Guide to Traffic Management Part 3

Traffic Studies and Analysis

Austroads

Guide to Traffic Management Part 3: Traffic Studies and Analysis

Austroads

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Guide to Traffic Management Part 3: Traffic Studies and Analysis

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Abstract

Austroads' *Guide to Traffic Management* has 13 parts and provides comprehensive coverage of traffic management guidance for practitioners involved in traffic engineering, road design, town planning and road safety.

Guide to Traffic Management Part 3: Traffic Studies and Analysis is concerned with the collection and analysis of traffic data for the purpose of traffic management and traffic control within a network. It serves as a means to ensure some degree of consistency in conducting traffic studies and surveys. It provides guidance on the different types of traffic studies and surveys that can be undertaken, their use and application, and methods for traffic data collection and analysis.

Part 3 covers applications of the theory presented in Part 2 of the Guide, and provides guidance on traffic analysis for uninterrupted and interrupted flow facilities and for various types of intersections. It outlines sound methods of analysis for effective traffic management, design and control.

Keywords

Traffic management, road network, data collection, data analysis, field study, speed, origin-destination, level of service, traffic capacity, traffic flow, intersection, unsignalised intersection, roundabout, signalised intersection.

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- In Section 4.1, corrects Equation 5 for the calculation of the capacity of a significant length of a single traffic lane for the prevailing roadway and traffic conditions.
- Corrects a worked example in capacity analysis in Section C.1.1.1.

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- This third edition updates Section 2 and Appendix K, and provides a new Appendix L with alternative data sources and their use cases for traffic studies. This version also includes a new Section 3.4 Multi-modal Level of Service and a new Section 3.5 Austroads Pedestrian Facility Selection Tool. A new Section 7 and Appendix M were produced and two commentaries were amended to provide high-level modelling guidelines.

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About Austroads

Austroads is the peak organisation of Australasian road transport and traffic agencies.

Austroads' purpose is to support our member organisations to deliver an improved Australasian road transport network. To succeed in this task, we undertake leading-edge road and transport research which underpins our input to policy development and published guidance on the design, construction and management of the road network and its associated infrastructure.

Austroads provides a collective approach that delivers value for money, encourages shared knowledge and drives consistency for road users.

Austroads is governed by a Board consisting of senior executive representatives from each of its eleven member organisations:

- Roads and Maritime Services New South Wales
- Roads Corporation Victoria
- Queensland Department of Transport and Main Roads
- Main Roads Western Australia
- Department of Planning, Transport and Infrastructure South Australia
- Department of State Growth Tasmania
- Department of Infrastructure, Planning and Logistics Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- Australian Government Department of Infrastructure and Regional Development
- Australian Local Government Association
- New Zealand Transport Agency.

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1. Introduction

1.1 Scope and Context

Part 3 of the Austroads *Guide to Traffic Management* has been given the title *Traffic Studies and Analysis* to define the limitations on its scope within the contexts of:

- the 13 different Parts of the *Guide to Traffic Management*
- other Guides spanning the range of Austroads publications.

The structure and content of the 13 Parts of *the Guide to Traffic Management* are discussed in *Part 1: Introduction to Traffic Management*. The 13 Parts are outlined in Table 1.1.

In the context of the Guide, *Part 3: Traffic Studies and Analysis* outlines the importance of traffic data and its analysis for the purpose of traffic management and traffic control within a network. It serves as a means to ensure some degree of consistency in conducting traffic studies and surveys. It provides guidance on the different types of traffic studies and surveys that can be undertaken, their use and application, and methods for traffic data collection and analysis.

Part 3 provides a means to assist in implementing the various traffic management and control measures described in:

- *Part 4*, which is concerned with the overall operational improvement needs of the road network as a whole and for all road users (Austroads 2016a)
- *Part 5*, which deals in detail with mid-block traffic management issues that apply to individual lengths of road (Austroads 2017a)
- *Part 6*, which deals with traffic management issues and solutions at particular intersections, interchanges and crossings (Austroads 2017b)
- *Part 9*, which covers the operational management of road space for all users and describes current practice for common systems including traffic signals, congestion management, incident management and traveller information (Austroads 2016b).

Part 3 covers applications of the theory presented in *Part 2* (Austroads 2015a), and provides guidance on traffic analysis for uninterrupted and interrupted flow facilities and for various types of intersections. It outlines sound methods of analysis for effective traffic management, design and control.

Part 3 updates previous Austroads guidance on traffic studies and capacity analysis, and makes reference to other seminal publications such as the US *Highway Capacity Manual (HCM)* (Transportation Research Board (TRB) 2016) (referred to subsequently as HCM 2016) and the *Traffic Engineering and Management* handbook from Monash University (2003).

The 2000 edition of the *Highway Capacity Manual* (TRB 2000) (referred to subsequently as HCM 2000) contained useful information and this Guide includes materials from both HCM editions.

Table 1.1: Parts of the Guide to Traffic Management

Part	Title	Content
Part 1	Introduction to Traffic Management	• Introduction to the discipline of traffic management. • Breadth of the subject and the relationship between the various Parts of the Guide.
Part 2	Traffic Theory	• An introduction to the characteristics of traffic flow and the theories, models and statistical distributions used to describe many traffic phenomena. • Processes that practitioners should consider.
Part 3	Traffic Studies and Analysis	• Traffic and transport data collection surveys and studies. • Traffic analysis for mid-block situations (including motorways). • Analysis of signalised and unsignalised intersections, including roundabouts. • High-level modelling guidelines.
Part 4	Network Management	• Broad strategies and objectives of managing road networks to provide effective traffic management for all road users. • Network needs for heavy vehicles, public transport users, pedestrians, cyclists and private motor vehicles. • Guidance on transport networks and network operation planning.
Part 5	Road Management	• Guidance on managing mid-block traffic conditions. • Good practice for access management, allocation of space to various road users, lane management. • Application of speed limits.
Part 6	Intersections, Interchanges and Crossings	• Types of intersection and selection of intersection type. • Appropriate use and design of various intersection types. • Traffic management issues and treatments for intersections, interchanges and other crossings.
Part 7	Traffic Management in Activity Centres	• Principles for planning the management of traffic in activity centres and associated transport nodes. • Techniques for traffic management in activity centres. • Examples and key considerations for various types of centres.
Part 8	Local Area Traffic Management	• Planning and management of road space in a local area. • Guidance on selection, design, application and effectiveness of traffic control measures on an area-wide or at least whole-of-street basis.
Part 9	Traffic Operations	• Applications used in traffic operations. • System configuration and operation guidance. • Current practice for common systems including network monitoring, traffic signals, congestion management, incident management, motorway management and traveller information. • Related systems integration and interoperability issues.
Part 10	Traffic Control and Communication Devices	• Signing and marking schemes. • Traffic signs, static and electronic. • Pavement markings and delineation. • Traffic signals and islands.
Part 11	Parking	• Parking policy. • Demand and supply. • On-street and off-street. • Parking guidance and control devices.
Part 12	Traffic Impacts of Developments	• Guidance on the need and criteria for impact assessment. • Detailed procedure for identifying and assessing traffic impacts and mitigating their effects. • Assessment of safety, infrastructure and environmental effects.
Part 13	Road Environment Safety	• Principles and management of the safety of road environments within a traffic management context. • Links to relevant sections of the Guide to Road Design and the Guide to Road Safety.

1.2 Introduction to Traffic Studies and Traffic Analysis

In the context of *Part 3*, the term 'traffic survey' refers generally to the procedures and activities involved with measurement and data collection. The term 'traffic study' involves the collection, analysis and synthesis of data relating to traffic and its characteristics. In practice, traffic study is often used generically to refer to the combination of survey and analysis activities.

Traffic studies and surveys provide essential information for much of traffic engineering planning, design and practice. The results are used in traffic planning, traffic management, traffic and environmental control, economic studies, road safety studies, land-use and transport interaction studies, and in establishing and revising design standards and system models.

Traffic surveys are the means of collecting traffic data, providing basic information about the operating characteristics of the traffic. This typically includes information on vehicle flows, speeds, travel times, delays, pedestrian and cyclist movements, journey origins and destinations, and vehicle types and characteristics.

The information may be sought at a number of levels of detail, from broad indications of traffic conditions over a region to a detailed measurement of individual vehicle movements at a chosen location. A good working knowledge of survey techniques and technology is required by all traffic practitioners.

A systematic approach to traffic surveys and studies provides a central framework for drawing together the wide range of needs, requirements and techniques for the various types of traffic surveys. This assists in a proper understanding of traffic data and their use in analysing and determining the characteristics of traffic systems.

Traffic studies include the synthesis and analysis of traffic information for the purposes of traffic management and control, and for the design of roads and associated traffic facilities. They often involve evaluations of existing conditions or projections relating to proposed changes. The analysis of the traffic characteristics for these purposes requires an understanding of the basic concepts of capacity analysis and level of service (LOS).

Capacity analysis is fundamental to the planning, design and operation of roads and transport services. It provides quantitative techniques for measuring the effectiveness of existing transport facilities in moving traffic and people, and is used to determine the impacts of proposed system improvements (e.g. additional traffic lanes, signal timing adjustments, enhanced public transport services).

Capacity analysis provides the basis for determining the number of lanes, lane disciplines and lane types to be provided, having regard for the volume and composition of traffic and the prevailing roadway and traffic control conditions. For a given number of traffic lanes, capacity analysis provides a means of determining the traffic carrying performance of a road under the prevailing roadway, traffic and control conditions.

Quite different analytical approaches are applicable for the basic types of traffic facility, which are categorised as:

- uninterrupted flow facilities, on which traffic flow conditions are the result of interactions between vehicles in the traffic stream, and between vehicles and the geometric characteristics of the road
- interrupted flow facilities, on which traffic flow conditions are subject to the influence of fixed elements such as traffic signals, stop signs or other controls which cause traffic to stop periodically.

Signalised intersections and major unsignalised intersections influence the capacity and traffic performance of interrupted flow facilities, and specific approaches are applicable for analysing the capacity of various intersection types.

Capacity analysis is not restricted to roads. HCM 2016 provides concepts and methods for undertaking capacity analysis for pedestrians and cyclists on footpaths and bicycle facilities.

A primary objective in planning, designing and managing traffic facilities is to achieve a balance between maximising flow rates and ensuring acceptable operating conditions for users of the facilities. The notion of acceptable operating conditions is encompassed in the concept of LOS.

In this Guide, the capacity of a facility is usually expressed in passenger car units (pcu) per time period, e.g. pcu/h. If the capacity is expressed in vehicles per hour (veh/h), then the traffic composition in the traffic stream should be mentioned to convert veh/h to pcu/h, i.e. the percentage of heavy vehicles (HV) and the pc equivalent of the HV should be available.

Level of service (LOS) is a qualitative measure for ranking operating conditions or service quality, based on service measures such as speed, travel time, delay, density, freedom to manoeuvre, interruptions, comfort and convenience.

The term volume-to-capacity ratio (VCR) (or degree of saturation) means the ratio of demand or arrival flow to capacity, and therefore can be larger than 1, representing oversaturation.

1.3 Structure of this Part

Section 2 describes an overall systems approach to traffic studies, including statistical and sampling issues. Major components of traffic studies are identified, and the interactions between these components are highlighted. The section also summarises various types of traffic studies and surveys. In many survey areas, new and alternative technologies are increasingly employed for data collection and analysis. These new and alternative data sources with supporting techniques are discussed at the end of the section.

Section 3 presents an overview of the concepts of capacity, LOS and degree of saturation and the factors which affect them.

Section 4, Section 5 and Section 6 provide information and guidance on capacity analysis as applied to uninterrupted flow facilities, interrupted flow facilities and intersections, respectively. Substantial reference is made to detailed information available in HCM 2016. In these sections traffic analysis methods using graphs and equations are included to give practitioners an understanding of the fundamental principles and relationships. In current practice, most methods are pursued via computer-based analysis, and in the context of electronic publications many of the graphical approaches and equations could eventually be replaced by on-line spreadsheets.

Section 7 provides high level modelling guidelines including the selection of appropriate modelling approach, modelling study procedures, microsimulation modelling applications and the use of modelling outputs.

The detailed information and guidance for each type of traffic surveys are described in Appendix A to Appendix I. Error in seasonal adjustment factors is described in Appendix J. Emerging and alternative survey technologies are described in Appendix K. Examples of the use of new and multiple data sources are given in Appendix L and example packages for modelling are provided in Appendix M.

2. Traffic Studies and Surveys

2.1 Traffic Studies and Data Needs

Traffic studies have a central place in transport planning and traffic engineering. They are performed to:

- provide a basis for planning and designing traffic facilities, including the selection of geometric standards, economic analysis, impact assessment, and the determination of priorities
- assist traffic operation by indicating the needs for traffic control devices such as signs, traffic signals, pavement markings, and school and pedestrian crossings
- evaluate the effects of road safety measures and other changes made for traffic by conducting 'before and after' studies
- determine the basic characteristics and the general laws of traffic behaviour
- provide heavy vehicle and freight data to improve pavement analysis and design capability, bridge management capability, and the monitoring of road network performance.

Road usage is also linked to funding of maintenance programs, so collection and analysis of data on road traffic performance can be a fundamental component of submissions for funding allocations.

The importance of traffic studies is expanding in the face of three influences (Taylor et al. 2000):

- the rapid developments in data collection hardware, driven by microprocessor developments and the corresponding ability to collect and analyse more data of better quality
- the increased demands placed on the traffic engineering profession to obtain improved efficiency of operations in traffic systems in order to meet increasingly complex objectives for the community
- the traffic engineer's own demands for better quality data to permit the use of the most recent traffic planning and design techniques and modelling procedures.

The purposes for which traffic data are required may be summarised as follows (Austroads 2006, Young et al. 1989):

- monitoring – the collection of information about traffic conditions prevailing at any time, and as they change over time
- forecasting – the use of data on existing traffic systems as one of the inputs to a procedure for estimating what the traffic would be like under different conditions, either now or in future
- calibration – the use of traffic data to estimate the values for one or more parameters in a theoretical or simulation model
- validation – the verification of an analytical or simulation model against information independent of that used to calibrate the model.

Information typically sought in traffic surveys includes:

- flows of vehicles (possibly classified by vehicle type) and pedestrians
- numbers of waiting vehicles or pedestrians
- flows of bicycle users
- numbers of parked vehicles
- numbers of occupants in vehicles
- speeds of vehicles
- travel times, delays and their components

- origins and destinations of journeys
- vehicle mass and dimensions
- fuel consumption and emissions
- traffic characteristics related to driver behaviour such as saturation flow rates at traffic signals and critical acceptance headway, and follow-up headway at roundabouts and unsignalised intersections.

In most cases, such information is generally obtained by passive observation and measurement, without making direct contact with road users or directly interfering with traffic flows. Technological developments in data acquisition and analysis are facilitating the efficiency and effectiveness of passive observation methods.

The common traffic surveys are described in Section 2.5 and details are provided in Appendix A to Appendix I. A comprehensive overview of traffic and transportation surveys is provided in Institute of Transportation Engineers (2010a) and survey methodology and data analyses are examined in Taylor et al. (2000).

General guidance on conducting traffic studies and surveys is given in Currin (2001), Homburger et al. (2007), Monash University (2003) and Institute of Transportation Engineers (2016).

2.2 Planning and Designing Traffic Studies

All traffic studies require careful planning and administration of data collection, collation and analysis tasks.

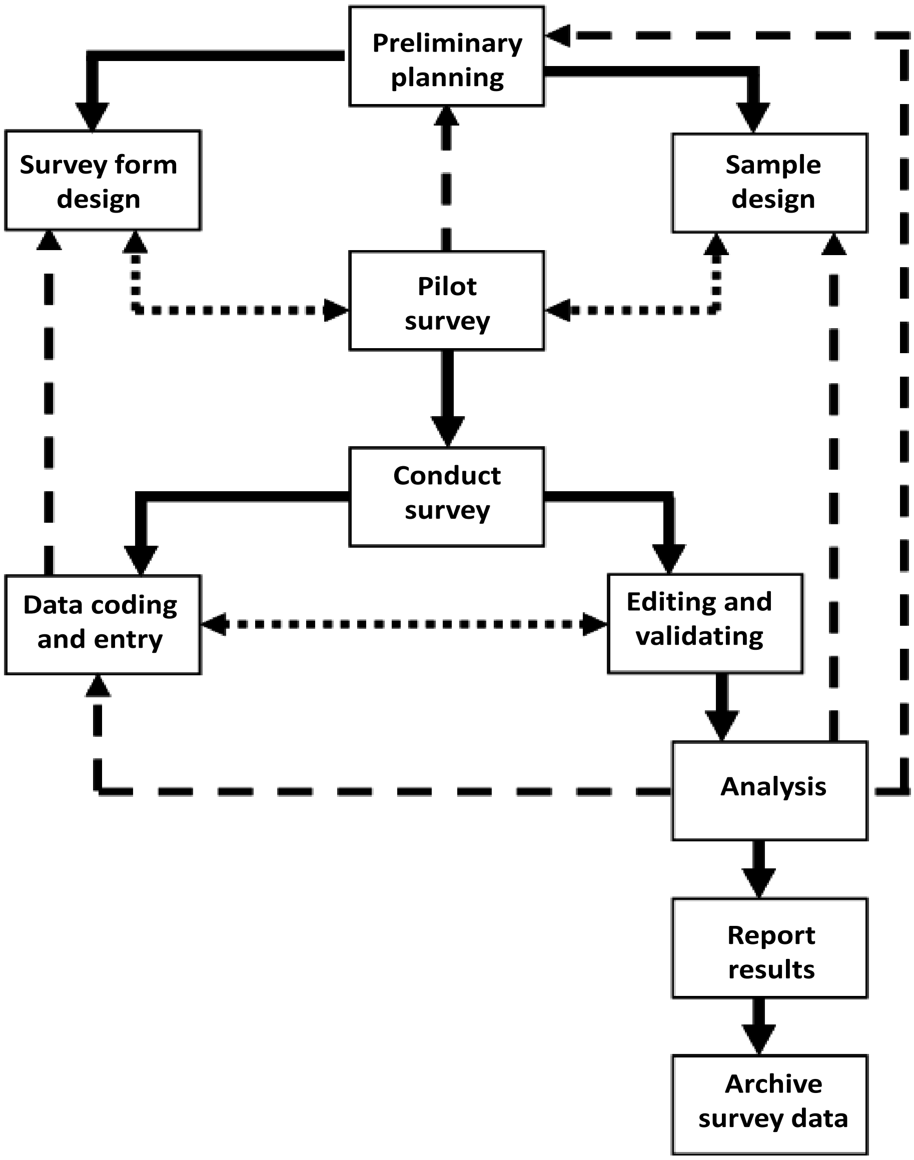
2.2.1 Systems Approach to Traffic Studies

The systems approach allows the identification of processes common to all traffic studies, which form the fundamental building blocks of good study design.

Figure 2.1 indicates that traffic studies should follow a series of logical, interconnected steps leading towards the final outcome of the study. It is also to be noted that there are significant linkages between the various activities. These linkages are of three types – forward, feedback and backward:

- Forward linkages (solid lines) indicate the basic progression through the process.
- The feedback linkages (dotted lines) between component activities indicate that two or more activities must be performed sequentially in a closed loop. For example, having performed the pilot survey, it may be necessary to redesign the survey form and then pilot test the new form.
- The backward linkages (dashed lines) indicate where information may be transferred back from a later activity to one which preceded it. For example, the design of the survey form may be affected by the coding methods used to extract data from the forms, while the type of data analysis may influence coding itself. It is important that consideration is given to backward linkages so that decisions made early in the survey process do not preclude options for later data analysis.

Figure 2.1: Systems approach to traffic studies



Source: Austroads (2013).

The following sections outline each component in the traffic analysis system in Figure 2.1 and highlight major considerations required for the design of various survey types.

2.2.2 Preliminary Planning

The initial activities in a study are of paramount importance. The best way to ensure the success of a traffic study is to recognise and explore the following steps.

Study objectives

The study team, in conjunction with the client for whom the study is to be conducted, should develop objectives specifying the basic questions to be answered in both qualitative and quantitative terms. For example, prior agreement with the client should be made on the level of sampling and the level of statistical significance before a survey starts.

Review of existing information

Before embarking on the collection of a new set of data, the investigator should ascertain just how much is known about the subject in question. Existing knowledge is available through the review of published literature, knowledge of current and completed studies and other relevant data sets compiled by other agencies. If these sources indicate that the subject has been partially researched, the existence of other data sets could be useful. In particular, it may provide new ideas about the study and assist the survey design, e.g. by providing estimates of population parameters for use in determining sample size.

