For an explanation of errors, please refer to section 7.5.

Total base case road user costs = RUC_1 +/- ε_1

Total project road user costs = RUC_2 +/- ε_2

Total benefits = RUC_1 - RUC_2 +/- $(\varepsilon_1 + \varepsilon_2)$

Let volume error be = ε

Then total error $$E = \sqrt{(\varepsilon^2} + (\varepsilon_1 + \varepsilon_2)^2) \qquad (37)$$

Then BCR range = E / Discounted capital costs

CHAPTER 8

GENERAL COMMENTS

8.1 Introduction

The previous chapter described the final step in the evaluation of urban road projects. The evaluation step also has other uses. One of these is to determine when closure has been reached in the iterative process of building trip tables. The process described so far is a single mode only evaluation system. It can easily be extended to make it an optional single or multi mode evaluation system. Other possible improvements are also described. Finally, the results of comparing TRAMS with other systems are reported on.

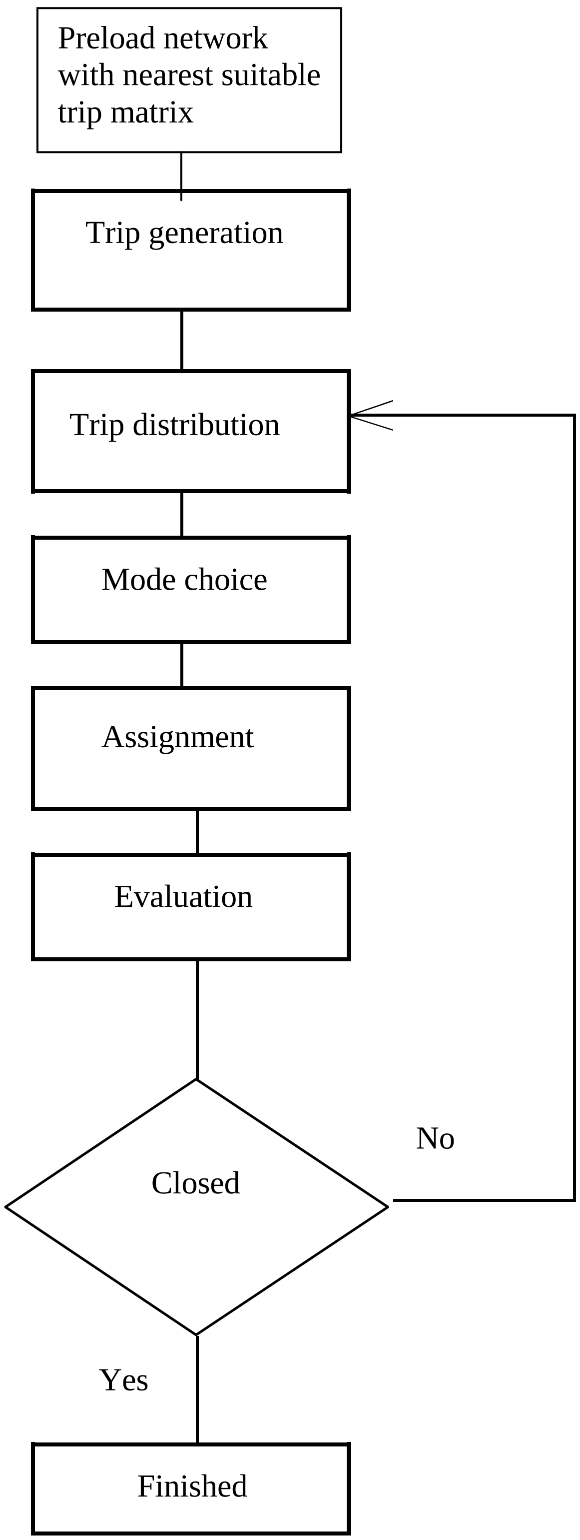
8.2 Alternate uses

The procedure for estimating travel on an urban road network is a four step process as follows:

- trip generation
- trip distribution
- choice of travel mode
- assignment of travel to the network.