Formation of hypotheses

If new data needs to be collected to satisfy the objectives, then the correct types of data must be collected. The only way to ensure the relevance of the new data is to develop the possible hypotheses to be tested. This is a fundamental part of the methodology of scientific inquiry on which traffic studies should be based. The scientific approach consists of four major phases: hypothesis formulation, observation, testing, and refinement of the hypothesis. Data collection is the basis of observation, and this must be preceded by hypothesis formulation. The recommended practice is to establish a range of plausible hypotheses, and then determine the survey requirements so that each hypothesis may be properly tested.

Definition of parameters

After hypothesis formulation, there is often a need to refine the meanings of several concepts and terms. Some effort is then needed to clearly define the terminology to be used in the survey.

This is important for two reasons. Firstly, without the agreement of analysts and users of survey data on terminology, ambiguities and disagreements over concepts and parameters are likely. Secondly, agreement allows comparisons between different data sets, survey methods and analyses.

A clear definition of parameters in a traffic study leads to a better interpretation of the results.

Resource determination

In most studies, there will be constraints in terms of the level of resources that can be committed to the survey. These are generally time, people and money, and a trade-off between available resources and the accuracy obtainable from a sample survey is possible (see *survey methods* below).

Survey content

The proper adoption of survey objectives facilitates the task of determining survey content.

Data items should only be included in the survey content if they are required for the intended analysis. Nevertheless, even this approach may result in a long list for the survey content. It might also be noted that some data additional to basic needs, collected at little or no extra cost, might be useful in future. The task of refining the list will be influenced by the trade-off between costs and available resources.

Survey methods

The choice of a survey method is crucial for the efficiency of the overall survey effort. The selected method will usually result from a compromise between the objectives of the survey, expressed in the survey content list, and the available resources. The quality of the data obtained will depend on the survey method selected and the amount of quality control performed. However, more precise survey techniques with higher quality control will generally consume more survey resources.

The use of advanced or alternative technologies such as video or global positioning system (GPS) technologies for recording a wide variety of road and traffic data is expanding. Issues relating to data quality, storage, access and integrity are just as pertinent for video methods as they are for other approaches using manual or computer-based methods.

The trade-off for survey type must be made on the basis of time, money and people so that a feasible survey method and sample size can be found for the successful completion of the survey.

2.2.3 Sample and Experimental Design

All traffic surveys are sample surveys. Traffic systems contain very large numbers of individual units and factors. It is uneconomic, if not virtually impossible, to observe all of them. What is required is to survey a suitably chosen sample from the population, on the basis of the experimental design, sampling methods and statistical theory in order to draw out statistically valid inferences (e.g. as described in Section 2.3 and sources such as Miller et al. 2010 and Taylor et al. 2000).

2.2.4 Survey Form Design

For standard surveys (e.g. turning movement counts or speed surveys), standard forms may exist. Most Austroads member jurisdictions will have their own forms developed over many years to meet their particular needs. However, special surveys may require new forms to be developed, e.g. lane saturation flows at signalised junctions (see *Parts 6* and *9* of the *Guide to Traffic Management* and Akçelik 1981).

An often overlooked aspect of traffic survey design is the design of forms for recording data. Careful attention to the design of a form can often lead to more efficient job performance and lower error rates by record and data entry officers. Modern information technologies have made the task of preparing quality survey forms much easier and cheaper; for example, electronic recognition of bar codes, or intelligent data collection forms mounted on laptop computers or personal digital assistant devices.

2.2.5 Pilot Survey

Pilot testing is an important component of a study design, but is often neglected. The usual reasons for ignoring pilot surveys are lack of time or money. Experience indicates that these are false economies for all but the smallest or most standard surveys. Pilot studies provide assistance with many aspects of the final survey design and analysis, including:

- adequacy of sampling and analysis method
- adequacy of survey forms
- efficiency of training methods
- suitability of coding and editing
- cost and duration of survey and analysis
- suitability of traffic control methods on-site, where applicable
- effectiveness of technology or data collection devices.

2.2.6 Conducting Surveys

The key to successful traffic studies lies in the attention paid by the study manager to the details of survey administration. Frequently a survey is undertaken by third parties according to a brief, with the analysis and reporting undertaken ‘in-house’. The brief should include:

- general outline of type of survey to be undertaken and purpose
- details of field survey and survey design (if included)

- competencies required and field instruction detailed
- criteria for data acceptance
- structure of data outcomes
- provision of the method for conducting the survey, data quality plan and training proposed by consultants.

Proposals should be assessed on the basis of management capability (scheduling, contingency planning, staff recruitment and training, quality plan, resource programming), technical capability (project management skills of key personnel, level of technical support), documented prior experience and price.

The following checklist may help to reduce the severity of problems arising during a survey:

- Personnel must receive training in the purpose of the survey and in the methods of measurements to be employed. On-hand support by competent administrators is especially critical where casual personnel are employed specifically for the survey. Field staff should be encouraged to provide feedback particularly on unusual events that might impinge on the quality of the data collected. The client should also be closely involved in the training process and identify deficiencies in the process.
- Replacement field staff should be available, especially on the first day of a multi-day survey. During the survey, rosters of rest periods for observers are essential, and replacement staff will be needed to cover these periods.
- Survey forms should be prepared and distributed, as far as possible, on the eve of the survey. The pre-study briefing is ideal for this distribution.
- Procedures should be in place to ensure quality control of data and survey operations. Plans and procedures should be in place to accommodate failure to obtain information. An example would be where an observer either fails to collect travel time data over a route due to collection equipment malfunction or there are systematic errors introduced due to non-compliance with instructions.
- Occupational health and safety policies must be followed. They are essential in maintaining the alertness and concentration of field personnel and reducing the errors of observation and recording. Depending on the survey type, a maximum three-hour shift (preferably two-hour maximum) should be considered. Often the full complement of field staff will be required for active surveying only during peak periods. In between these times some staff can take rest periods while others share their duties. The obvious proviso is that the times of peak traffic movements are known before the survey schedule is set. Proper rostering of field personnel taking account of the above factors may avoid or reduce high overtime costs and safety risks.
- Privacy legislation must be followed. Data of a confidential nature should not be disclosed without proper clearance.

2.2.7 Data Coding and Entry

The time spent in the field is usually a small proportion of the total time required to produce the study results. Data entry and compilation is time consuming and new technologies often provide the potential for significant savings.

Data coding is the process of converting data from field records into computer-readable formats. Editing is the parallel process of data scanning to detect recording errors and logical errors in the data. Editing may be performed manually or automatically.

The major opportunity for time saving is the combination of coding and editing into a single data-entry process, where the data are entered interactively by the user. Many inconsistencies can be detected immediately by the editing program, enabling the user to quickly correct faulty data entries. Further time savings are possible with the use of hand-held data loggers, laptop personal computers (PCs) and voice recognition methods for field recording: the data are ready for analysis as soon as recording finishes. In some cases, the same PC used to collect the data can perform analyses in the field. The possibilities for using PCs and wireless Internet technologies for data transfer are expanding rapidly.

2.2.8 Data Analysis

Data analysis is an important element of any traffic study. It is the means by which the collected data are used to test the hypotheses set during the planning of the study. Statistical procedures are employed to make this part of the process as effective as possible. Sources such as Miller et al. (2010) and Taylor et al. (2000) provide good references for the application of statistical analysis in traffic studies. See also Section 2.3 for guidance on methods for ensuring adequate sample sizes for statistical testing.

2.2.9 Presentation of Results

The preparation and distribution of reports is the primary means of conveying the study methods and results to other people. There is little point in undertaking a study if it is not adequately reported, and proper preparation of the report is essential. The following principles should be used when preparing a survey report:

- The purpose and scope of the survey should be fully explained.
- The report must cater for the types of readers for whom it is being written, the likely extent of their knowledge and understanding, the types of problems and questions that are likely to be of interest, and the kind of language (technical or otherwise) to which they are accustomed. Statistical technicalities should be translated into language which will be understood by readers who are interested in the broad and substantive results of the study.
- Statistical results should be framed jointly in terms of percentage error and confidence interval. Because a study typically deals with a sample of a broader target population, the degree to which the sample is representative of that population must be addressed. It is appropriate to specify a risk factor which indicates the extent to which the sample is unrepresentative. Commonly, a risk of 1 in 20 chance (sample) is taken, which translates to the ‘95% confidence interval’.
- Qualitative and graphical descriptions of the data are useful in developing an understanding of the survey results. Many readers find tables difficult to comprehend. A written summary of the salient points in a table, and graphs and diagrams displaying table contents, make a report easier to understand and less prone to misinterpretation.
- One successful course is to prepare three types of reports: a preliminary report, a summary report and a technical report. The preliminary report contains the primary results and should be released early to enable those who wish to use the data immediately to do so. The summary report specifies the objectives of the traffic study, the general method used, the major analytical results and the interpretation of the results. The technical report documents the survey and provides complete details of the results.

In general, a traffic study report should include: summary, introduction, study objective and hypothesis, experimental design, study procedure and statistical results. The results should include descriptive statistics such as tables, graphs, histograms, pie-charts, box plots, stem-and-leaf plots, which are readily available from computer software. Standard statistical test results should be clearly stated, including the strength of relationships between variables and relevant confidence intervals. The report should outline the statistical methods used, present and discuss the results, and draw conclusions. Data listings can be included in appendices where appropriate.

2.2.10 Archiving

The effective archiving of data and details of the survey procedure is a means of ensuring that the data will be available for secondary analysis at a future date.

This important task involves documentation of the survey technique, storage of the data with sufficient information to permit easy retrieval, indication of where the data are stored, and nomination of those who have continuing responsibility for maintaining the database.

In summary, the systems approach to the design and management of traffic studies provides a formal structure for indicating the importance of good practice in study planning, design, execution and data analysis. There are significant linkages between these components, and the systems approach offers valuable insights into the traffic study process. The needs for reliable and representative traffic data are so great, and data collection is so expensive that ad hoc and unrelated survey procedures should be avoided.

2.3 Statistical Methods and Sampling

2.3.1 Statistical Methods

The use of statistical methods is an important component of traffic analysis. Statistical procedures can be divided into three general groups:

- data reduction – provides an initial look at the data, and involves procedures for descriptive statistics, which describe the nature of the observed data
- hypothesis testing – uses the theories of statistical inference to estimate the nature of the population from the sample data
- relationship determination – examines the relationships between observed variables, checks the quality and does preliminary exploratory analysis to confirm if survey data is sufficient.

A full description of each of these groups is beyond the scope of *Part 3*. Details are available in statistics textbooks. Where some of the procedures from these groupings are required in this Part, a brief account is given in the section for the particular type of traffic study.

2.3.2 Sampling Methods

Almost all traffic data collection is based on sample surveys. This section indicates how to select a sample.

A prerequisite for good sample design is that there is a clearly defined objective for the study. Otherwise, it can be too easy to collect too much data, which is expensive and wastes resources; too little data results in inconclusive, incomplete answers; the wrong data provides useless results and wastes resources.

The following main elements of sample design may be identified:

- definition of target population
- definition of sampling unit
- selection of sampling frame
- choice of sampling method
- consideration of likely sampling errors and biases
- determination of sample size.

Target population

Changes to a traffic system are often aimed at a particular class of traffic or road user, and their performance needs to be assessed with respect to that class or group. Seldom will changes have equal effects on all road user groups. This may bring complications when trading-off positive effects for some groups (e.g. users of radial arterial roads) against negative effects for others (e.g. cross-town travellers), when assessing the impacts of, say, signal coordination systems. The particular subgroup of road users or sites that the survey is intended to cover is the target population. Clear definition of the target population will subsequently reduce the level of uncertainty associated with decision making, and lead to optimisation of the resources needed to complete the survey.

The population being sampled must be representative of the target population. The survey might include the collection of supplementary information to determine the nature of the differences between the sample and target populations.

Sampling unit

A population consists of individual elements, e.g. individual vehicles. In one study, the focus may be on individual vehicles, whereas in another study the focus may be on types of vehicles. The definition of a sampling unit therefore depends on the nature and purpose of the study, and is constrained by the efficiency, effectiveness and productivity (of survey personnel) in collecting the required data.

Examples of a sampling unit include individuals, vehicles, households, car parks, and road links.

Sampling frame

The sampling frame is the 'register' of the target population. It is the list or definition which identifies the members (units) of the target population. A proper specification of the sampling frame is required for sampling to proceed, and for any inferences to be made about the target population from the sample results.

The choice of the sampling frame depends on the needs and constraints of the particular study. For instance, the sampling frame might consist of the list of vehicles registered as being garaged in a certain area, or it could be all those vehicles using a particular road or junction during a specified period. The main requirement is that the sampling frame can be quantifiably identified so that the size of the target population may be determined.

Sampling method

The methods for selecting samples from a target population include random sampling and judgement sampling. In random sampling, all members of the target population have a chance of being selected in the sample, whereas judgement sampling uses personal knowledge, expertise and opinion to identify sample members.

Judgement samples avoid the statistical analysis necessary for random samples (indeed it is wrong to apply statistical analysis to them), and have a certain convenience. There is a particular role for judgement sampling in exploratory or pilot surveys where the intention is to examine the possible extremes of outcomes with minimal resources. However, judgement samples have no statistical meaning. They cannot represent the target population and the results are almost certainly biased.

A random sampling scheme should be used to ensure that the sample taken is statistically representative. Random samples may be taken by basic sampling methods such as:

- simple random sampling
- systematic sampling
- stratified random sampling
- cluster sampling.

Simple random sampling is the most commonly used method. A sample is selected by some method that allows each possible sample to have an equal probability of being chosen, and each unit in the target population has an equal probability of being included in any one sample. Sampling may be either with replacement (i.e. any member may be selected more than once in any sample draw) or without replacement (i.e. after selection in one sample, that unit is removed from the sampling frame for the remainder of the draw for that sample). Selection of that sample is by way of a number assigned to each unit in the sampling frame, with repeated random digits being the most convenient means of drawing a random sample. Random-number tables are contained in statistical textbooks, or may be generated by computer.

Systematic sampling, stratified random sampling and cluster sampling all attempt to approximate simple random sampling. They have been developed for their precision, economy or ease of application. However, it is the principle of simple random sampling that lies behind the mathematical theories of statistical inference, i.e. the process of drawing inferences about a population from information on samples drawn from that population.

For a systematic sampling in two dimensions, the Latin square design should be used. This design can minimise biases (Miller et al. 2010) and is a balanced experimental design that aims to control extraneous sources of variability while investigating the impact of different treatments. It adjusts for these sources of variability by systematically blocking in two directions, so that there are two restrictions on the rows and columns of the square.

For example, let the treatment be three methods of counting traffic, and the two factors affecting traffic counting accuracy be field staff and equipment type. A Latin square design requires equal numbers of field operators and equipment types. Let there be three field operators and three equipment types. Then the Latin square design is a three-by-three matrix (Table 2.1) with each method of counting traffic being used once by each operator using each equipment type. Each counting method appears once and only once in each row and in each column of the matrix.

Table 2.1: An example of a 3x3 Latin square design

	Field staff A	Field staff B	Field staff C
Equipment type I	Method 1	Method 2	Method 3
Equipment type II	Method 2	Method 3	Method 1
Equipment type III	Method 3	Method 1	Method 2

Applications of the Latin square design in traffic surveys were reported for example in Johnston et al. (1982) (Appendix C) and Best and Rayner (2011).

2.3.3 Sampling Error and Bias

Nearly all data observations form samples from some larger population. There is a possibility that the sample does not adequately reflect the nature of the parent population. The random fluctuations that are inherent in the sampling process can be examined using the methods of statistical inference. There may also be biases (systematic errors) in sample data, and these may distort an analysis that assumes all errors are random by nature. For example, a traffic management measure that produces unequal benefits for different road user groups may cause the proportions of those groups to alter in the area influenced by the measure, as road users adjust to the new system. The proper treatment of samples is therefore important if biases and variations in the unseen section of the population are not to invalidate an analysis.

The means for overcoming systematic errors lie in proper experimental design and survey planning, as described in Section 2.2.

All possible means of samples, drawn from a target population, have a distribution that is different from the target population. The mean of the sample means is equal to the mean of the population. However, the standard deviation of the sample means, which is termed the ‘standard error’, is not equal to the standard deviation of the population. The standard error is important in the selection of a sample size. Standard error indicates:

- the size of the error in the estimation of the true mean (or proportion) due to random variations in extracting sample statistics
- the accuracy of estimation of a population parameter from the sample statistic.

2.3.4 Sample Size Determination

Traffic surveys for specific investigations attempt to provide data for the estimation of particular population parameters, or to test statistical hypotheses about a population. In either case the size of the sample selected will be an important element, and the reliability of the estimate will increase as sample size increases. On the other hand, the cost of gathering data will also increase with increased sample size.

A trade-off may occur, and the additional returns from an increase in sample size will need to be evaluated against the additional costs incurred. If the target population is infinite (i.e. very large compared to the sample size), then the standard error S_X of the mean of the variable x is given by Equation 1:

$$S_X = \frac{\sigma}{\sqrt{n}} \tag{1}$$

where

σ = population standard deviation

n = sample size.

This equation implies that as sample size increases, standard error decreases in proportion to the square root of n. To double the precision of an estimate will require the collection of four times as much data. Consequently, there is value in seeking an optimum sample size. Note that it is the absolute size of the sample that determines sampling precision, rather than the fraction of the target population.

These results provide preliminary information on how to estimate a minimum sample size. For example, assume that the aim is to estimate a mean speed at a site, with an accuracy of 1 km/h or a standard error of 0.5 km/h. At the 95% confidence level, the standard error (s.e.) is about half the value of accuracy required. If it is known that the sample standard deviation (s.d.) of the speed distribution is 6.7 km/h (e.g. from a pilot survey), then by applying the above equation, or using Table 2.2. The minimal sample size is given by (n) in Equation 2:

$$\text{n} = \left(\frac{\text{s.d.}}{\text{s.e.}}\right)^2 = \left(\frac{2 \times \text{s.d.}}{\text{accuracy}}\right)^2 = \left(\frac{2 \times 6.7}{1}\right)^2 = 180 \text{ (rounding up 179.6)} \tag{2}$$

Table 2.2: Relative error

Relative error = $\frac{\text{s.d.}}{\text{s.e.}}$	Sample size n
20	25
15	44
12.5	64
10	100
7.5	178
5	400
2.5	1 608
2	2 500
1	10 000

The accuracy and standard deviation can also be specified as a percentage of the mean. For example, the accuracy required is 5% of the mean and the standard deviation is estimated to be 20% of the mean (i.e. the coefficient of variation CoV = 20%). The sample size requirement is therefore $(2 \times 20\% / 5\%)^2 = 64$ at the 95% confidence level.

In a before-and-after traffic study, traffic data (e.g. travel times) are often measured before-and-after a traffic management measure is implemented or trialled. Let the difference between the mean values of the before-and-after situations be d% = 5%, and the before-and-after standard deviations are the same and equal to 20%. An approximate indication of the sample size requirement for each situation is as Equation 3 (Ferreira & Dandy 1984):

$$n = 2 \times \left(\frac{2 \times CoV}{d\%} \right)^2 = 2 \times \left(\frac{2 \times 20\%}{5\%} \right)^2 = 128 \qquad 3$$

Hence, 128 samples should be used for the ‘before’ survey, and another 128 samples for the ‘after’ survey.

Issues relating to sample sizes are of particular concern in the conduct of speed surveys, and further discussion of sample size determination is given in Appendix B.

2.4 Data Integrity

The objective of data integrity is to provide accurate data in a user-friendly format on demand by a wide range of users. The data should contain a statistically valid sample drawn from a population. It should be recorded accurately in a consistent and convenient format and organised for easy access, transfer and reporting. Limitations include equipment failure, non-representativeness of data, and problems associated with processing and editing. Although any surveyed data would have limitations, effort is required to ensure that the quality of data is acceptable at the end of a chain of activities at each stage of which compromises would have been made.

Data procedures and travel time survey management have been through the process of quality certification to *AS/NZS ISO 9001* on requirements in quality systems for design, development, production, installation and servicing. Overseas guidelines such as the US Federal Highway Administration’s (FHWA) *Traffic Monitoring Guide* (FHWA 2016) have also promoted good data collection practice.

It is generally challenging to achieve consistency in the data collected within a state/territory road agency, across road agencies and across all jurisdictions. This is partly due to differences in road use data requirements and the methods for collection, processing and reporting. The data quality issues that need to be addressed include:

- accuracy
- timeliness
- accessibility
- completeness and relevance
- comparability and compatibility.

Other issues of concern are:

- methodologies
- reporting, publication and feedback
- provision of analytical tools
- limitations in equipment, resource and field options
- collection, processing and distribution costs.

The availability of data from intelligent transport systems (ITS) and other sources would require better quality control at each source before all data sources could be shared, comprehended and coordinated. For example, data from a freeway incident management system, digital speed cameras or a signal control system are valuable sources of useful information. However, the limited resources of the freeway and signal operational units are often employed for operational improvements rather than analysing these data to become useful traffic statistics. Compatibility of these data with existing vehicle classification systems is another issue that should be noted.

It is worth repeating that traffic phenomena often arise by chance from the combinations of many factors, all of which have high degrees of variability. Environmental factors such as rain and magnetic fields also affect equipment accuracy. It is therefore necessary to treat events in traffic systems as random variables, and use statistical theory to investigate the impacts of the interacting factors. Unusual variability within traffic statistics derived from one survey station should be verified if possible from surrounding survey stations or previous data at the same station. Although equipment failure accounts for some errors, other errors arise because of the occurrence of abnormal traffic events in the connecting road network. The success of a statistical investigation depends largely on the size and validity of the traffic survey used to collect the data.

2.5 Traffic Surveys

Traffic surveys have long been a major part of the traffic engineer's work. Most traffic control and design problems demand some knowledge of the operating characteristics of the traffic concerned.

The most common types of traffic surveys are overviewed in Sections 2.5.1 to 2.5.9, the details of which can be found in the various appendices as indicated. These surveys are as follows:

- traffic volume surveys (Appendix A)
- speed surveys (Appendix B)
- travel time, queuing and delay surveys (Appendix C)
- origin-destination surveys (Appendix D)
- pedestrian and cyclist surveys (Appendix E)
- noise, fuel and emission surveys (Appendix F)
- vehicle mass and dimension surveys (Appendix G)
- parking surveys (Appendix H)
- traffic generation surveys (Appendix I).

These surveys range from 'point-based' studies that involve the measurement and analysis of data collected at a point in space (e.g. traffic volume and spot speed) to 'link-based' studies that involve measurement of a parameter (e.g. travel time) over a road link, and to 'area-based' studies that involve data collection over an area (e.g. for origin-destination studies, or trips generated by land use developments).

Part 3 does not directly cover the various types of social science surveys used elsewhere in transport and traffic planning, such as questionnaires and household interviews relating to travel behaviour and choice.

The traffic studies described in *Part 3* relate fundamentally to longitudinal movement in the direction of travel. It is also possible to study lateral movements efficiently and accurately with new technologies such as image processing, laser beams, and light tracking. As pointed out in Gunay (2003), lateral movement studies are of more importance in countries where lane discipline is 'less tidy', especially for understanding the capacity utilisation of a lane or an approach. The issue of lateral movement is not addressed further in *Part 3*.

2.5.1 Traffic Volume Surveys (Appendix A)

Traffic volume data may be used for facility planning and design, to monitor trends in use, to assist in highway classification and budgeting or to validate predictions from traffic forecasting models. Traffic count data have traditionally been the most important observations for inferences about the state of a traffic system. However, vehicle classification data are increasingly important in shaping the information for all phases of the planning, design and operation of traffic networks.

The period over which a count is recorded is an important consideration in assessing the load borne by a traffic facility. Consequently, traffic count information should always be expressed as a flow rate, i.e. the number of traffic units per unit time. For road traffic, the units are vehicles per hour (veh/h) or vehicles per day (veh/day), sometimes with the type of vehicle more precisely defined, e.g. passenger cars per hour (pc/h), passenger car units per hour (pcu/h), axles or axle-pairs (passenger car equivalents) per hour, trucks or heavy vehicles of different types per hour (trucks per hour), or cyclists per hour. Traffic volumes also need to consider people movement (passenger occupancy). Consideration needs to be given to high-occupancy vehicles (HOV), buses and transit lanes.

Traffic flow rates (commonly called volumes) are used to establish the following:

- relative importance and role of a road in a traffic system
- variations in the levels of traffic flow over time
- extent of the use of a facility in terms of its capacity to carry traffic
- distribution of travel demand in a network
- coordination of traffic signals
- estimating the loading on pavements and bridges from classified vehicle counts in the absence of weigh-in-motion (WIM) equipment.
- road geometric design standards and guidelines, e.g. clear zones, lane widths, auxiliary left turn treatment / channelised left turn treatment and channelised right turn treatment requirements etc.

Traffic count data are important in economic evaluation, environmental analysis (e.g. urban air quality) and as an exposure measure in road crash studies. Further uses of traffic count data are shown in Table A 1 of Appendix A.

2.5.2 Speed Surveys (Appendix B)

Speed is an important road design parameter. Knowledge of vehicle speeds in a traffic system provides useful information about travel conditions, levels of service, and quality of traffic flow. Data on vehicle speeds are necessary for both setting design standards, and as a measure of the effects of changes to a traffic system such as installing a traffic control device or widening a curve.

Speed data can be used in the context of setting speed limits, establishing the need and effectiveness of traffic control devices, examining road safety issues to identify crash countermeasures, and for establishing trends in vehicular speeds. Speed data can be collected directly or indirectly.

The most common form of direct measurement is by radar, infrared and lidar (an optical remote sensing technology using laser light pulses rather than radio waves as in radar). Indirect speed measurement commonly involves measuring travel time for vehicles between two detectors separated by a known distance. The travel times may be measured manually or electronically.

Knowledge of speed data is used extensively in road safety programs. Spot speed data are useful for traffic engineers in the study of driver behaviour as they provide estimates of the range of likely vehicle speeds. The prevailing distribution of speeds at a site under different environmental conditions can also be determined. Speed distributions provide indications of traffic conditions at the observation site and are useful in assessing the need for appropriate traffic control devices, speed limits or advisory speed signing. They are also useful aids in studies concerning overtaking manoeuvres and the effects of lane widths and lateral clearances. Before-and-after speed studies can help in assessing driver responses to new regulatory and warning signs, road markings, street lighting and pavement surfaces.

The more useful applications of speed survey data are:

- determining the need for traffic control devices, including speed zoning
- evaluating the effectiveness of traffic improvements, in before-and-after studies
- assisting in the design of roadways or intersections
- finding relationships between speeds and crashes or between speeds and geometric features
- undertaking an economic analysis
- determining the range and magnitude of speeds as a basis for formulating design standards.

2.5.3 Travel Time, Delay and Queuing Surveys (Appendix C)

Travel time has been identified by Austroads as an important system performance measure and regular travel time surveys are now conducted by road agencies. The conventional way of collecting travel time data is having observers recording data as they travel over the road network. Some organisations have developed instrumented vehicles for conducting travel time surveys and include global positioning systems (GPS) with time and position data.

The use of advanced technology and probe vehicles has become increasingly common in recent years for the automatic collection of travel time data. These technologies include GPS-equipped or tagged vehicles, Bluetooth technologies, automatic number plate recognition (ANPR) and mobile phone tracking (see Section 2.6 and Appendix K on emerging and alternative technologies for travel time measurements).

A travel time and delay study measures the average travel and running times along sections of a route while at the same time collecting information on the location, duration and cause of delays. A delay study measures stopping time delay at specific points such as intersections or railway level crossings. From the measured times, average travel times and running speeds can be calculated.

Another relevant measure is queuing or queue length. A vehicle is in a queue when it is controlled in its actions by the vehicle in front of it or has been stopped by a component of the traffic system. Queues can, therefore, occur in traffic moving along the open road as well as at constrictions in the traffic system. Bunching of vehicles is, by the above definition, a queue. Queue length and delay measures are quite related and often can be estimated from each other for traffic signals and other traffic constraints.

Travel time, delay and queuing data are often used to:

- assess the quality of the traffic route
- evaluate the before-and-after effects of traffic improvements
- undertake economic analysis
- identify locations and causes of congestion
- determine needs for traffic signals
- determine the storage requirements
- develop optimum timing sequences at traffic signals.

Travel time, delay and queuing data are often sought as performance measures of some part of a transport network. Several other performance indicators are available, and these may be more relevant for some investigations. They include:

- variability in travel times and delays
- delays to pedestrians
- total person delay
- number of vehicular stops
- fuel consumption
- vehicle emissions
- vehicle headways
- gap acceptance.

Note that while most of the technical literature refers to 'gap acceptance', the procedure involved is actually headway acceptance. Refer to Section 6 for further detail on headway and gap acceptance, and Austroads (2015a) for the underlying theory.

The specific objectives of any study will provide guidance as to the appropriate performance indicators and the corresponding survey methods to collect the required data.

2.5.4 Origin-destination Surveys (Appendix D)

Origin-destination surveys provide valuable information on where motorists desire to travel, i.e. the origins, destinations and travel times, as well as volumes, in conjunction with traffic surveys. They range from relatively simple studies that determine the amount of traffic bypassing a small country town, to comprehensive transportation surveys that are used in the planning and design of transport systems in large metropolitan areas. Data on trip patterns (trip distribution) are essential in situations where diversion of traffic may result from a new traffic management scheme and it is difficult to locate traffic count equipment at the point where diversion occurs, whether this be at the local, regional or metropolitan level.

There are several techniques used for gathering origin-destination data. Some are used more commonly than others, but each has its advantages and disadvantages. Origin-destination surveys are liable to significant errors and biases and therefore a good understanding of the techniques and their inherent problems is required to produce useful data. Origin-destination surveys are usually labour-intensive and costly to undertake. Observations under conditions of darkness can be difficult, and care is needed to avoid wasting resources.

As mentioned, there have been significant improvements in imaging technologies to automatically recognise numberplates and Bluetooth technologies for travel time measurements. These technologies can also be used for origin-destination studies (Appendix D and Appendix K).

2.5.5 Pedestrian and Bicycle Surveys (Appendix E)

The provision of facilities for pedestrians and cyclists has been steadily increasing due to an increased focus on user needs and safety. Data on some of the movements made by pedestrians and cyclists can be collected using methods similar to those described in other sections on collecting traffic data. The nature of pedestrian and bicycle movements, however, is not as restricted to specific roadways as that of vehicles, hence the greater difficulty in collecting information. Bicycles are defined as vehicles under road traffic regulations and therefore have a right to use virtually all roads.

In recent years there have been significant developments in policy and strategic planning initiatives aimed at giving greater recognition to walking and cycling activity in the transport sector. This has arisen from policy settings in the transport and health sectors recognising the need to move towards more sustainable forms of transport (by foot, bicycle or public transport) and towards healthier activity (walking, cycling) by the community generally.

The development of strategies and plans have typically involved a multi-agency (health, planning, transport) approach to creation of an ‘active living’ vision. Benefits from the implementation of these strategies are envisaged from health, social, economic, environmental and transport perspectives.

Much work has been done internationally, particularly in developed countries, aimed at understanding, measuring and providing for walking generally. It is recognised that walking is not just a transport mode – it is also a recreational activity – and there is a need to reflect this in related measurement and survey techniques.

Studying pedestrian and bicycle movements may also be complicated by the spatial distribution of routes they can choose. For example, they can easily reverse their direction of travel and exit a system where they enter. The main similarities between vehicles and pedestrians or cyclists occur when pedestrians or cyclists are constrained to a footpath, road lane or corridor, as this situation is similar to vehicles on a road.

Any study of pedestrian or cyclist behaviour requires a clear statement of the problem to be addressed and a statement of the objectives of the study. This statement should lead to a set of parameters to be measured by the study. Harbutt and Richardson (2000) recommends that base data be collected in study areas that are consistent with the geographic areas used by the Australian Bureau of Statistics, so as to ensure consistency with population characteristics.

The majority of data collected in pedestrian and bicycle surveys will come from sample surveys. When deciding on the size of the sample, it is necessary to consider confidence limits, levels of confidence and inherent variability (Appendix A.2). A trade-off exists between the required accuracy of the sample, and therefore the size of the sample, and the cost of the study.

The sampling of cyclists is difficult because information on bicycle ownership is rarely available. The concentration on particular groups such as school children or bicycle clubs will also not provide information on all bicycle users. Interviewing in the field may provide an overall idea of travel characteristics but survey locations need to be selected carefully and in a random manner to ensure a broad spectrum of cyclists is interviewed.

An area of concern in the study of pedestrian and bicycle movements is the unit of measurement. A study of pedestrian flow may use the pedestrian as the unit of measurement. If the pedestrian units are not independent, as in the case of a parent with a child, it may be necessary to divide the pedestrians into a number of categories. Another factor that needs to be taken into account occurs in pedestrian congestion studies. When defining the amount of space occupied by each pedestrian, pedestrians with a disability and pedestrians with prams and shopping trolleys need to be considered. These ‘pedestrian modules’ are larger than an average pedestrian and can affect the final results. Pedestrian trip purposes may also be an important issue for the planning of pedestrian facilities.

Various ongoing household travel surveys exist, and useful data on bicycle and pedestrian trips can be obtained from them. They include the Victorian Integrated Survey of Travel and Activity (Transport for Victoria 2016), the Sydney Household Travel Survey (Transport for NSW 2014) and New Zealand Household Travel Survey (Minister of Transport 2015). The surveys have recorded daily travel patterns, including bicycle and walking trips, of household members in the survey areas. Other databases such as the Bicycle Imports of the Bicycle Industries and Traders Association and the Serious Injury Database of the Australian Transport Safety Bureau also provide useful bicycling and pedestrian data.

When using existing information, it is necessary to consider the original purpose of the data, the represented population (e.g. were children under 10 included?), the treatment of multi-mode trips and the sampling techniques used.

2.5.6 Noise, Fuel and Emission Surveys (Appendix F)

Fuel consumption and environmental degradation are of special importance in traffic studies and in the evaluation of traffic plans and management schemes.

Road traffic systems can have significant impacts on:

- noise and vibrations
- fuel consumption and means of conserving liquid fuels
- air pollution, including gases and particulates from vehicle emissions.

Pollution problems may be considered at two levels: immediate and prolonged exposure. In extreme cases, the pollutant may be a danger to the physical health and well-being of the individuals subjected to it. Excessive noise and concentrations of some air pollutants such as carbon monoxide (CO) may inflict immediate damage. Prolonged exposure to large concentrations of other pollutants, or smaller doses of noise and CO, may lead to harmful effects. The excessive levels referred to above are rarely if ever solely or even largely attributable to road traffic. However, individuals may become distressed or anxious in the presence of pollutants, at levels well below those hazardous to health.

Appendix F considers some of the survey management and data collection techniques that the traffic engineer will encounter. Very often the traffic engineer will not be directly involved in the data collection and analysis in these specialist areas. This will be the province of another authority, and may require the expertise of another discipline, such as mechanical engineering, environmental engineering or meteorology. Traffic engineers will be contributing their expertise in traffic systems to a multi-disciplinary study team. They should have some familiarity with the approaches adopted by other team members, so that their work can be properly and successfully related to a traffic impact study.

2.5.7 Vehicle Mass and Dimension Surveys (Appendix G)

Information on vehicle mass and dimensions is necessary for effective management of road transport networks. Relevant data has traditionally been obtained by diverting a sample or all heavy vehicles into a specially equipped station in which each axle and/or the gross vehicle mass is measured using a weighbridge or other special weighing equipment. The vehicle dimensions can be measured manually. Vehicle mass is also regularly measured on roadside or commercial weighbridges for regulatory and enforcement purposes. These methods, particularly the latter, may involve bias because overloaded vehicles alerted via mobile phones or CB radio may avoid routes on which a survey is being made. The development of WIM technology has meant vehicle mass data is now more easily obtainable. The CULWAY systems described in Appendix G.3.2, in particular, are expected to provide unbiased estimates of vehicle or axle loads because they are installed in culverts and are not readily detected by passing drivers. Appendix A.5 describes vehicle classification systems by vehicle characteristics (such as vehicle types, lengths or axle configurations) which are related to vehicle mass.

2.5.8 Parking Surveys (Appendix H)

Parking plays an important role in the provision of road transport services, allowing drivers to store their vehicles at the origins and destinations of their journey. Parking can be of two main types, on-street (kerbside, median) or off-street (residential, commercial).

Well-planned on-street parking facilities for cars can provide valuable short-stay parking space. However plentiful on-street parking or complicated parking restrictions can lead to significant time being spent searching for parking spaces adding to traffic circulation and disrupting traffic flow. As a general rule the regulation and use of on-street parking should be prioritised to support those road users with needs for high levels of access such as pedestrians, public transport, taxi operators, car share couriers and service vehicle users, people with disabilities and emergency services.

Collecting and analysing data on parking characteristics forms the basis of the design process for parking systems, and assists in managing potential effects upon traffic operations. The demand for parking is generally related to the land use served. Valuable information may be needed on the demand for parking, the supply of parking spaces, and the use of existing facilities. Typical data collected include:

- spatial distribution of parking demand
- types and extent of parking requirements
- numbers, type and location of parking facilities
- parking restrictions and costs
- duration and turnover of parking activities.

Depending on the type of information required, data collection can be undertaken by interview surveys (including reply-paid questionnaires) or observational surveys (including both cordon counts and patrol surveys). Technologies are available for conducting observational surveys by video with automatic data logging facilities, electronic tagging or via new ITS technologies such as electronic parking guidance systems and signs.

Part 11 of the Guide presents advice and guidance on the parking management process, in terms of parking policy, demand and supply, parking control and management systems (Austroads 2017c).

2.5.9 Traffic Generation Surveys (Appendix I)

An important aspect of the planning and design of land use developments is the provision of adequate traffic access. These developments include shopping centres, sports and recreation centres, office blocks and multi-unit residential developments. Besides the immediate connections of the development to the road system, traffic planners and engineers are concerned with the effects of the new development on the traffic system in the surrounding area. A fundamental issue is the amount of new traffic generated by a development. This traffic may place an additional load on the traffic system, and could result in the need for upgrading parts of the road network or provision of new traffic facilities. It is worth noting that redevelopment of a site may reduce the number of trip ends within the traffic system (e.g. changing the land use from a high traffic generating site to a lower traffic generating site).

Traffic generation is the measured level of traffic activity associated with a site, development or land use. It is the amount of vehicular traffic arising from the number of person-trips associated with a development. Traffic generation is usually measured in terms of the number of trip ends (the total of trip production and trip attraction) at a site, per unit time. Data collection is by means of observational or questionnaire surveys.

Part 12 of the Guide is concerned with identifying and managing the impacts upon the road system arising from land use developments (Austroads 2016c). It provides guidance on the need and criteria for assessing the traffic impacts of those developments, and a detailed procedure for identifying and assessing the impacts, and mitigating their effects.

2.6 New and Alternative Data Sources

2.6.1 Use of New and Alternative Data Sources

Conventional traffic surveys usually use stationary, fixed-point measurement devices distributed at selected locations across the road network (the exceptions being mobile floating car surveys, which have traditionally been used to measure route-level traffic speeds and travel times). The conventional data collection mechanisms have been developed by road agencies over many decades as the most cost-effective sources of data to manage operations and plan for future network needs. There is generally a trade-off between collection cost, network coverage and data accuracy.

New and alternative traffic data sources which include Bluetooth technologies, GPS-based probes, cellular mobile phones and more, are becoming increasingly available in providing more data sources for traffic studies (Appendix K). These data could be collected by road agencies or private organisations and some of them are commercially available on the market, which could be originally collected for other purposes (e.g. location-based services for mobile phone users etc.). These technologies and systems can potentially provide both historic and real-time measures of traffic and road operating conditions across large parts of the road network at relatively lower cost.

The Bureau of Infrastructure, Transport and Regional Economics (BITRE) new traffic data workshop in 2014 (GHD 2014) reported that the possible benefits and opportunities of utilising emerging technologies and new data sources for traffic studies include:

- They could provide great geographic coverage at local, regional and national levels.
- They could provide information for 24 hours in temporal coverage and include weekends and holidays, which conventional survey methods do not usually cover.
- They offer easier acquisition of long-term continuous traffic data (high resolution and frequent intervals) with little to no interaction with subjects.
- Traffic data from many technologies e.g. probes and Bluetooth have demonstrated increased accuracy and detail with the rapid upgrade of technologies.
- Cost per unit of useful information in comparison with traditional methods could be much lower. These emerging technologies are mostly non-intrusive, and some technologies rely on existing mobile devices that do not require any additional costs for installation and maintenance.
- Currency of data collection and processing enables real-time and near real-time traffic data to be increasingly available for transport researchers and operators.
- Data collected for one purpose can often be re-used to answer other questions. Social networks can also be used to collect data (e.g. Waze, a world-wide free community-based traffic and navigation app https://www.waze.com). Some platforms are better or more suitable than others.
- Use of Bluetooth and WiFi sensors to construct local (point-to-point) trip and travel time matrices is common in the USA and Europe. GPS data are often used to deliver real-time speed data and identify congestion. Mobile phone data are used commercially to deliver trip matrices disaggregated by several attributes in the USA e.g. by Airsage, a major US commercial supplier of national-wide location and movement data from mobile devices (www.airsage.com).
- The new data sources are better seen as a complement to existing data sources. However, it would be possible to use traditional methods more selectively. For example, availability of GPS probe data makes traditional floating car surveys less attractive.