Trip generation is where the ends of journeys are related to the land use, such as employment, educational enrolments, dwelling units etc. Trip distribution connects the two ends of journeys into a matrix of travel between areas. Choice of mode deals with the means of travel while assignment selects the route through the network. The speed of travel on the network affects trip distribution, which in turn affects the speed of travel. Thus, the process is iterative, starting with an assumed speed of travel on each link and progressively adjusting speeds until the speed on each link is correct for the volume of travel on the link. The evaluation program is used in this process as an additional step after the assignment step to measure the change in the road user costs between successive iterations (see Figure 8.1).

Figure 8.1 Process of building trip tables



When this change reduces below a set value, the process of building trip tables is terminated. The value in current use in the model for the Perth Metropolitan area is +/- $1,000,000 per annum in the year of the assignment. With the total annual road user cost in the system of about $13,000m, this represents a change of about 1 in 13,000 or about 0.008%. For a $50m dollar project with a favourable BCR, the annual benefits are likely to be of the order of $20m per annum in the first year. In this case the closure criteria represents a difference of about 1 in 20 or 5%. Results from one such run are listed in Table 8.1.

Table 8.1 Closure results after each iteration of building a trip table

Total trips	% of old skims	% of new skims	Change in rubft $m pa	No. links > 500 difference	Mean absolute difference vpd	Largest volume change vpd
3940426	N/A	N/A	-60.1326	584	125.64	18180
3940539	30	70	9.7427	229	77.84	1381
3940559	50	50	3.4693	73	55.35	1354
3940567	60	40	1.7072	87	58.73	1106
3940567	60	40	1.3282	141	64.23	1098
3940568	60	40	0.2764	137	68.72	2276

N/A = not applicable

With the passage of time, two changes have occurred which make it possible to tighten the closure criteria with little or no increase in the number of iterations required. Computers are now much faster making it possible to reduce the maximum number of trips leaving a zone being loaded in one increment from 960 down to say 400 to 500. It is now possible to store trip matrices in units of 0.001. This will considerably reduce the rounding errors compared to working with integer trip matrices. Some experimentation is warranted to determine a suitable closure value.

A second use of the evaluation program is when doing corridor planning studies to choose a preferred alignment out of many possible routes to be incorporated in statutory plans. At a very early stage, before alignments are sufficiently detailed to do an estimate of the likely project cost, an evaluation is run using a nominal construction cost to see which routes are more beneficial in terms of reducing road user costs. This helps in reducing the number of options requiring more detailed study.

The system has also been used to estimate the amount of fuel consumed. This is done by setting the price of fuel to $1.0 per litre and setting all other prices to zero. The annual total road user costs are then the annual fuel consumed in litres.

8.3 Four step travel forecasting procedure versus equilibrium solution of combined distribution and assignment

Over the years, researchers have raised concerns that traffic modellers have not been doing sufficient iterations of the four step process of trip table building to ensure that the resulting travel is in equilibrium with the speeds assumed for the trip distribution stage. To overcome this problem Evans combined the trip distribution and assignment into one stage. This was further developed by Florian, Nguyen and Ferland.

Boyce, Zhang and Lupa used a large number of iterations of the combined model to set up a ‘perfect assignment’ and then proceeded to compare the results of alternate methods of building and assigning trip matrices with this ‘perfect assignment. Four of the methods are variations of the four step procedure and the fifth method uses the combined distribution – assignment model. There are several problems with this work. In the various four step model methods, some component of each iteration is retained in the ‘final’ assignment. This means that the method will never close to equilibrium, although it may get close given enough iterations. For closure to occur, interpolation must be restricted to the most recent iterations. Commercial packages store matrices as integers, albeit with a user specified scale factor. The magnitude of numbers in the travel separation matrix is always non zero and larger than the numbers in the trip matrix where up to 70% of the cells contain between zero and two trips. Rounding errors will therefore be less if the interpolation is done between iterations of the travel separation matrix.

The combined trip distribution and assignment model limits travel estimation to only one matrix. While this may be reasonably accurate for a strategic model with very large zones where upwards of 70% of all travel does not leave the zone, modellers have found that matching model flows to traffic counts at the operational level where upwards of 90% of all travel is out on the network is much more accurate when travel is divided into somewhere between seven and 13 purposes. The four step method gives a better match between estimated and counted traffic volumes than the combined model. Any errors resulting from the trip table not being in equilibrium are minimised during project evaluation by using constant trip tables. On the odd occasion when the project being evaluated is so large that separate trip tables have to be prepared for the with and without project, additional care is taken during trip table preparation to get closer to equilibrium.