However, it is acknowledged that the use of new data sources in Australia and New Zealand is still under investigation and the limitations on their current applications may include:

- limited sample size on low-volume roads for many emerging technologies (Bluetooth, GPS probes etc.)
- some technologies still rely on roadside equipment (portable or permanent) such as Bluetooth
- limited applicability to measuring total traffic volumes, as they will not be present or captured in all vehicles and so penetration rates (or sample size) have to be estimated
- historic data may or may not be available depending on the time the technology was adopted
- potential sample bias due to reliance on fleet vehicles or toward drivers that are more likely to have enabling devices
- concerns regarding locational privacy may limit the access and use of raw data.

It is also suggested that comprehensive data specifications be investigated before the use of these emerging and alternative data sources.

Appendix K further discusses the strengths and weaknesses of the emerging and alternative technologies in details. While each data source has its own strengths and weaknesses, Table 2.3 shows some specifications for alternative data collection methods, both conventional and new, in terms of coverage, cost, delay, intrusion and other features.

Table 2.3: Illustration of alternative data source features

	Household survey	Intersection roadside interview	ANPR	Bluetooth WiFi	Mobile phone	GPS probes
Sample size	1-3%	10-25% local	100% local	10-30% local	20-50%	5-15%
Data collected	Trips, tours, vehicle ownership, journey purpose, mode, OD etc.	OD purpose, (vehicle ownership)	Local OD, travel times	Local OD, travel times	OD, travel times, journey purpose, mode	Travel times, OD
Time coverage	1-3 average days	1 average day	1 or more days	1 or more weeks	Any time period, any day	Any time period, any day
Geographic coverage	General as sampled	General as sampled	Local	Local	General	General
OD matrices	Yes	Yes	Entry/exit	Entry/exit	Yes	Yes
Journey purpose	Yes	Yes	No	No	Yes with data fusion	Yes with data fusion
Modes used	Yes	Yes observed	Vehicles only	Possible but difficult	Possible but difficult	Rare
Travel times	No	No	Local	Local	Yes	Yes
Set-up task	Yes	Yes	Yes	Yes	Minimal	Yes
Intrusive	Yes	Yes	No	No	No	No
Delay for analysis	Months	Weeks	Days	0 to 1 week	1 to 4 weeks	0 to 1 week
Collection and processing cost	Very high	High	Medium	Medium	Medium	Low
Vehicle classification	Yes	Yes	Yes	Difficult	Difficult	Yes
Limitations	Access, underreporting, simplification	Coverage, disruption, refusal, simplification	Masking, local OD, vehicles only	Correct sampling, mode, local OD	Access, correct sampling, mode	Correct sampling, mostly vehicles
Bias	Can be corrected	Can be corrected	Not significant	Mostly unknown	Correctable	Mostly unknown
Data fusion with	Census, GIS, counts	Census, GIS, counts	Counts, GIS	Counts, GIS	Census, GIS, counts	Census
Privacy concerns	Some	Low	Some	Medium	Medium	Low

Source: Adapted from GHD (2014).

The new and multiple data sources for traffic data collection and network performance monitoring in Australia and New Zealand have been very actively trialled and tested. Some examples of recent and ongoing research and exploration of alternative data sources are as follows:

- Use of probe data for traffic studies by Main Roads Western Australia (MRWA) (Espada, Salt & Li 2014, also in Appendix L.1)
- Network performance analysis for Perth congestion response by MRWA (Bennett, Espada & Weeratunga 2016, also in Appendix L.2)
- Measuring excessive delay and travel time reliability cost using *go card* data for buses along Gympie Road in Queensland, by Queensland Department of Transport and Main Roads (TMR) (Han et al.2016, also in Appendix L.3)
- Post-implementation assessment for Pinch Point Projects, by Roads and Maritime Services (Espada & Inglis 2015, also in Appendix L.4)
- Using probe data and drone for signal performance review, by VicRoads
- Intelligent hybrid prototype to use available data sources for speed/travel time, volume and vehicle classification, by TMR (Geers & Karndacharuk 2016)
- Monitoring Hobart network with probe speed data, by Department of State Growth
- Measuring excessive congestion delay cost and travel time reliability cost by using multiple data sources such as Bluetooth data and probe data, by TMR (Han et al. 2017)
- Understanding the causes of congestion, by TMR (Johnston & Johnston 2016)
- Monitoring Adelaide arterial and freeway networks with rich Bluetooth data and providing real-time traveller information through Traffic SA website and Addinsight smartphone app, by Department of Planning, Transport and Infrastructure (DPTI) South Australia.

Appendix L provides further details of four case studies on the use of multiple data sources for traffic studies and network performance monitoring. Some highlights on the jurisdictional experience in the use of new or alternative data sources are as follows:

- Various studies (e.g. Espada, Salt & Li 2014, Espada & Inglis 2015, Bennett, Espada & Weeratunga 2016) have demonstrated that probe data is a viable alternative to other methods of travel-time measurement or estimation, e.g. sensor-based travel time estimation and floating car surveys. The probe data presents advantages such as a high level of coverage which is not dependent on the presence of sensors, a high level of resolution, and low cost. The growth in the number of probe data points also means that probe data sets will be improving in quality. Further probe data points can provide other useful information such as turning ratios, origin-destination percentage and route preferences.
- Trip and travel time data derived from Bluetooth data can provide an accurate estimation of the daily traffic conditions where congested and uncongested conditions can be clearly identified (Geers & Karndacharuk 2016). The origin-destination data derived from Bluetooth are also shown to compare favourably to both video detection and automated number plate recognition (ANPR) (Blogg et al. 2010).
- Combined with data sources such as Sydney Coordinated Adaptive Traffic System (SCATS) or STREAMS volume and freeway detector station data, these individual data sources could potentially provide a wider range of data and useful insights to inform investment decision making in addressing urban congestion and monitor performance at a network level.

2.6.2 Supporting Techniques and Issues

Compared to the conventional technologies and methodologies, new features and technical requirements in using new and alternative technologies have been identified as follows:

- Additional requirements for data collection, storage and processing

  Probe or mobile data sources could generate large amounts of data and create a corresponding need for data collection and storage that traditional approaches could not handle. With advances in IT technology, various data storage models are possible, ranging from a centralised repository, where disparate data sets are imported and combined, to distributed models, where separate data sets in decentralised and virtual locations (i.e. cloud) and handles structured and unstructured data and combined virtually.

  The data process is often more automated, with computers doing most of the work to find large and complex patterns among a massive number of variables that may not intuitively appear related. Data is often geo-referenced and displayed with high resolution and accurate base maps, at both a micro and macro level.

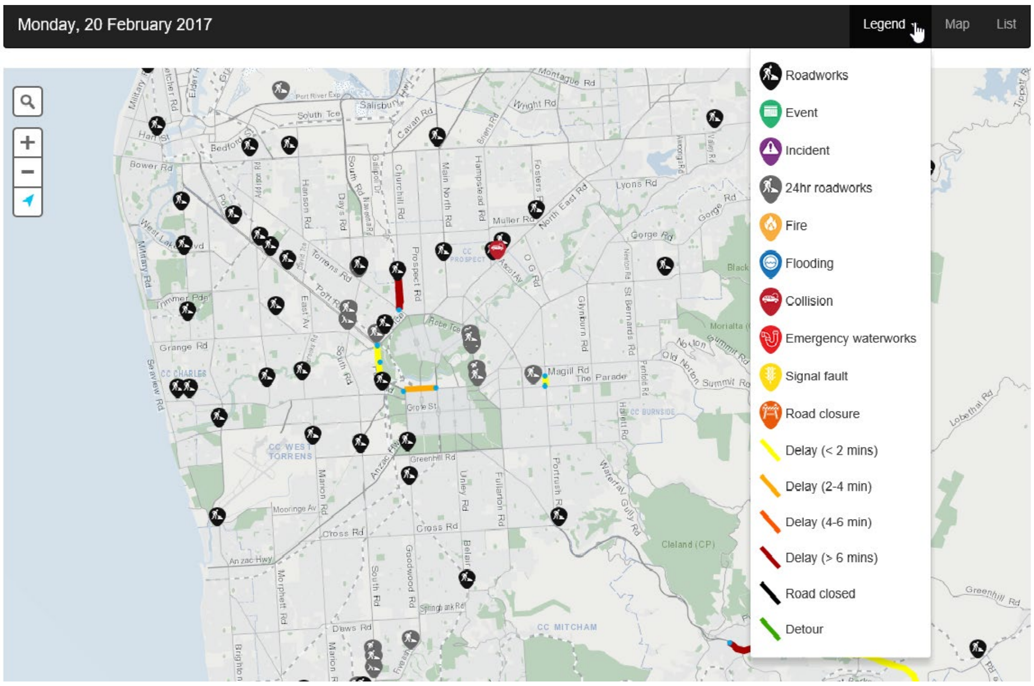
- Advanced data analytics

  Traffic data collected from multiple sources require some degree of transformation (e.g. aggregation, averaging, scaling) and reinterpretation to provide meaningful information for network managers, planners, researchers and travellers. For example, inductive loop data which provides an indication of traffic volumes can be fused with travel-time data derived from probes to give an estimate of overall demand.

  Data analytics is a catch-all term for the mathematical process of discovering useful information from data to support the decision-making process, and includes techniques from data and information fusion, machine learning, data mining, stream processing and high-performance computing. Techniques from the analytics domain will come to the fore in transport operations, planning and modelling as more data from vastly different sources, with different spatial and temporal characteristics become available and needs to be combined (Geers & Karndacharuk 2016).

  DPTI South Australia has integrated data from multiple sources (e.g. Bluetooth, WiFi, RFID and ANPR) and presented them in a GIS map format (Figure 2.2). The real-time information about travel delays, incidents, roadworks, etc. can be easily accessed through both the TrafficSA website and Addinsight smartphone app.

Figure 2.2: A snapshot of TrafficSA website



Source: DPTI (2017).

TMR is investing in traffic data fusion through the intelligent hybrid prototype project being run by the Engineering and Technology Branch.

Globally, several of the larger transport data providers (e.g. INRIX, AirSage) already use various fusion methods to combine GPS, mobile phone data and others with road sensor information. Bachmann et al. (2012) provide a (technical) summary of some of the more common algorithms used to fuse disparate traffic data sources.

- Statistical techniques

  For the use of the large volumes of traffic data, specific statistical techniques would be required to estimate the nature of the population of the collected data, validate the relationships between observed variables, check the quality of the collected data and conduct cluster analysis and identify outliers. For example, to determine if the sample speed data from probe distribution were representative of the reference data population at selected sites, appropriate statistical metrics and tests could be applied to compare the following:

  – the speed distribution (e.g. the Kolmogorov-Smirnov (KS) test, Chi Square test or Mann Whitney Test depending on distribution features of the data)

  – mean speed (e.g. root mean squared difference, R2 – term, t-test)

  – 85th percentile speed (e.g. Analysis of variance (ANOVA) and the Bootstrapping method).

  These tests could be conducted in statistical packages such as Excel or IBM's Statistical Package for Social Sciences (SPSS) or by coding directly into programming packages. Under certain circumstance, conversion factors may also be developed by using the regression techniques to help interpret probe speed data in the same context as traditional spot speed data. The evaluation criteria and acceptance thresholds for the feasibility assessment of new data sources should be developed based on an agency's need for different applications.

- Road network visualisation

  Numerous visualisation tools are available for presenting spatial road network data and fused information from multiple data sources to enhance the utilisation of data results for the decision-making process. Figure 2.3 and Figure 2.4 show some examples of Aperture and Tableau interfaces reported in Bennett, Espada and Weeratunga (2016).

  The fundamental functions required for a network visualisation tool may cover:

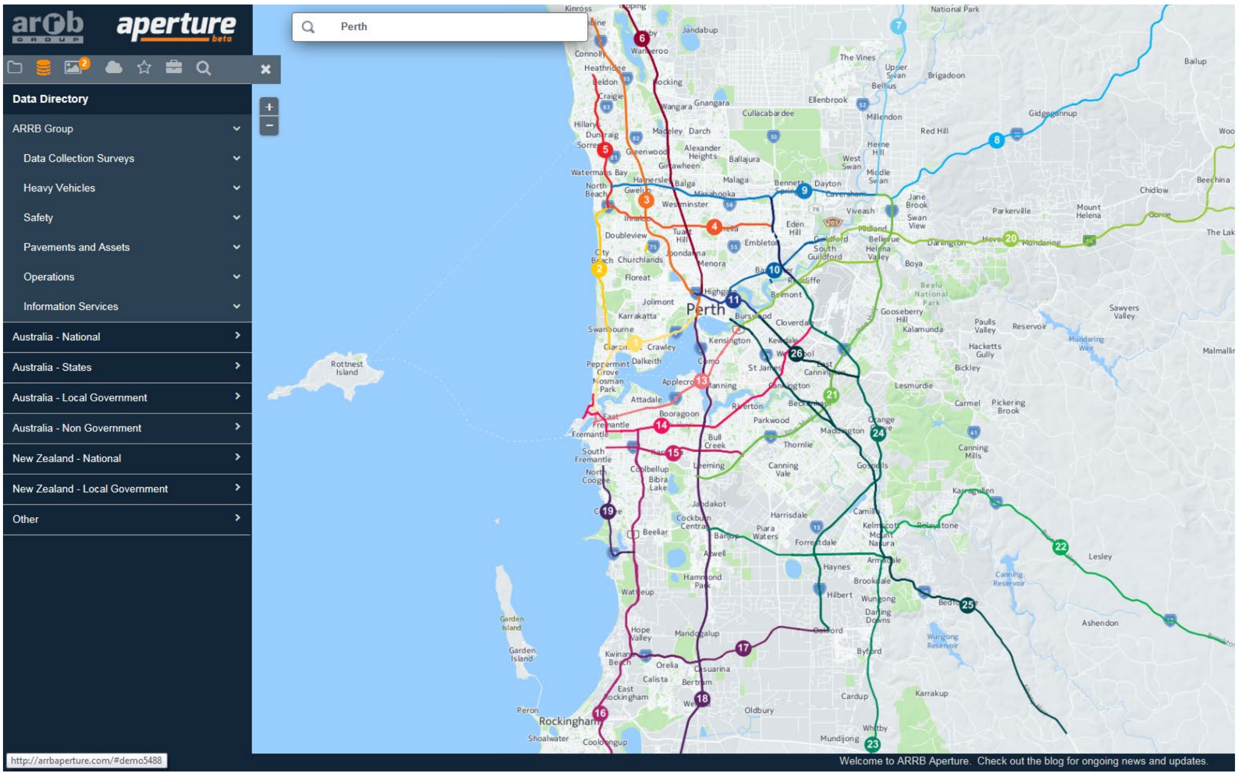
  – uploading and viewing all spatially referenced data

  – moving around the network and zooming in and out at any part of the network at any desired level of detail

  – switching between different base maps and satellite images

  – providing basic statistical reporting interfaces for major key performance indicators (KPIs).

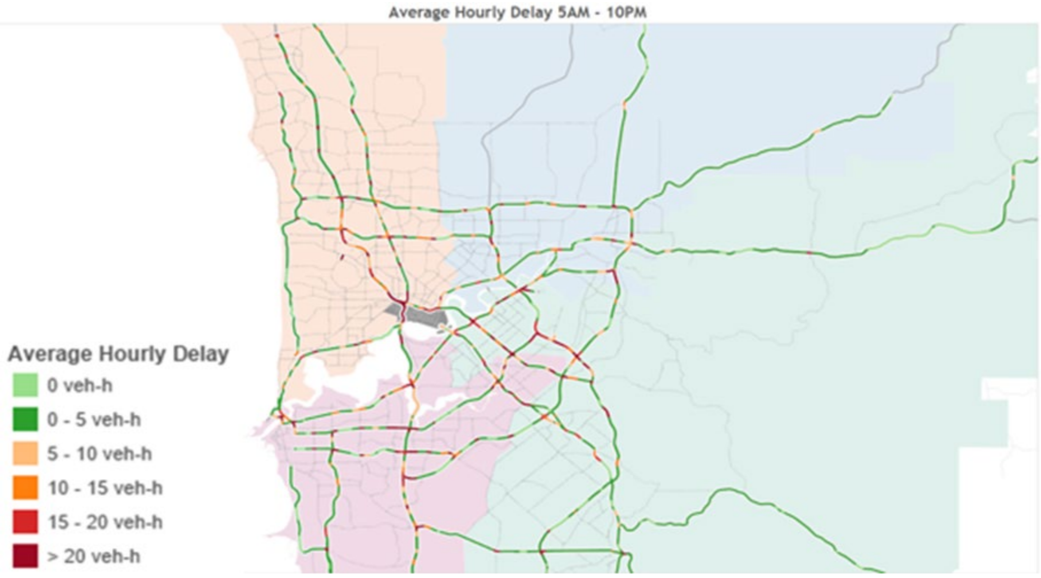
Figure 2.3: Screen shot of Perth in Aperture showing the 26 major corridors over a HERE base map



Note: Aperture is an interactive web-based tool for visualisation of spatial road network data. A selection of the spatial data developed for this project has been uploaded to Aperture including major routes, different levels of congestion delays and delay intensity by color-coded legends for the road network grid.

Source: Bennett, Espada and Weeratunga (2016).

Figure 2.4: Average hourly delay on the 26 corridors showing in Tableau



Source: Urbsol as quoted in Bennett, Espada and Weeratunga (2016).

- Information privacy and security

  Data based on GPS-enabled devices, which are generally registered to an individual, have potential to be mined for personal information. ANPR video images are also potentially subject to privacy concerns.

  The concerns regarding locational privacy due to the use of new and alternative data sources include the risks of accessing raw location data, location information about computed using the raw data, the location transmission channel being hijacked, and identification of the person who is generating location information while using a web service or mobile device.

  Thakuriah and Geers (2013) reported that there are three fundamental approaches to addressing locational privacy: legal, consumer awareness and technology-based.

  Legal and consumer awareness strategies and privacy principles include the ability for the users to opt out, having user consent and data protection.

  For technology-based approaches there are numerous methodologies. There is one methodology in particular that has gained considerable support to deal with location privacy threats. This methodology is privacy-by-design where it ensures that the collected data is only being accessed for the purpose for which the user agreed to. To reduce the number of breaches of privacy, the application of simple ideas like the privacy concept being built into physical and software systems would be beneficial. Commonly used approaches include anonymising proxy for all users and applications when collecting location-based information from mobile apps or any probes. Other approaches include degrading the accuracy of the user's location using temporal and geographical masking and encryption. Relevant safeguards relating to data storage methods and duration, and communication encryption could also help in dealing with privacy issues.

  In Australia, a number of legislative instruments throughout the states and territories relate to the safe use of personal information. Further Information can be obtained from the privacy commissioners or related offices.

- Intellectual property rights

  Another challenge in dealing with mass data sources is to encourage an environment of sharing data and protecting the intellectual property (IP) rights of agencies and private sector entities that collect and assemble data.

  Clearly identifying the IP rights of data providers and terms of use for users, preferably in advance, will reduce the likelihood of later disputes and maximise data sharing.

3. Traffic Analysis – Capacity and Level of Service (LOS)

The concepts of capacity analysis and LOS are fundamental to the planning, design and operation of traffic facilities.

HCM 2016 provides detailed information on capacity analysis and is the primary reference document on this topic. Additional material, drawing on substantial Australian work, is well summarised in the Monash University (2003) handbook, from which some of the following material is taken.

3.1 Types of Traffic Facilities

Traffic facilities may be classified into two broad categories:

- **Uninterrupted flow facilities**, on which traffic flow conditions are the result of interactions between vehicles in the traffic stream, and between vehicles and the geometric and environmental characteristics of the road; there are no fixed elements external to the traffic stream, such as traffic control signals, that cause interruptions to traffic flow; examples include two-lane rural roads, multi-lane rural roads, and motorways.
- **Interrupted flow facilities**, on which traffic flow conditions are subject to the influence of fixed elements such as traffic signals, stop signs, give-way signs, roundabouts or other controls which cause traffic to stop periodically, irrespective of the total amount of traffic; examples include urban streets, unsignalised and signalised intersections.

Uninterrupted flow and interrupted flow are terms that describe the type of road facility and not the quality of traffic flow on it.

On uninterrupted flow facilities, the causes of traffic congestion are basically due to the amount of traffic using them, and are thus internal to the traffic stream itself. Capacity analysis for uninterrupted flow facilities is presented in Section 4.

On interrupted flow facilities, control devices external to the traffic stream require traffic to stop periodically for their satisfactory operation. Capacity analysis for interrupted flow facilities is presented in Section 5.

The capacity of an urban street is largely related to the type and density of intersections. At unsignalised intersections, the major road traffic normally has priority over the minor road. The special importance of unsignalised intersections in the context of traffic impact assessment (refer to *Part 12* of the Guide), traffic management for intersections (refer to *Part 6* of the Guide) and the road safety implications should not be overlooked. A roundabout is a particular form of unsignalised intersection and can give rise to quite different capacity considerations. For signalised intersections, the additional element of time allocation between the various conflicting movements must also be considered. Capacity analysis for intersections is presented in Section 6.

3.2 Capacity, Level of Service, and Degree of Saturation

3.2.1 Capacity

Capacity, as defined in HCM 2016, is the maximum sustainable hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under the prevailing roadway, environmental, traffic and control conditions. The concept applies equally to motorised vehicular traffic and to bicycle and pedestrian traffic.

The following points should be noted with respect to the above definition:

- The time period used in capacity analyses should be one hour, but in practice analysis typically focuses on a 15-minute period (specifically, the peak 15 minutes of the peak hour) which is usually accepted as being the shortest interval during which stable flow exists. In Australia and New Zealand this has not been rigorously specified in practice, and the default peak analysis period is often set as 30 minutes.
- The prevailing roadway, traffic and control conditions should be reasonably uniform for the section of facility being analysed.
- Roadway conditions refer to the geometric characteristics of the road, including the type of facility and its development environment, the number of lanes, lane disciplines and lane types in each direction, lane and shoulder widths, lateral clearances, design speed and horizontal and vertical alignments.
- Traffic conditions refer to the characteristics of the traffic stream using the road, including vehicle type and the lane and directional distribution of the traffic.
- Control conditions refer to the types and specific design of the control devices and traffic regulations applicable to the particular section of road.
- Driver characteristics play a central role in determining such key parameters as saturation flow rates at signals, and critical gap and follow-up headways at roundabouts and sign-controlled intersections, used in capacity calculations.

Capacity analysis typically focuses on vehicle capacity, in terms of vehicles (or passenger car equivalents) per hour. In comparisons between different transport modes and systems designed to increase vehicle occupancy (e.g. high-occupancy vehicle lanes) it is relevant to also consider the number of persons per hour passing a point (Monash University 2003).

It is worth noting that the operational capacity as used in Austroads *Guide to Smart Motorways* (Austroads 2016d) is the actual real-time capacity for a road segment, which can vary depending on prevailing roadway, traffic and control conditions. These variable conditions include the percentage of heavy vehicles, driver population (passive or aggressive driving, familiar or unfamiliar with road), road geometry, road surface, time-of-day, weather and light. (Theoretical capacity for a road segment is an average capacity estimate over a period.) Operational capacity, which can be either measured in total vehicles per hour or passenger car equivalents per hour, is particularly relevant to the control of motorways. For example, ramp signals maintain operational capacity while regulating inflow demand to prevent flow breakdown. Austroads (2016d) provides further discussion on motorway operational capacity and the merge capacity for smart motorways with ramp signals.

3.2.2 Level of Service

LOS is a qualitative stratification of the performance measure or measures representing quality of service. A LOS definition is used to translate complex numerical performance results into a simple stratification system representative of road users’ perceptions of the quality of service provided by a facility or service (HCM 2016). These service measures include speed and travel time, delay, density, freedom to manoeuvre, traffic interruptions, comfort and convenience, and safety. In general, there are six levels of service, designated A to F, with LOS A representing the best operating condition and service quality from the users’ perspective (i.e. free-flow) and LOS F the worst (i.e. forced or breakdown flow or having reached a point that most users would consider unsatisfactory, as described by a specific service measure value or a combination of service measure values).

A key issue is determining the LOS that is deemed acceptable, and whether that level should be a projected level for future operation of a facility, or the level existing at the current operation of the facility. The appropriate LOS for a particular jurisdiction will be determined in the context of the policies indicating what are regarded as acceptable levels.

The levels of service for **interrupted flow facilities** are described in Section 5.2.2. The levels of service for **uninterrupted flow facilities** are described in Section 4. The LOS concept may be used as the basis of capacity and operational analysis for all types of road facilities and for multi-modal road users. While there is a range of parameters that could be used to define LOS for each type of road facility, for practical purposes certain quantitative performance measures have been developed for the different types of facility to assist in defining levels of service. These are summarised in Table 3.1 (adapted from HCM 2016).

Table 3.1: Performance measures used for defining LOS

Element	LOS measure[1]
Vehicular	
Interrupted flow	
• Urban street	Speed
• Signalised intersection	Delay
• Two-way-stop intersection	Delay
• Roundabout	Delay
• Interchange ramp terminal	Delay
Uninterrupted flow	
• Two-lane highway	Speed, per cent time spent following
• Multi-lane highway	Density
• Freeway	
- basic segment	Density
- ramp merge or diverge	Density
- weaving	Density
Other road users	
Public transport	[2]
Pedestrians	Speed, delay, space [3]
Cyclists	Speed, event, delay [3]

1 *Service measure for a given facility is the primary performance measure and determines the LOS.*

2 *Several measures capture the multidimensional nature of transit performance when defining LOS (HCM 2016, Chapter 2)*

3 *A LOS score method is used in HCM 2016 and is also discussed in Section 3.4.*

Source: Adapted from Exhibit 2-2 in HCM 2016.

In HCM 2016, LOS is reported separately in each mode for a given system element. Each mode's travellers have different perspectives and could experience different conditions while travelling along a given road. It is worth noting that the Austroads Movement and Place Framework (Austroads 2016a) applies LOS concept to define relative mobility priorities of different modes. In the framework road agencies can understand which user type should have which priority within a certain place. The priority may change by time of day as the role and primary purpose of place changes emphasis throughout the course of the day, for example established urban road networks where key 'movement' roads have major 'place' uses abutting the road (e.g. significant strip shopping centres). These locations create a need for balancing competing demands for movement and place by time of day and day of week. Each jurisdiction would need to develop their own priorities through stakeholder engagement and consultation and assign relative LOS goals for different user types and places based on route type and time of day. Section 3.4 provides further information on the use of multi-modal mobility LOS metrics for the implementation of the Austroads Movement and Place Framework.

3.2.3 Service Flow Rate

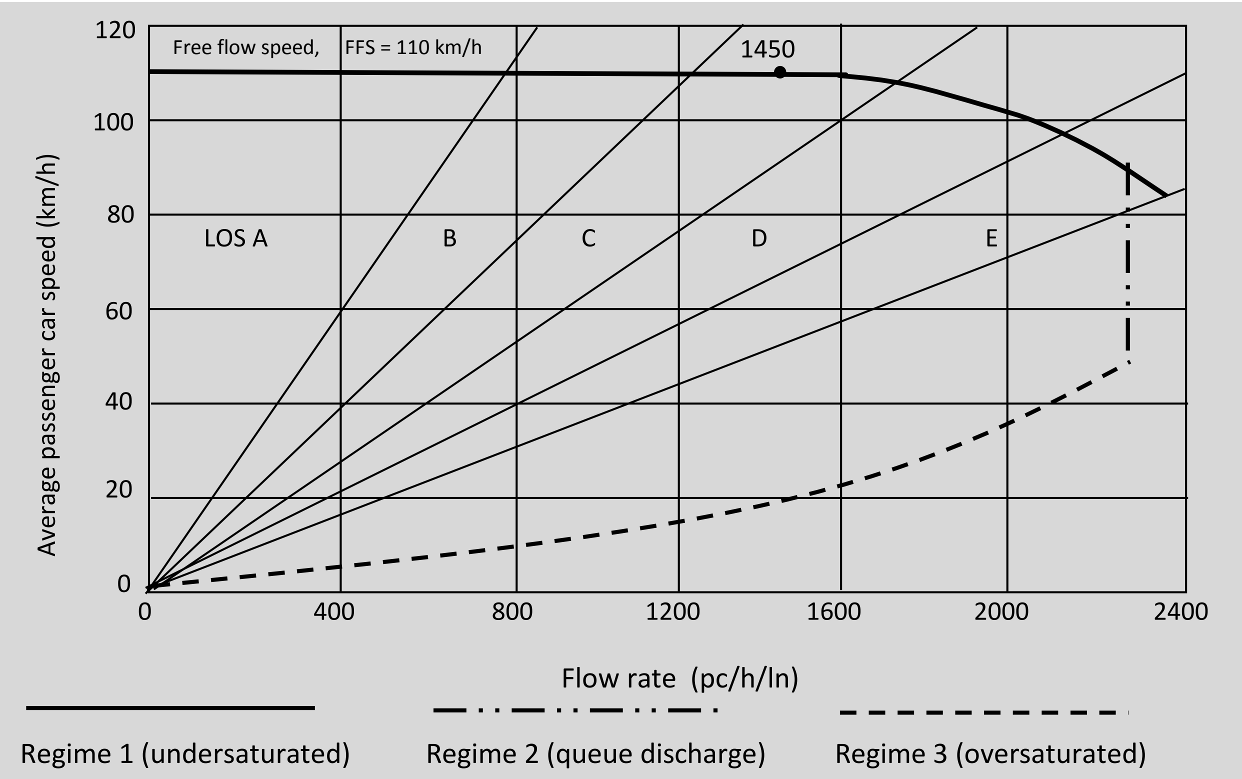
Service flow rates indicate the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point under the prevailing roadway, traffic and control conditions while maintaining a designated LOS. They indicate the vehicle or person capacity for each LOS and are used to determine the LOS corresponding to actual traffic volumes. As with capacity, the service flow rate is generally taken over a 15-minute time period.

Each type of facility has five service flow rates, one for each of the levels of service A to E, as illustrated in Figure 3.1. At LOS F flow breakdown occurs, and a meaningful service flow rate cannot be specified.

Figure 3.1 depicts an example of a freeway operation curve for a free speed of 110 km/h. Other facilities and freeways with different operating speeds will have different curves. The different levels of service are described by the density (pc/km/ln).

At each LOS, the service flow rate is defined as the maximum for that level. Service flow rates are discrete values, whereas the LOS represents a range of conditions. Service flow rates therefore effectively define the flow boundaries between the levels of service.

Figure 3.1: LOS and service flow rates for freeways



Note: The LOS is defined by density as the slopes shown in the diagram. Highway Capacity Manual, TRB (2016)

Source: Adapted from Exhibits 12-3, 12-7 and 12-16 in HCM 2016 (TRB 2016).

3.2.4 Degree of Saturation

The concept of degree of saturation, or VCR, is used in the capacity and operational analysis of intersections.

The degree of saturation of a signalised intersection approach may be defined as the ratio of the arrival flow (demand) to the capacity of the approach during the same period.

The degree of saturation of an intersection approach ranges from close to zero for very low traffic flows up to 1.0 for saturated flow or capacity. A degree of saturation greater than 1.0 indicates oversaturated conditions in which long queues of vehicles build up on the critical approaches. In general, the lower the degree of saturation the better the quality of traffic service. However, the degree of saturation, delay and queue length parameters should always be used together to assess intersection performance.

In practice the target degrees of saturation of 0.90 for signals, 0.85 for roundabouts and 0.80 for unsignalised intersections are generally agreed to. These are usually called ‘practical degrees of saturation’.

3.3 Factors Affecting Capacity, Level of Service, and Degree of Saturation

3.3.1 Ideal Conditions

In the analysis of capacity, LOS or degree of saturation, the starting point is often to select values that are applicable to ideal conditions, and then to apply correction or adjustment factors that reflect the actual roadway, traffic and control conditions.

In general, an ideal condition is one for which further improvements will not result in any increase in capacity or LOS or decrease in the degree of saturation. More specific details of ideal conditions are given in later sections.

3.3.2 Roadway Conditions

Roadway conditions that affect capacity, LOS and degree of saturation include:

- type of facility and its development environment
- traffic lane widths
- shoulder widths and/or lateral clearances
- design speed
- horizontal and vertical alignment.

Adjustment factors for these conditions are referred to in subsequent sections of this Guide, and details are given in the relevant sections of HCM 2016.

Note that these are based on US data but generally apply to Australian and New Zealand conditions. Judgement may need to be exercised in their use for local conditions.

3.3.3 Terrain Conditions

For capacity and related analyses, the general terrain of a roadway is classified into three categories, namely:

Level terrain – any combination of grades and horizontal and vertical alignment permitting heavy vehicles to maintain about the same speed as passenger cars.

Rolling terrain – any combination of grades and horizontal and vertical alignment causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but not causing them to operate at crawl speeds for any significant length of time.

Mountainous terrain – any combination of grades and horizontal and vertical alignment causing heavy vehicles to operate at crawl speeds for significant distances and/or at frequent intervals.

It is also necessary to consider pedestrian and cyclist needs as their presence in the traffic mix for the given terrain is likely to affect the capacity.

3.3.4 Traffic Composition

For capacity and related analyses, road traffic is typically classified into three categories, namely:

Passenger cars – vehicles registered as passenger cars, plus other light vehicles and light delivery vans having no more than four single tyres.

Trucks – vehicles having more than four single tyres, and involved primarily in the transport of goods or services.

Buses – vehicles having more than four single tyres, and involved primarily in the transport of people. For capacity analysis, passenger cars towing caravans, boats and other similar recreation equipment should be included in this category.

This classification of vehicles into three categories is generally consistent with current classification counting, although it is noted that modern loop detectors classify vehicles into four categories. The Austroads vehicle classification categories are described in Appendix A.

3.3.5 Pedestrians and Cyclists

Pedestrians form the largest single road user group. Most individual trips, whatever the primary mode used, begin and/or finish with a walk section. Walking is a fundamental component of all travel. Pedestrian safety and mobility must be explicit factors in the planning, design, implementation and maintenance of road facilities.

Initiatives are being implemented at the planning and policy levels, which give explicit consideration to the walking mode and to pedestrian safety and amenity.

The framework for pedestrian planning and design lies within the practice of traffic engineering. Pedestrians may be considered as being similar to vehicles, operating on a transport network consisting of footways, stairs, travelators etc. Network capacity can be defined, demand measured or predicted, operational levels calculated, and areas of congestion and hazard identified. To achieve maximum safety, the pedestrian network should be separate from, but integrated with, the main road and public transport system. This will necessitate regular crossings in order to sustain the coverage and continuity of the network for walking.

Cyclists also form a significant road-user group with specific needs. Solutions addressing their capacity needs can significantly affect road capacity, LOS and degree of saturation.

Because of increasing environmental, physical and financial constraints, attention must be given to issues of traffic calming and demand management in order to encourage pedestrian, cycling and higher vehicle occupancy modes of transport. Planning, design and management of traffic facilities must cater for pedestrian or cyclist traffic along or across roads as this can affect road capacity.

3.3.6 Driver Population

The traffic stream characteristics associated with the information provided in sections of this Part are representative of regular weekday commuter drivers and other regular users of a facility. In situations where weekend or recreation, or even mid-day (off-peak) drivers are a significant proportion of the traffic stream, the capacity and/or LOS may be reduced.

An adjustment factor is provided where relevant, but the range of values given is relatively wide and engineering judgement and/or local data should be used in selecting the appropriate value.

3.3.7 Control Conditions

The control of the time available for individual traffic movements is a critical factor affecting the capacity, LOS and/or the degree of saturation of interrupted flow facilities. Typical forms of control include traffic signals, stop and give-way signs, turn restrictions, lane-use controls and parking restrictions. The effects of various forms of control are described in later sections.

3.4 Multi-modal Level of Service

HCM 2016 considers assessment of the LOS of road or highway facilities from a multi-modal LOS approach, derived from a major research project NCHRP 3-70 (Dowling et al. 2008). The multi-modal approach recognises that different mode users could perceive the quality of service differently due to their different perspectives and experiences. This approach considers the different road-user perspectives for vehicles (cars), public transport, pedestrians and cyclists.

A facility can be assessed by determining the LOS score for each of the modes, and comparing the numerical scores against the LOS criteria. An overall LOS is not calculated. Judgments on the selection or development of the facility must be made based on the different modal scores, and additional relevant information (e.g. safety performance), depending on the function intended for the roadway concerned. This necessarily involves consideration of how one mode affects the service quality of other modes, and trade-offs between modes.

The method includes a complete street analysis approach for interrupted flow facilities (segments and crossings, including at roundabouts). Emphasis is placed on ‘quality of service’ to consider how well a facility or service operates from a user’s perspective. Details of the computational methods are given in Dowling et al. (2008) and in the electronic Volume 4 *Applications Guide* of HCM 2016.

There are merits in using a quantitative framework such as the HCM 2016 framework, but practitioners tend to favour either a qualitative or semi-qualitative approach such as the SmartRoads LOS framework (VicRoads 2015). Austroads (2015d) has adopted the qualitative/semi-qualitative approach and developed a framework to establish a LOS rating for network operation outcomes relevant for different road users. The Austroads LOS framework enables integrated planning and decision making within network operations that recognises the various transport needs (including mobility, safety, access, information and amenity) and various transport user groups (private motorists, transit users, pedestrians, cyclists and freight operators).

As mentioned, Austroads (2016a) applies the Austroads multi-modal LOS metrics for the implementation of the Austroads Movement and Place Framework. This framework addresses network needs of the various categories of users, the characteristics of different networks and, importantly, describes a planning process for balancing or prioritising the competing needs of different users.

3.5 Austroads Pedestrian Facility Selection Tool

Austroads reviewed jurisdictional practices (e.g. NZ Transport Agency (NZTA) 2009 and Queensland Department of Transport and Main Roads (TMR) 2017) and developed a tool to help in selecting the most appropriate pedestrian crossing facility by applying the pedestrian LOS concept (Austroads 2015c). This tool is available to all practitioners in Australia and New Zealand through the Austroads website (https://austroads.com.au/network-operations/network-management/pedestrian-facility-selection-tool). The tool is designed to help in the selection of the type and characteristics of the facility, such as uncontrolled, zebra and signalised with or without raised platforms, kerb extensions and median refuges based on the walkability, pedestrian safety and economic viability. It has been linked to the jurisdictional economic evaluation procedures (e.g. NZTA 2016) to support economic cases for funding applications. It also takes into consideration the recent research on pedestrian perceptions of walkability in different facilities, as well as considering feasibility criteria based on inputs from different jurisdictions in both Australia and New Zealand.

4. Uninterrupted Flow Facilities

HCM 2016 provides detailed information on capacity analysis and is the primary reference document for this section. However, it is acknowledged that Austroads jurisdictions have adopted HCM methodologies and other relevant guidelines (e.g. Highways Agency 2006 and Research Society for Roads and Transportation (ed) 2015). Research into and enhancement of the freeway capacity analysis methodology has also been very active in Australia and New Zealand. A thorough review of highway capacity analysis is currently being undertaken by Austroads.

4.1 Single-lane Flow

In certain situations, traffic flow may be constrained to a single traffic lane without overtaking, typical examples being a single lane provided in one direction on an undivided urban road with reversible lane flow, in a tunnel or at a construction or maintenance site. In these types of situations, depending on the length of the single lane restriction and on the volume of traffic, the speed of all vehicles will tend to the speed of the slowest vehicle, and traffic will operate in accordance with car-following mechanisms.

Several of the earliest studies of traffic capacity examined single-lane traffic flow. A linear relationship between the average speed and the density of vehicles was assumed, and the capacity was related to the free speed (space mean speed at low flows, space mean speed is the arithmetic mean of the measured speeds of all vehicles within a given length of lane or carriageway, at a given instant of time) and the jam density (the maximum density for stopped traffic) as in Equation 4:

$$C = k_j \, V_f / 4 \qquad 4$$

where

C = capacity (passenger cars per hour - pc/h)

V_f = free speed (km per hour)

k_j = jam density (pc per km)

In some cases, the speed-spacing relationship was based on limited field observations, but in most cases it was determined by using factors such as driver reaction time, coefficients of friction and braking distances. The capacity of a single lane so determined varied from 1000 to 4800 pc/h, but with most results in the range 1500 to 2400 pc/h.