8.4 Possible future improvements

The current system is a single mode, road based system. While the user does have the flexibility to vary the traffic composition by road type, in practice, a constant composition is used over the whole network, as it is difficult to obtain suitable disaggregated composition data. Now that the assignment program has the capability to separately assign light commercial and heavy commercial vehicles, it would be possible to change the system to expect a three way division of the composition on each and every link in the network. This would improve the accuracy as it is known that although the system average percentage of heavy commercial vehicle kilometres of travel is about 6%, on some links approaching industrial areas, manual classification counts have recorded percentages as high as 29%, while on local roads, the percentage can be less than 2%.

The system can easily be expanded to a full multi-modal system by reading matrices of travel and travel time for modes other than vehicles and their drivers. Vehicle passengers could be counted in with vehicle drivers (the preferred option as the annual increase in the travel time between zones is non linear even with linear increases in traffic) or counted in the matrices component at the discretion of the user. For modes other than road, this would provide traveller cost estimates for only the years of the travel forecasts. Travel costs for non road users vary only with trip travel time and do not also vary with the speed of travel, as is the case for road vehicles. This makes the assumption of a linear interpolation of travel costs for non road users less of a problem than for road users, as the error will be smaller. The system can be set up so that it could be run either as a multi-mode system or as a single road mode system as now, depending on users' needs for evaluating the project.

The method of estimating the number of phases at signals works very well most of the time. However, there are occasions when it would be very useful to have the ability to nominate the actual number of phases. Examples are intersections in the city area where there is an additional exclusive walk phase. In this area, the assignment program consistently overestimates the speed of travel. Other examples are large intersections such as West Coast Highway and Karrinyup Road that operate almost exclusively on four phases. This can be done by allowing the user to read in optional data of node number followed by number of phases. For those intersections where no data was supplied, the program would estimate the number of phases as now. The question of how to ensure that the number of phases used in the assignment is also used in the evaluation needs careful thought.

Vehicle emissions are now one of the items to be included in assessing the suitability of a road project. It would be possible to develop a method along similar lines to estimate vehicle emissions as these are closely related to fuel consumption, but also depend on the speed of travel, the age of the vehicle fleet and whether the vehicle has warmed up.

8.5 Comparison with other systems

About ten years ago, Austroads became concerned about the non comparability of road project economic evaluation results between States. They formed a committee (the Road User Cost Steering Group – RUCSG) to oversee research to improve the accuracy of evaluations. Initial work concentrated on the uninterrupted flow conditions experienced on rural roads. An existing model for estimating road user costs that had been developed for the Commonwealth Bureau of Roads in the late 1960s had its relationships for fuel consumption, tyre wear and repair and maintenance updated to reflect current vehicle technology. However, this model is for uninterrupted flow conditions and is not a network model.

As the first step in setting up a model for calculating road user costs in an urban network, ARRB Transport Research Ltd was commissioned to do a literature search both in Australia and overseas for a suitable urban model (Tsolakis et al 1998). Evaluation models from Australia, the United Kingdom, Sweden, New Zealand, the World Bank, U.S.A., South Africa and Canada were reviewed. Of all the models reviewed, the one used by MRWA and the subject of this thesis was the only one that estimated intersection effects separately from mid block. At that stage, the model was still based on research in the sixties and seventies so it was in need of updating. The result of the search was to recommend the updating of this evaluation system to use as the benchmark standard for use by the Australian State Road authorities. The recommendation accepted that the evaluation method depended on using one particular traffic forecasting package and that this may not be acceptable to all members. For those states that continued to use other systems, TRAMS would be used to arrive at road user costs as a function of link speed to use in those systems. The vehicle operating cost module has since been updated as part of the on going Austroads work (Lloyd and Tsolakis 2000). The model described in this thesis includes that updating.

8.6 Conclusion

There are many demands on funds for Transport projects to provide access to new developments, to improve the service in the existing network and to maintain the existing assets. To ensure that the available funds are spent wisely means that in addition to other criteria, projects must be subjected to an economic analysis. The results of these analyses are used to ensure value for money spent, and to rank projects in a priority order, as not all worthwhile projects can be funded.