4.1.1 Capacity

If there is a small bunch of vehicles in a single traffic lane all moving at relatively high speed, the average headway between them may be small. However, on a long length of road without overtaking, these small headways usually cannot be sustained over a long period of time.

If single-lane conditions without overtaking are retained over a significant length of the road, then as traffic volumes increase, a long unbroken line of vehicles (or a long bunch) develops and the speeds of all vehicles tend to that of the slowest vehicle, and stop-start conditions may develop. Once this occurs, the maximum flow rate of a single lane is reduced to that equivalent to a headway of about 2 seconds, i.e. to an ‘operational capacity’ of about 1800 pc/h. In general, this figure can be regarded as the capacity of a single lane without overtaking.

Factors affecting capacity

The capacity of a single traffic lane will be affected by factors such as the pavement width and restricted lateral clearances, the presence of heavy vehicles and grades.

The capacity of a significant length of a single traffic lane for the prevailing roadway and traffic conditions can be calculated by using Equation 5:

$$C = 1800\, f_W\, f_{HV} \tag{5}$$

where

C	=	capacity in veh/h under prevailing roadway and traffic conditions
f_W	=	adjustment factor for narrow lanes and lateral clearances, obtained from Table 4.1
f_{HV}	=	adjustment factor for heavy vehicles
	=	$1/[1+ P_{HV}\,(E_{HV} - 1)]$
P_{HV}	=	the proportion of heavy vehicles in the traffic stream, expressed as a decimal
E_{HV}	=	the average passenger car equivalents for heavy vehicles obtained from Table 4.2.

Factual data on adjustment factors that should be applied to the capacity of 1800 pc/h for a single lane is very limited. In the absence of specific measurements at comparable sites, the adjustment factors given in Table 4.1 and Table 4.2 may be used as a reasonable approximation, although care needs to be exercised in their use.

A worked example is given in Commentary C1.1.1.

[see Commentary 1]

Where a roadway with two or more lanes in one direction is restricted over a short length (say less than about 100 m) to a single- lane flow and provided that the upstream merge is adequately designed, the capacity may be calculated using a base flow rate of 2400 pc/h rather than 1800 pc/h as in the above equation.

Table 4.1: Adjustment factors for lane width and lateral clearance

Lateral clearances on each side (m)	Lane width		
	3.7 m	3.2 m	2.7 m
2	1.00	0.90	0.70
1	0.90	0.80	0.63
0	0.65	0.60	0.50

Table 4.2: Average passenger car equivalents for heavy vehicles on grades with single-lane flow

Grade	Passenger car equivalents
Level	2.00
Moderate	4.0
Long sustained	8.0

4.2 Two-lane Two-way Roads

Two-lane rural roads have one lane for use by traffic travelling in each direction. Overtaking of slower vehicles requires the use of the opposing traffic lane when sight distance and gaps in the opposing traffic stream permit. At low traffic volumes and under ideal conditions, drivers are able to travel at their desired speed without interference. As traffic volumes increase, and as roadway, terrain and traffic conditions become less than ideal, drivers are increasingly affected by the presence of other vehicles on the road, and bunches form in the traffic stream. Vehicles in these bunches are subjected to delay because of the inability to overtake slower-moving vehicles.

The analysis process outlined in *Part 3* is based on HCM 2016. The manual should be consulted for further explanation of the concepts and a detailed description of the process. Software that provides solutions in agreement with the HCM processes is available, from McTrans (2017) for example, for all HCM procedures.

HCM 2016 distinguishes between three categories of two-lane highways, as follows:

- Class I – These are two-lane highways on which motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or primary links in state or national highway networks generally are assigned to Class I. Class I facilities most often serve long-distance trips or provide connecting links between facilities that serve long-distance trips.
- Class II – These are two-lane highways on which motorists do not necessarily expect to travel at high speeds. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary arterials, or pass through rugged terrain generally are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning and ending portions of longer trips, or trips for which sightseeing plays a significant role.
- Class III – These are two-lane highways serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns or developed recreational areas. On such segments, local traffic often mixes with through traffic, and the density of unsignalised roadside access points is noticeably higher than in a purely rural area. Class III highways may also be longer segments passing through more spread-out recreational areas, also with increased roadside densities. Such segments are often accompanied by reduced speed limits that reflect the higher activity level.

The three classes of road perform markedly different functions and are not totally dependent on the highway's role in the hierarchy. A highway between major urban centres through mountainous terrain might be classified as Class II instead of Class I if it is established that drivers appreciate that higher speeds are inappropriate.

HCM 2016 provides a method to analyse the performance of a two-way segment: as directional segments, with each direction of travel considered separately; for sections with steep grades and for segments with passing lanes. The two-way analysis assumes roughly similar flows in each direction.

The performance of a particular road section is calculated from the predicted performance of base or ideal conditions which include:

- lane widths greater than or equal to 3.6 m
- clear shoulders wider than or equal to 1.8 m
- no no-overtaking zones
- all passenger cars
- no impediments to through traffic, such as traffic control or turning vehicles
- level terrain.

The process estimates the free-flow speed (FFS) based on the base free-flow speed (from the base conditions) and termed the BFFS. The FFS is based on the lane widths, shoulder widths and the average number of access points per kilometre. Reduce the shoulder width or the lane width and free speed is reduced. Increase the frequency of access points and the free speed is reduced. The actual travel speed is a function of the FFS and the passenger car equivalent flow rate which is based on the proportion of heavy vehicles, the grade (or terrain type) and broad flow rates. The effect of an additional vehicle at low flows differs from the effect of an additional vehicle at higher flows.

The per cent time-spent-following is a measure of the level of opportunities to overtake. The per cent time-spent-following approaches 100 asymptotically:

- as the traffic demand increases
- as the percentage of no-overtaking zones increases
- as the directional split moves closer to 90/10.

HCM 2016 considers a no-overtaking zone to exist when the sight distance is below 300 m. For the analysis of an existing roadway, a no-overtaking zone could be defined by a barrier line. This would make it easier for practitioners to measure the zone in the field, or from plans, than to determine where the sight distance is below 300 m.

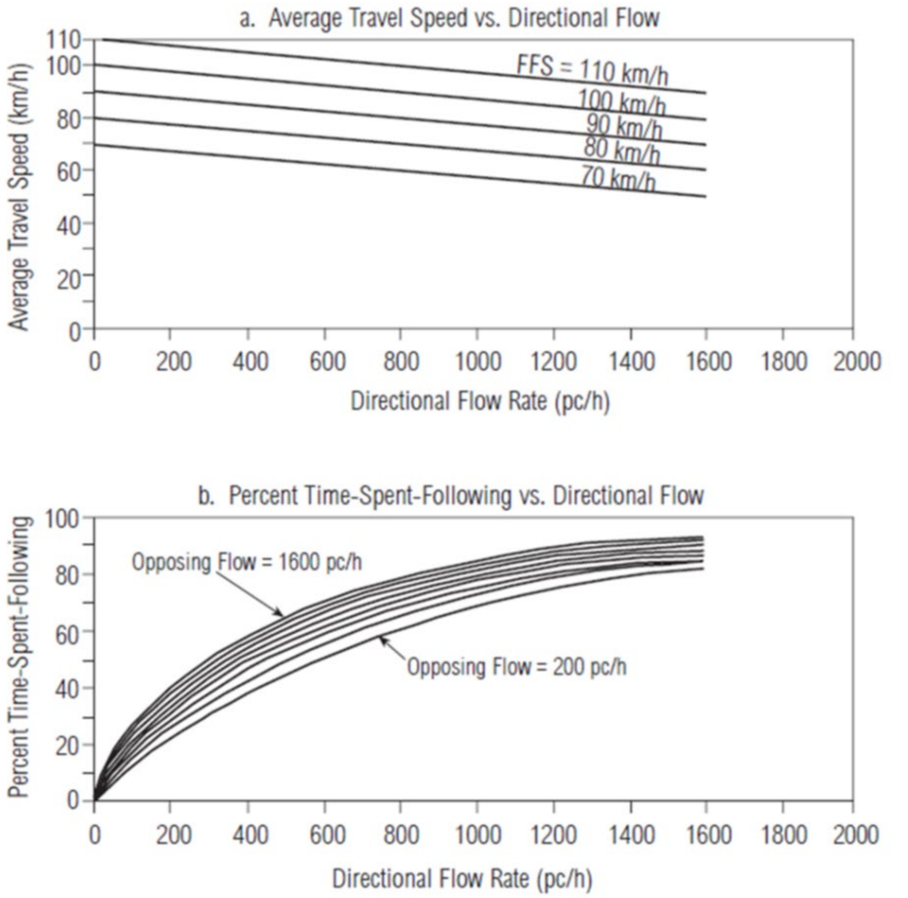
Figure 4.1 shows the expected average travel speed and the per cent time-spent-following for ideal or base conditions.

The LOS criteria used for the different highway classes differ. HCM 2016 notes that:

- The LOS for Class I highways on which efficient mobility is paramount is defined in terms of both per cent time-spent-following and average travel speed.
- On Class II highways, mobility is less critical, and LOS is defined only in terms of per cent time-spent-following.

On Class III highways, high speeds are not expected. Because the length of Class III segments is generally limited, passing restrictions are also not a major concern. In these cases, drivers would like to make steady progress at or near the speed limit. Therefore, on these highways, per cent free-flow speed (PFFS) is used to define LOS.

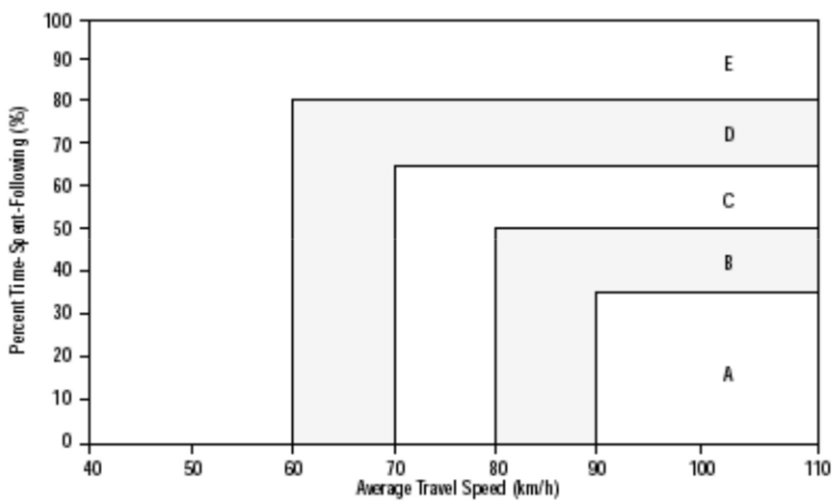
Figure 4.1: Speed-flow and per cent time-spent-following relationships for directional segments with base conditions



Source: Adapted from Exhibit 15-2 in HCM 2016 (TRB 2016).

Figure 4.2 shows the LOS criteria for Class I highways. The criteria for automobile two-lane highways are shown in Table 4.3. The LOS can be evaluated from field measurements on an existing road. The per cent of vehicles within 3 seconds, collected at fixed regular intervals and averaged for the section in question, is a good substitute for the per cent time-spent-following.

Figure 4.2: LOS criteria for two-lane highways in Class I



Source: Adapted from Exhibit 15-3 in HCM 2016 (TRB 2016).

Table 4.3: Automobile LOS for two-lane highways

LOS	Class I highway		Class II highways PTSF (%)	Class III highways PFFS (%)
	Average travel speed ATS (km/h)	Per cent time-spent-following (PTSF) (%)		
A	> 90	≤ 35	< 40	> 91.7
B	> 80 – 90	> 35 – 50	> 40 – 55	> 83.3 – 91.7
C	> 70 – 80	> 50 – 65	> 55 – 70	> 75.0 – 83.3
D	> 60 – 70	> 65 – 80	> 70 – 85	> 66.7 – 75.0
E	≤ 60	> 80	> 85	≤ 66.7

Note: LOS F applies whenever the arrival flow exceeds the segment capacity.

Source: Adapted from Exhibit 15-3 in HCM 2016 (TRB 2016).

HCM 2016 indicates that the capacity of a two-lane highway is 1700 pc/h for each direction of travel and is nearly independent of the directional distribution of traffic. For extended lengths of two-lane highway, the capacity will not exceed 3200 pc/h for both directions of travel combined.

The conditions for the different levels of performance of two-lane highways are describes in the following terms:

- At LOS A, motorists experience high operating speeds on Class I highways and little difficulty in passing. Platoons of three or more vehicles are rare. On Class II highways, speed would be controlled primarily by roadway conditions. A small amount of platooning would be expected. On Class III highways, drivers should be able to maintain operating speeds close or equal to the FFS of the facility.
- At LOS B, passing demand and passing capacity are balanced. On both Class I and Class II highways, the degree of platooning becomes noticeable. Some speed reductions are present on Class I highways. On Class III highways, it becomes difficult to maintain FFS operation, but the speed reduction is still relatively small.
- At LOS C, most vehicles are travelling in platoons. Speeds are noticeably curtailed on all three classes of highway.
- At LOS D, platooning increases significantly. Passing demand is high on both Class I and II facilities, but passing capacity approaches zero. A high percentage of vehicles are now travelling in platoons, and PTSF is quite noticeable. On Class III highways, the fall-off from FFS is now significant.
- At LOS E, demand is approaching capacity. Passing on Class I and II highways is virtually impossible, and PTSF is more than 80%. Speeds are seriously curtailed. On Class III highways, speed is less than two-thirds the FFS. The lower limit of this LOS represents capacity.
- LOS F exists whenever arrival flow in one or both directions exceeds the capacity of the segment. Operating conditions are unstable, and heavy congestion exists on all classes of two-lane highway.

4.3 Multi-lane Roads

Multi-lane roads have two or more lanes for use by traffic in each direction. They may be classified as:

- divided – when opposing directions of traffic are physically separated by a median
- undivided – when opposing directions of traffic are not physically separated.

Multi-lane roads have at-grade intersections including signalised intersections; this attribute distinguishes them from freeways. Multi-lane roads and urban roads have different traffic signal densities. HCM 2016 provides procedures for both classes that should be reviewed when analysing urban roads in suburban areas.

The recommended analysis procedure is based on HCM 2016, Chapter 12. The analysis is extended for road sections with higher grades over longer distances. The analysis involves a comparison with base conditions. The base represents the highest operating level of a multi-lane highway with the following characteristics. Software, from McTrans (2017) used earlier for example, provides solutions in agreement with these HCM 2016 procedures. The characteristics are:

- 3.6 m minimum lane widths
- 3.6 m minimum total lateral clearance in the direction of travel – this represents the total lateral clearances from the edge of the travel lanes to obstructions along the edge of the road and in the median (in computations, lateral clearances greater than 1.8 m are considered in computations to be equal to 1.8 m)
- only passenger cars in the traffic stream
- no direct access points along the roadway
- a divided highway
- FFS equal to 100 km/h.

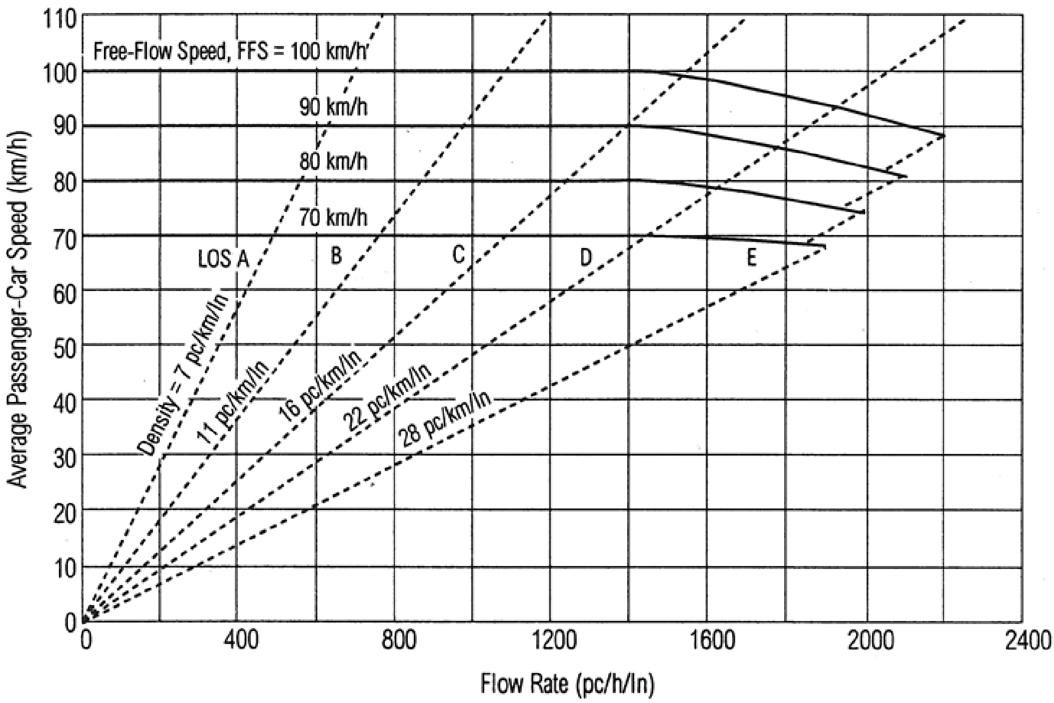
The FFS is estimated from the FFS for the base conditions. A value between 8 and 10 km/h above the speed limit could be used if there is no other evidence from similar roads. The FFS is reduced if lane width is reduced, if the total lateral clearance is reduced, and as the frequency of access points is increased. The free speed on an undivided road is 2.6 km/h lower than on a divided road with the same characteristics.

The design flow rate is calculated for a peak 15 minutes and accounts for the percentage and type of heavy vehicles, whether drivers are commuters or not, and on the variation of the traffic within a peak hour. HCM 2016 uses a peak hour factor (PHF) term in almost all procedures. The PHF is the ratio between the hourly flow and the peak 15-minute flow rate.

When using the HCM 2016 procedure, there may be a need to consider the vehicle equivalency of Australian and New Zealand trucks. There is little evidence to indicate appropriate values to be used, but a sensitivity analysis could be used to establish if higher vehicle equivalency values are appropriate.

The average trip speed is then estimated using the FFS and the design flow rate. Figure 4.3 shows the relationship between these parameters although HCM 2016 provides equations for a computerised solution. A vehicle density, given by the design flow rate divided by the average trip speed, is the prime measure used in the evaluation of the LOS. Other characteristics for different LOS levels are shown in Table 4.4.

Figure 4.3: Speed-flow curves with LOS criteria for multi-lane roads



Source: Adapted from Exhibit 12-17 in HCM 2016 (TRB 2016); note that parts of the curves are obscured by grid lines.

Table 4.4: LOS criteria for multi-lane highways

Free-flow speed	Criteria	A	B	C	D	E
100 km/h	Maximum density (pc/km/ln)	7	11	16	22	25
	Average speed (km/h)	100.0	100.0	98.4	91.5	88.0
	Maximum volume to capacity ratio (v/c)	0.32	0.50	0.72	0.92	1.00
	Maximum service flow rate (pc/h/ln)	660	1080	1550	1980	2200
90 km/h	Maximum density (pc/km/ln)	7	11	16	22	26
	Average speed (km/h)	90.0	90.0	89.8	84.7	80.8
	Maximum volume to capacity ratio (v/c)	0.30	0.47	0.68	0.89	1.00
	Maximum service flow rate (pc/h/ln)	600	990	1430	1850	2100
80 km/h	Maximum density (pc/km/ln)	7	11	16	22	27
	Average speed (km/h)	80.0	80.0	80.0	77.6	74.1
	Maximum volume to capacity ratio (v/c)	0.28	0.44	0.64	0.85	1.00
	Maximum service flow rate (pc/h/ln)	550	900	1300	1710	2000
70 km/h	Maximum density (pc/km/ln)	7	11	16	22	28
	Average speed (km/h)	70.0	70.0	70.0	69.6	67.9
	Maximum volume to capacity ratio (v/c)	0.26	0.41	0.59	0.81	1.00
	Maximum service flow rate (pc/h/ln)	290	810	1170	1550	1900

Source: Exhibit 21-2 in HCM 2000; the content here is largely consistent with the LOS criteria in HCM 2016 (TRB 2016) (Exhibit 12-15 and Exhibit 12-38).

4.4 Freeways

A freeway is a divided road with two or more lanes for traffic travelling in each direction, with no at-grade intersections and with full control of access from abutting property.

Freeways have three types of elements, namely:

- basic freeway segments – which are sections of freeway that are outside the influence of any ramp or weaving area
- ramps and ramp terminals – which provide access to and from the freeway
- weaving areas – which are sections of freeway on which two or more vehicle flows must cross, such as when a merge area is closely followed by a diverge area.

These elements can be combined to evaluate a freeway facility. HCM 2016 provides a method to evaluate the performance of a freeway that has congested elements for more than one 15-minute time period.

The recommended analysis procedure is the current procedure in HCM 2016, and the default values documented in it should be used. Research has shown that the speed-flow relationship for freeways in the USA, Canada, Germany, the UK and Australia have similar values for comparable freeway geometries and free speeds.

Guidance on road space allocation and lane management for freeways is provided in Austroads (2017a). Guidance on smart motorways is provided in Austroads (2016d).

4.4.1 Basic Freeway Segments

As for other uninterrupted facilities, the evaluation of a basic freeway segment relies on an evaluation of the average travel speed. The first consideration is the FFS, which could be measured in the field, or estimated using relationships in HCM 2016 (Chapter 12).

Basic freeway segments can be analysed for sections with a specific grade. This process is typically applied to sections with steeper grades over extended lengths. The process is not discussed here and HCM 2016 should be consulted for further information.

The FFS is a function of the base FFS and a number of adjustment parameters. The base conditions are:

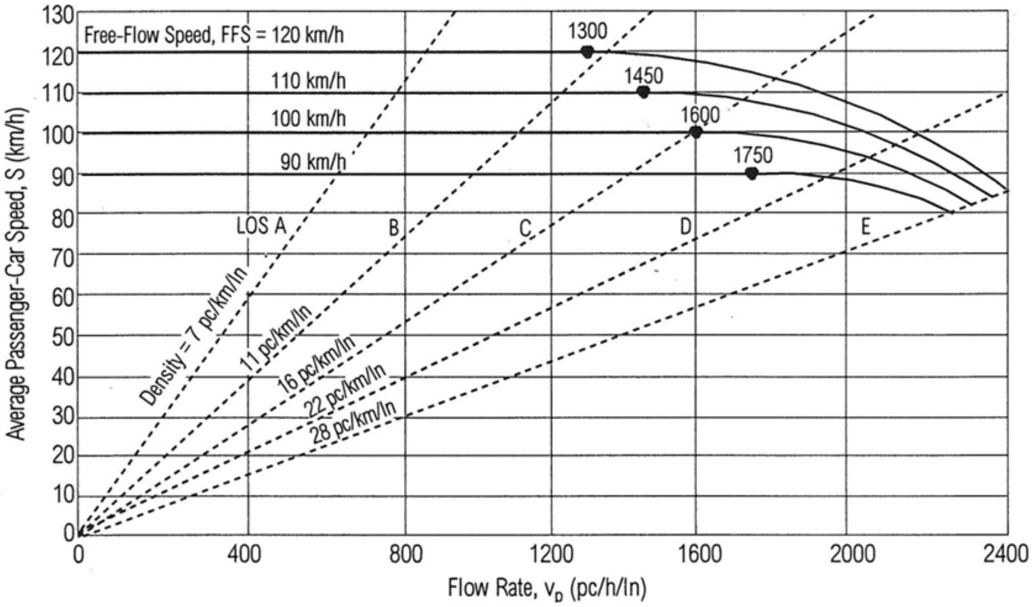
- minimum lane widths of 3.6 m
- minimum left-shoulder lateral clearance between the edge of the travel lane and the nearest obstacle or object that influences traffic behaviour of 1.8 m
- minimum median lateral clearance of 0.6 m
- traffic stream composed entirely of passenger cars
- five or more lanes for one direction (in urban areas only)
- interchange spacing at 3 km or greater
- level terrain, with grades no greater than 2%
- a driver population composed principally of regular users of the facility
- the free speed is 120 km/h in rural areas and 110 km/h in urban areas.

The FFS is reduced when the lane width is reduced, when the left shoulder width is reduced, when the number of lanes is reduced and when the interchange density is increased.

The design flow rate is a function of the type and proportion of heavy vehicles and whether or not drivers are commuters. The effect of long combination vehicles such as road trains may need to be considered in this approach, but appropriate values for the vehicle equivalencies are not readily available.

The LOS is calculated from the vehicle density, being the design flow rate divided by the average passenger-car speed. Figure 4.4 shows the speed-flow curve for basic freeway segments. Results can be measured or estimated and plotted directly onto this curve to predict the LOS. Table 4.5 is a list of the LOS criteria with density being the prime term.

Figure 4.4: Speed-flow relationship for basic freeway segments



Source: Adapted from Exhibit 12-16 in HCM 2016 (TRB 2016).

Table 4.5: LOS criteria for basic freeway segments

Criteria	LOS				
	A	B	C	D	E
FFS = 120 km/h					
Maximum density (pc/km/ln)	7	11	16	22	28
Minimum speed (km/h)	120.0	120.0	114.6	99.6	85.7
Maximum (v/c)	0.35	0.55	0.77	0.92	1.00
Maximum service flow rate (pc/h/ln)	840	1320	1840	2200	2400
FFS = 110 km/h					
Maximum density (pc/km/ln)	7	11	16	22	28
Minimum speed (km/h)	110.0	110.0	108.5	97.2	83.9
Maximum (v/c)	0.33	0.51	0.74	0.91	1.00
Maximum service flow rate (pc/h/ln)	770	1210	1740	2135	2350
FFS = 100 km/h					
Maximum density (pc/km/ln)	7	11	16	22	28
Minimum speed (km/h)	100.0	100.0	100.0	93.8	82.1
Maximum (v/c)	0.30	0.48	0.70	0.90	1.00
Maximum service flow rate (pc/h/ln)	700	1100	1600	2065	2300
FFS = 90 km/h					
Maximum density (pc/km/ln)	7	11	16	22	28
Minimum speed (km/h)	90.0	90.0	90.0	89.1	80.4
Maximum (v/c)	0.28	0.44	0.64	0.87	1.00
Maximum service flow rate (pc/h/ln)	630	990	1440	1955	2250

Source: Adapted from Exhibit 23-2 in HCM 2000 and updated from Exhibit 12-15 in HCM 2016 (TRB 2016).

Engineering judgement is necessary in applying the above table for local practices. The maximum (service) flow rate at a LOS E is a geometric or physical capacity suitable for strategic planning purposes when the future demand would operate at a design LOS. If the design LOS for the future year is C, then the maximum VCR is 0.70 at an FFS of 100 km/h and this VCR determines the number of lanes required for the future based on a capacity of 2300 pc/h/lane.

As mentioned in Section 4.1.1, the maximum flow rates on a freeway could fall below the geometric capacity due to flow breakdowns during peak-flow periods. VicRoads (2013) reported that the maximum flow per lane at an FFS of 100 km/h could fall from 2000 veh/h to 1700 veh/h (or from 2200 pc/h to 1870 pc/h at 10% HV with 1 HV = 2 pc), and that a maximum flow of 2100 pc/h could be maintained with ramp metering.

On basic freeway segments, the LOS definitions from HCM 2016 are also applicable to multi-lane highways as follows:

- LOS A describes free-flow operations. FFS prevail on the freeway or multi-lane highway, and vehicles are almost completely unimpeded in their ability to manoeuvre within the traffic stream. The effects of incidents or point breakdowns are easily absorbed.
- LOS B represents reasonably free-flow operations, and FFS on the freeway or multi-lane highway is maintained. The ability to manoeuvre within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents are still easily absorbed.
- LOS C provides the flow conditions with speeds near the FFS of the freeway or multi-lane highway. Freedom to manoeuvre within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service will be substantial. Queues may be expected to form behind any significant blockage.
- LOS D is the level at which speeds begin to decline slightly with increasing flows, with density increasing more quickly. Freedom to manoeuvre within the traffic stream is seriously limited, and the drivers experience reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.
- LOS E describes operation at or near capacity. Operations on the freeway or multi-lane highway at this level are highly volatile because there are virtually no usable gaps within the traffic stream, leaving little room to manoeuvre within the traffic stream. Any disruption to the traffic stream, such as vehicles entering from a ramp or an access point or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic stream. Towards the upper boundary of LOS E, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown and substantial queuing. The physical and psychological comfort afforded drivers is poor.
- LOS F describes unstable flow. Such conditions exist within queues forming behind bottlenecks. Breakdowns occur for a number of reasons as follows:
  - traffic incidents can temporarily reduce the capacity of a short segment, so that the number of vehicles arriving at a point is greater than the number of vehicles that can move through it
  - points of recurring congestion, such as merge or weaving segments and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles that can be discharged
  - in analyses using forecast volumes, the projected flow rate can exceed the estimated capacity of a given location.

Note that in all cases, breakdown occurs when the ratio of existing demand to actual capacity, or of forecast demand to estimated capacity, exceeds 1.0. LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point.

Operations immediately downstream of such a point, however, are generally at or near capacity, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharging vehicles move away from the bottleneck.

Guidance on the operational capacity analysis for smart motorways is provided in Austroads (2016d).

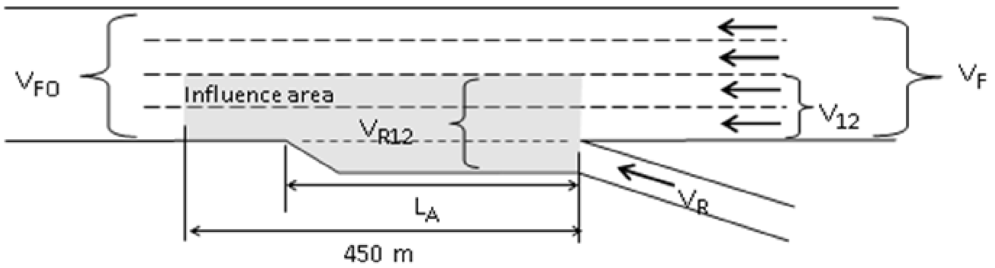
4.4.2 Ramps and Ramp Junctions

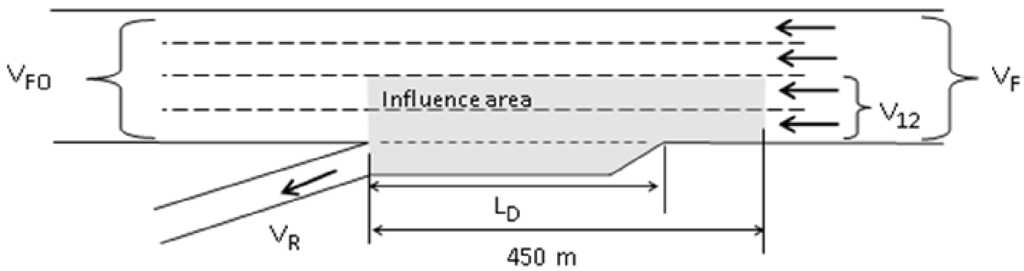
The recommended procedure for the evaluation of ramps is given in HCM 2016 (Chapter 14). The evaluation of ramps considers vehicle interactions that occur within 450 m of the ramp, shown as the influence area in Figure 4.5. This figure also shows important traffic-flow variables used in the analysis. The traffic flows are recorded in passenger car units per hour and are a function of the type and proportion of heavy vehicles and the attributes of the driving population.

The terminology for the peak 15-minute traffic flows is as follows:

- V_F is the total traffic across all lanes of the carriageway entering the ramp area
- V_R is the ramp traffic
- V_{12} is the traffic in the two kerb-side lanes of the carriageway entering the ramp area
- V_{R12} is the traffic in the two kerb lanes and on the ramp
- V_{FO} is the total traffic across all lanes of the carriageway exiting the ramp area.

Figure 4.5: Influence area at ramps





Source: Adapted from Exhibit 14-7 in HCM 2016 (TRB 2016).

The process to evaluate ramps is to predict the traffic in the two kerb-side lanes (lane 1 and lane 2). (If there are only two lanes on the carriageway, then this flow is equal to the total flow.) The proportion of traffic in the two kerb-side lanes depends on the proximity and type of the previous upstream ramp and the proximity and type of the next downstream ramp.

The density of the merge area is then calculated using a linear relationship with the peak 15-minute ramp flow, V_R, the flow in the two kerb-side lanes, V_{12}, and the acceleration lane length L_A. HCM 2016 defines the acceleration-lane length measured from ‘the point at which the right edge of the ramp lane or lanes and the left edge of the freeway lanes converge to the end of the taper segment connecting the ramp to the freeway. The point of convergence can be defined by painted markings or physical barriers or by some combination of the two. Note that both taper area and parallel ramps are measured in the same way’.

Equation 6 is used to estimate the density in the merge influence area, replicated from HCM 2000 Equation 25-5. Note that the equation for density applies only to undersaturated flow conditions.

$$D_R = 3.402 + 0.00456\, V_R + 0.0048\, V_{12} - 0.01278\, L_A \qquad 6$$

where

D_R = density of merge influence area (pc/km/ln)

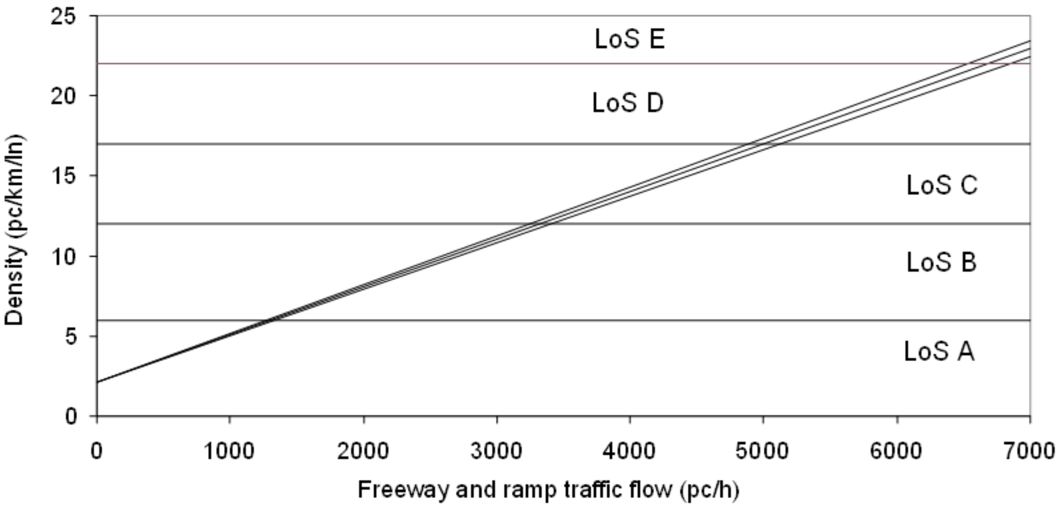
V_R = on-ramp peak 15-min flow rate (pc/h)

V_{12} = flow rate entering ramp influence area (pc/h)

L_A = Length of acceleration lane (m).

The calculated densities for on-ramp traffic onto a three-lane freeway carriageway are shown in Figure 4.6. The three lines represent the merging traffic being 5%, 10% and 15% of the total traffic. The densities are reasonably independent of the ramp flows when plotted against the total downstream traffic (freeway and ramp flows). This plot is based on Equation 6 with an acceleration lane length of 100m, three lanes on the freeway and upstream and downstream ramps have no influence on the proportion of traffic in the first two lanes. By increasing the acceleration lane by 100 m, the density in the influence area will decrease by 1.3 pc/km/ln.

Figure 4.6: Influence-area density for on-ramp flows of 5%, 10% and 15% of the freeway flows; the acceleration lane length is 100 m



Source: Figure 4.6 in Austroads (2013).

Equation 7 is used to estimate the density within the diverge influence area, replicated from HCM 2000 Equation 25-10.

$$D_R = 2.642 + 0.0053\ V_{12} - 0.0183\ L_D \quad 7$$

where

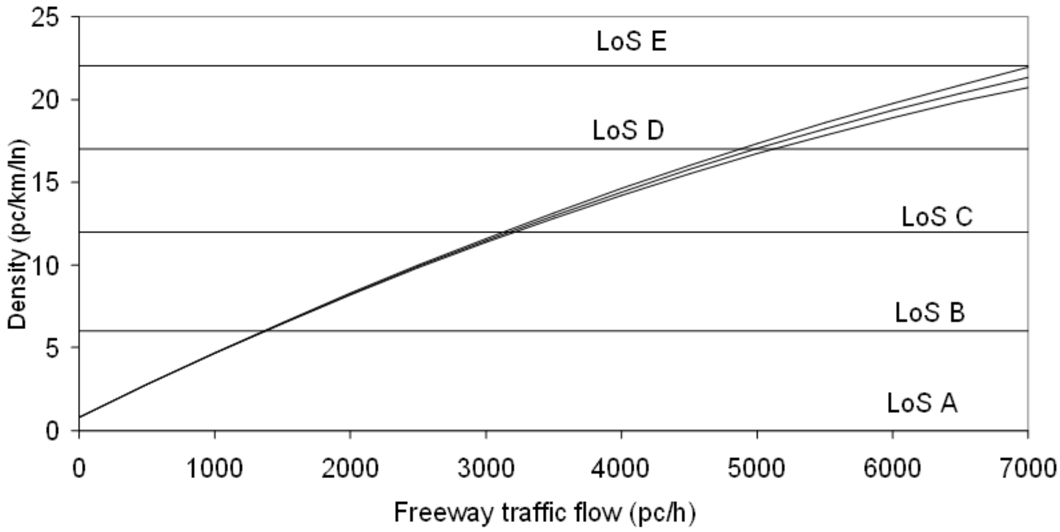
D_R = density of diverge influence area (pc/km/ln),

V_{12} = flow rate entering ramp influence area (pc/h), and

L_D = length of deceleration lane (m)

For an off-ramp, the calculated densities in the influence area of a three-lane freeway are shown in Figure 4.7. The three lines represent the diverging traffic being 5%, 10% and 15% of the total traffic. Again, changing the off-ramp traffic flow has only a marginal effect on the traffic densities. This plot is based on Equation 7 with a deceleration lane length of 100 m, three lanes on the freeway and upstream and downstream ramps have no influence on the proportion of traffic in the first two lanes. By increasing the deceleration lane length by 100 m, the density in the influence area will decrease by 1.8 pc/km/ln.

Figure 4.7: Influence-area density for off-ramp flows of 5%, 10% and 15% of the freeway flows; the deceleration lane length is 100 m



Source: Figure 4.7 in Austroads (2013).

Capacities of the freeway approaching the diverge area, departing from the merge or diverge area, and of the ramp and the influence area, are given in HCM 2016 (Chapter 14). The HCM 2016 lists the capacity of ramp roadways and ramp freeways, as reproduced in Table 4.6 and Table 4.7. It is assumed in HCM 2016 that the capacity of a basic freeway segment under base conditions is 2400 pc/h/ln (FFS = 120 km/h) and 2300 pc/h/ln (FFS = 100 km/h).

As mentioned, the capacities in Table 4.6 and Table 4.7 the geometric capacities before flow breakdowns and are more suitable for strategic planning purposes. The maximum flow on a freeway falls below the geometric capacity due to flow breakdowns during peak-flow periods and VicRoads (2013) has reported the use of ramp metering to achieve an operational capacity of 2100 pc/h at an FFS = 100 km/h.

Table 4.6: Approximate capacity of ramp roadways in passenger cars/hour

Free-flow speed of ramp, SFR (km/h)	Capacity (pc/h)(1)	
	Single-lane ramps	Two-lane ramps
> 80	2200	4400
> 65–80	2100	4200
> 50–65	2000	4000
≥ 30–50	1900	3800
< 30	1800	3600

1 *The operational capacity is less than the value indicated above when flow breakdown occurs.*

Source: Exhibit 14-12 in HCM 2016 (TRB 2016).

Table 4.7: Capacity values for merge and diverge areas in passenger cars/hour

Freeway free-flow speed (km/h)	Capacity of upstream/downstream freeway segment (pc/h)(1)(3)				Max desirable flow entering merge influence area V_{R12} (pc/h)(2)	Max desirable flow entering diverge influence diverge area V_{12} (pc/h)(2)
	Number of lanes in one direction					
	2	3	4	> 4		
120	4800	7200	9600	2400/ln	4600	4400
110	4700	7050	9400	2350/ln	4600	4400
100	4600	6900	9200	2300/ln	4600	4400
90	4500	6750	9000	2250/ln	4600	4400

1 *Demand in excess of these capacities results in LOS F.*

2 *Demand in excess of these values does not result in LOS F; operations may be worse than predicted by this methodology.*

3 *The operational capacity is less than the value indicated above when flow breakdown occurs.*

Source: Exhibit 14-10 in HCM 2016 (TRB 2016).