As part of the development of this evaluation system for urban roads, new concepts not known to the author to be previously used elsewhere were developed. These are

- Estimation of delays at signalised intersections and traffic merge areas using volume to capacity ratio as the independent variable. This reduces the need for data at individual intersections. The delay formulae have been derived from a conventional analysis using volumes, green times and cycle times with the delay then expressed in terms of volume to capacity ratio.
- Delays at intersections include the excess time involved in the speed change regimes of deceleration and acceleration combined with the probability of having to stop. The normal delay formulae estimate the stationary time only.
- Delays at signal controlled intersections are directly proportional to the number of signal phases. This allows the use of a single set of coefficients for estimating delays by including the number of phases in the formulae.
- The use of a genuine incremental assignment of traffic to the road network using the speed of the marginal vehicle. This obviates the need for a trial and error procedure to match link speeds to the resulting link volume. This incremental procedure is not to be confused with the optional ‘incremental’ assignment in commercial traffic forecasting packages. The latter procedure is another variation of the problem of matching link speeds to the resulting volume, but it uses the assumption that all vehicles on a link travel at the same speed. This assumption is not correct, even for peak periods.
- A similar process is used to combine variable traffic flow rates throughout the day with traffic composition to develop all day speed flow curves and to estimate road user costs in terms of volume to capacity ratio. No other system

provides a process for developing all day speed flow curves or road user costs using a profile of the varying flow rate throughout the day.

- Expressing vehicle fuel consumption per kilometre in terms of vehicle characteristics of average vehicle mass, engine power, frontal area and drag. This makes it easy to introduce vehicles with other characteristics to the system. It also allows for the use of a much wider range of vehicles than the limited number for which fuel consumption measurements have been obtained.
- Expressing the correction for grade effect on fuel consumption in terms of vehicle mass. This allows for a much wider range of vehicles to be used than the limited number for which fuel consumption measurements have been obtained. It also means that when the permitted axle masses are next increased, the fuel consumption will be automatically adjusted, just by changing the average gross vehicle mass.
- Expressing the effect of stopping on vehicle consumption variables (fuel, tyre wear etc) in terms of distance travelled at cruising speed to produce the same consumption. Again, this makes it easier to extend the formulae to include vehicles for which data has not been collected. It also means that if the cruising speed consumption coefficients are changed, the intersection component is automatically adjusted.
- When hourly demand exceeds hourly capacity, vehicle times and costs are estimated by calculating the time spent in the queue instead of notionally extending the speed flow curve. This means the data is directly calculated instead of being estimated from a notional extension of the speed flow curve.
- When projecting traffic volumes for up to thirty years, volumes on individual links are likely to exceed realistic estimates of traffic as by then additional

capacity would be added to the network. In these circumstances, the system proportions the traffic between the existing link and the unknown future link(s) so that estimates of benefits only apply to a reasonable level of traffic on the existing link.

- The evaluation system reads the traffic forecasting system network files directly. Estimating benefits for years other than when the forecasts apply is by linear interpolation of traffic volumes at the individual link level between the two traffic forecasts provided. Road user costs are then calculated for each year using the interpolated traffic volumes.
- The system performs an evaluation for three discount rates and two project start dates.
- The system calculates an error term in the magnitude of the estimated road user benefits and expresses this as a range in the benefit cost ratios.

The system disaggregates travel into its basic components by separately analysing intersection effects and travel between intersections for all intersections represented by node numbers in the network. All input into the system is prime values such as the hourly capacity per lane between the intersections and the hourly capacity per hour of green time at signalised intersections. There are no composite values used, such as a single capacity to represent travel between intersections and the capacity of the intersection approach. The effort of providing the necessary intersection information adds only marginally to the work involved in network preparation and the system is economical in its use of resources. An assignment to a 947 zone network containing over 12,000 links runs in about 15 secs with intersection delays calculated at over 4,000 nodes. The system responds to changes in any of the elements of road design (such as road type, number of lanes, speed limit, type of intersection control and number of turning lanes) that a road designer may change and is in everyday use.

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