The LOS is estimated from the densities of vehicles in the influence area. Appropriate densities from HCM 2016 are given in Table 4.8.

Table 4.8: LOS criteria for freeway merge and diverge segments

LOS	Density (pc/km/ln)
A	≤ 6
B	> 6–12
C	> 12–17
D	> 17–22
E	> 22
F	Demand exceeds capacity

Source: Exhibit 14-3 in HCM 2016 (TRB 2016).

HCM 2016 notes that LOS in merge and diverge influence areas are defined in terms of density for all cases of stable operation, from LOS A to E. LOS F exists when the demand exceeds the capacity of upstream or downstream freeway sections or the capacity of ramps. The different LOS are as follows:

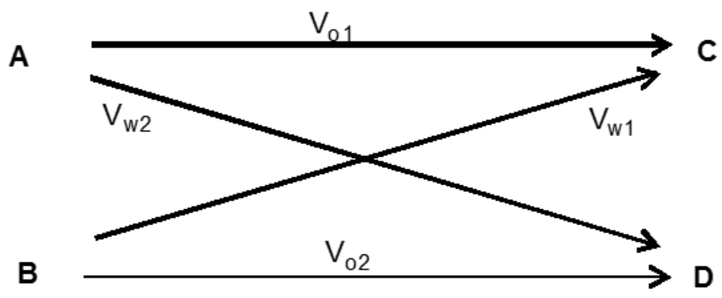
- LOS A represents unrestricted operations. Density is low enough to permit smooth merging and diverging, with virtually no turbulence in the traffic stream.
- At LOS B, merging and diverging manoeuvres become noticeable to through drivers, and minimal turbulence occurs. Merging drivers must adjust speeds to accomplish smooth transitions from the acceleration lane to the freeway.
- At LOS C, speed within the influence area begins to decline as turbulence levels become noticeable. Both ramp and freeway vehicles begin to adjust their speeds to accomplish smooth transitions.
- At LOS D, turbulence levels in the influence area become intrusive and virtually all vehicles slow to accommodate merging and diverging. Some ramp queues may form at heavily used on-ramps, but freeway operation remains stable.
- LOS E represents conditions approaching capacity. Speeds reduce significantly, and turbulence is felt by virtually all drivers. Flow levels approach capacity, and small changes in demand or disruptions within the traffic stream can cause both ramp and freeway queues to form.

The HCM 2016 process allows for closely spaced ramps, and ramps with more than one lane. The manual should be consulted for details about the analysis process.

4.4.3 Weaving Sections

The analysis of weaving sections is less developed than other freeway analysis procedures. The analysis process requires the weaving and the non-weaving traffic to be estimated. This is shown diagrammatically in Figure 4.8.

Figure 4.8: Diagrammatic representation of traffic on freeway weaving areas



Source: Adapted from Exhibit 13-9 in HCM 2016 (TRB 2016).

In this figure, V_{o1} and V_{o2} are the outer, non-weaving flow rates and V_{w1} and V_{w2} are the two weaving flow rates.

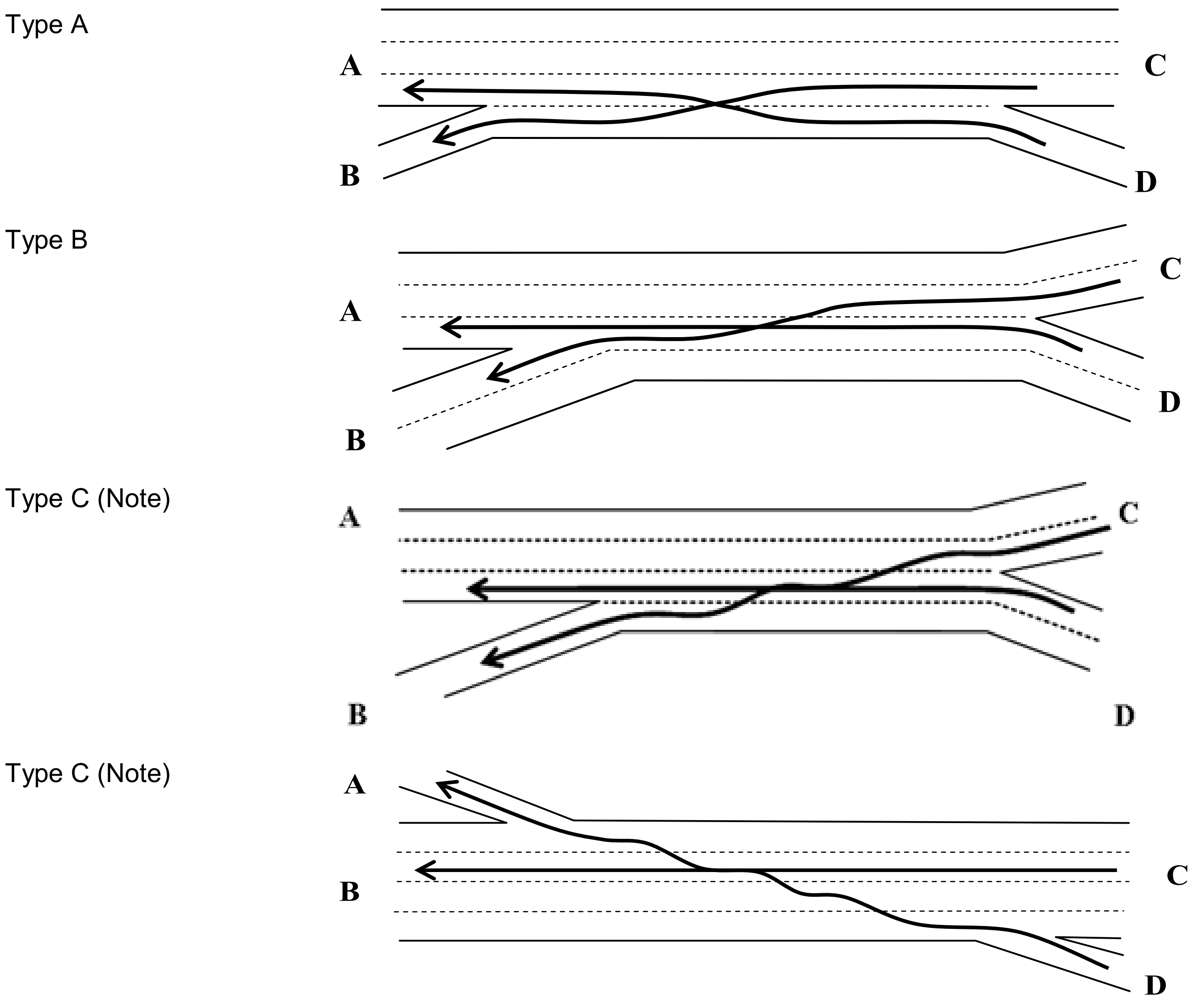
The number of lanes crossed by both weaving traffic flows is used to classify weaving sections. HCM 2016 (Chapter 13) defines the different types of weaving sections as follows, and these are illustrated in Figure 4.9.

- *Type A weaving segments* – all weaving vehicles must make one lane change to complete their manoeuvre successfully. All of these lane changes occur across a lane line that connects from the entrance gore area directly to the exit gore area. The most common form of Type A weaving segment is formed by a one-lane on-ramp followed by a one-lane off-ramp, with the two connected by a continuous auxiliary lane. All on-ramp vehicles entering the freeway must make a lane change from the auxiliary lane to the shoulder lane of the freeway. All freeway vehicles exiting at the off-ramp must make a lane change from the shoulder lane of the freeway to the auxiliary lane. This type of configuration is also referred to as a ramp-weave.

- *Type B weaving segments* – one weaving movement can be made without making any lane changes, and the other weaving movement requires at most one lane change.
- *Type C weaving segments* – similar to those of Type B in that one or more through lanes are provided for one of the weaving movements. One weaving movement requires a minimum of two lane changes for successful completion of a weaving manoeuvre while the other movement can be made without making a lane change.

The configuration of the weaving segment has a marked effect on operations because of its influence on lane-changing behaviour. A weaving segment with 1000 veh/h weaving across 1000 veh/h in the other direction requires at least 2000 lane changes per hour in a Type A segment, since each vehicle makes one lane change. In a Type B segment, only one movement must change lanes, reducing the number of required lane changes per hour to 1000. In a Type C segment, one weaving flow would not have to change lanes, while the other would have to make at least two lane changes, for a total of (at least) 2000 lane changes per hour.

Figure 4.9: Classification of weaving types



Note: Two Type Cs.

Source: Adapted from Exhibit 13-3 and Exhibit 13-4 in HCM 2016 (TRB 2016).

Configuration has a further effect on the proportional use of lanes by weaving and non-weaving vehicles. Since weaving vehicles must occupy specific lanes to efficiently complete their manoeuvres, the configuration can limit the ability of weaving vehicles to use outer lanes of the segment. This effect is most pronounced for Type A segments because weaving vehicles must primarily occupy the two lanes adjacent to the crown line. It is least severe for Type B segments since these segments require the fewest lane changes for weaving vehicles, thus allowing more flexibility in lane use.

The length and width of the weaving segment are two geometric parameters that describe the area used by weaving vehicles. The weaving length is defined in HCM 2016 as 'measured from a point at the merge gore where the right edge of the freeway shoulder lane and the left edge of the merging lane(s) are 0.6 m apart to a point at the diverge gore where the two edges are 3.7 m apart'. All weaving vehicles must make their lane changing within the length of the weaving segment. If the length of a weaving segment decreases, the intensity of lane changing and the resulting turbulence increases. The weaving width is the number of lanes affected or influenced by the weaving traffic.

The analysis process estimates the average speed of the weaving and non-weaving traffic assuming that the weave is unconstrained. This assumption is later tested and adjusted if necessary. Given the average speeds and flows, the traffic density is calculated and compared with values listed in Figure 4.9.

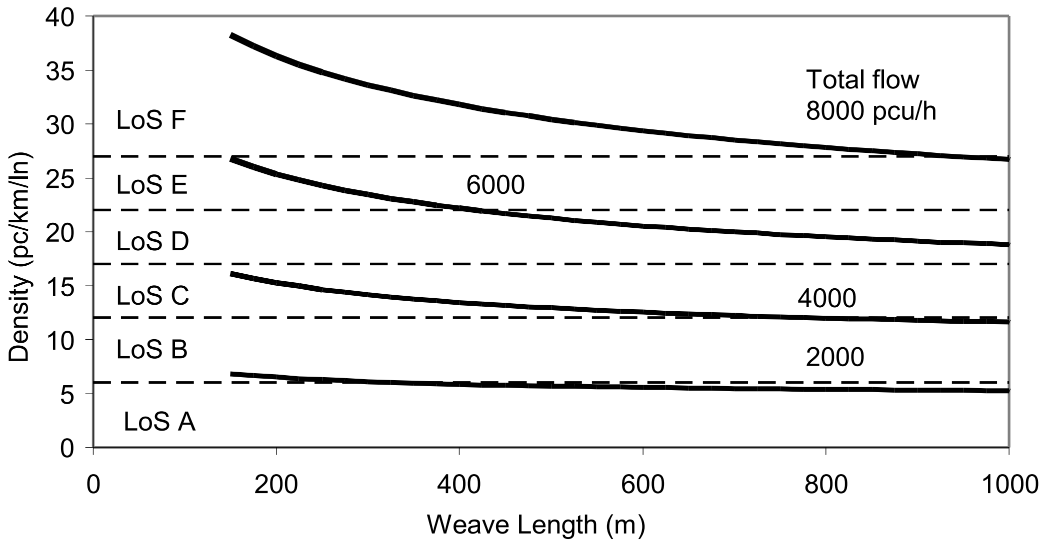
Table 4.9: LOS criteria for weaving segments

LOS	Density (pc/km/ln)	
	Freeway weaving segment	Multi-lane and collector-distributor weaving segments
A	≤ 6.0	≤ 8.0
B	> 6.0–12.0	> 8.0–15.0
C	> 12.0–17.0	> 15.0–20.0
D	> 17.0–22.0	> 20.0–23.0
E	> 22.0-27	> 23.0-25
F	> 27 or demand exceeds capacity	> 25 or demand exceeds capacity

Source: Exhibit 13-6 in HCM 2016 (TRB 2016).

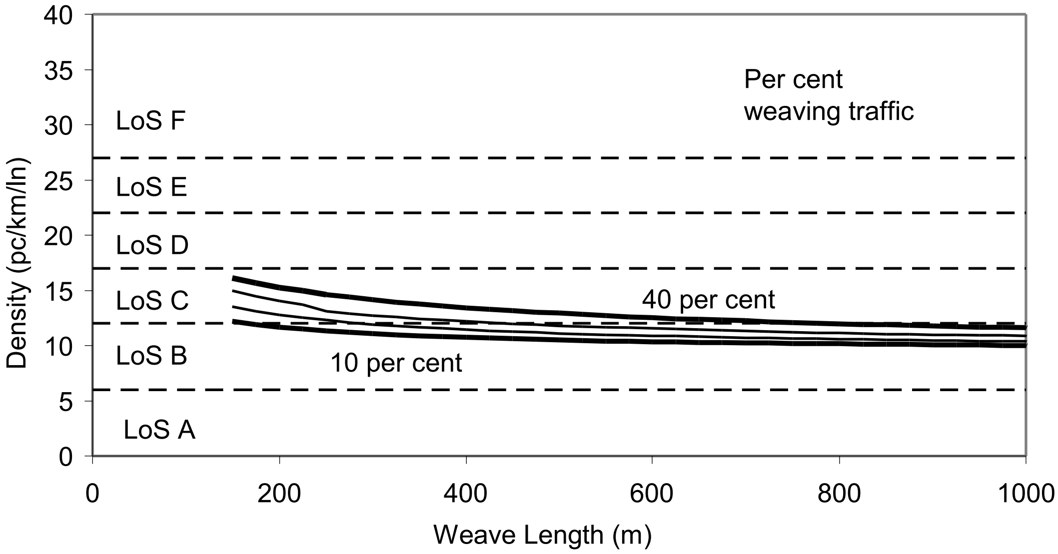
Figure 4.10 illustrates the predicted densities when the weaving traffic is 40% of the total traffic. The different curves relate to different total traffic. The weaving section has four lanes and is Type A. The densities and the LOS are largely dependent on the total traffic and to a lesser extent on the length of the weaving section. Figure 4.11 demonstrates that the effect of changing the proportion of weaving traffic is marginal.

Figure 4.10: LOS for weaving sections of Type A, with 40% weaving traffic and varying total traffic



Source: Figure 4.10 in Austroads (2013).

Figure 4.11: LOS for weaving sections of Type A, with 4000 pc/h total traffic and with different proportions of weaving traffic



Source: Figure 4.11 in Austroads (2013).

5. Interrupted Flow Facilities

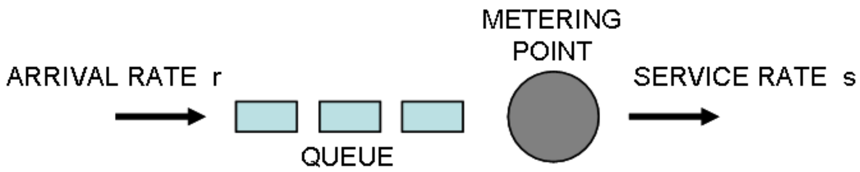
5.1 Metered Flow

Metered flow occurs on interrupted flow facilities when vehicles arriving at a given point on the road are required to pass through it in accordance with certain rules. Typical examples of metered flow are at toll booths, at various drive-in facilities (such as bottle shops, bank-teller points, take-away food outlets and parking stations), at points where a reduction in the number of traffic lanes requires traffic to merge, and at freeway entrance ramps where a traffic control signal may be used to restrict the volume of traffic entering the freeway. Basically, metered flow is a queuing situation, and queuing theory may be applied to it.

Details of queuing theory applicable to this section are given in Austroads (2015a).

A typical metered flow situation is shown in Figure 5.1 Vehicles arrive at the metering point at an average arrival rate of r vehicles per second. They may or may not queue before being serviced depending on whether or not other vehicles are being serviced, and they then pass through the metering point at an average service rate of s vehicles per second.

Figure 5.1: Typical metered flow situation



The main characteristics of the process are as follows:

- the arrival pattern, namely the average arrival rate and the distribution of the arrivals
- the number of service channels; there may be one service channel in the case of two lanes merging into one lane or at a metered freeway entrance ramp, and there may be one or several at a toll booth or at a drive-in facility
- the queue discipline; first-come first-served as usual, although in special cases other disciplines may be used, for example, buses or high-occupancy vehicles may be given priority
- the service pattern, namely the average service rate and the distribution of the service times.

5.1.1 Capacity

Under stable flow conditions, the capacity of a metering point is given by Equation 8:

$$C = 3600\ s \qquad (8)$$

where

C = capacity in pc/h

s = average service rate in pc per second.

At a point where two lanes merge into one lane, the average service time per vehicle may vary from 1.5 seconds if vehicles are not required to queue to about 2.0 seconds if a queue develops and stop-start conditions result. Corresponding capacities are 2400 pc/h and 1800 pc/h respectively.

At toll booths, the average service time per vehicle may vary from 4.5 to 15 seconds depending on the collection arrangements. Corresponding capacities per booth are 800 vehicles per hour and 250 vehicles per hour. AS 2890.1 (*Parking Facilities: Off-street Car Parking*) gives the following capacities:

- automatic ticket issuing and boom gate 400 pc/h/ln
- manually operated 250 pc/h/ln.

In other metering situations, the average service time per vehicle and hence channel capacity will depend on the particular case. At existing facilities, service times may be measured directly, and for proposed facilities service times may be estimated or preferably obtained by direct measurement at comparable existing facilities.

Factors affecting capacity

In general, the actual servicing operation (such as, for example, toll collection) usually has by far the most significant effect on the service time in a metering situation. However, in some cases, and particularly when service times are relatively small (say less than 3 or 4 seconds), the departure lane width and grade, and the presence of heavy vehicles may adversely affect the service time.

Each case should be considered on its merits, and if service times are likely to be small, some small allowance should be made for the effect of departure lanes less than 3 m wide, and of upgrades steeper than 3%. As a guide, the allowances for these factors at signalised intersections, given in Section 6, may be used in the absence of actual measured data from other comparable sites. Also, as an approximation, each heavy vehicle may be considered equivalent to two passenger cars. However, it should be noted that such adjustments are necessary only for small service times. Except in unusual cases, these types of adjustments should not be necessary for service times over about 4 seconds, and certainly not when they exceed 6 seconds.

5.1.2 Queue Lengths and Delays

For many metering situations, it is satisfactory to treat the arrival of vehicles at the metering point as random arrivals. In most situations, vehicles are usually serviced on a first-come first-served basis, and it can be assumed that service times follow a negative exponential distribution.

On this basis, for a single service channel the following relationships apply using the same vehicle type:

- average arrival rate: r vehicles per second
- average service rate: s vehicles per second
- utilisation factor (Equation 9):

$$\rho = r / s \qquad 9$$

- probability of n vehicles in the system, including the one being serviced (Equation 10):

$$P(n) = (1 - \rho)\, \rho^{n} \qquad 10$$

- probability of more than n vehicles in the system, including the one being serviced (Equation 11):

$$P(>n) = \rho^{n+1} \qquad 11$$

- the mean queue length, including the vehicle being serviced (Equation 12):

$$n_q = \rho / (1- \rho) \qquad 12$$

- the variance of the mean queue length, including the vehicle being serviced (Equation 13):

$$\sigma^2 (n) = \rho / (1 - \rho)^2 \qquad 13$$

- the mean waiting time (delay) in the system, including the time being serviced (Equation 14):

$$W_m = n_q / r = 1 / s (1 - \rho) \text{ seconds} \qquad 14$$

As the flow through a metering point approaches capacity, the utilisation factor ρ approaches 1 and this situation is associated with long queues and long waiting times. As far as practicable, metering points should be designed so that the utilisation factor does not exceed about 0.8. A worked example is given in Commentary C1.2.1.

5.2 Urban Arterial Roads with Interrupted Flow

5.2.1 Capacity

Table 5.1 sets out typical mid-block capacities for various types of urban roads with interrupted flow, with unflared major intersections and with interruptions from cross and turning traffic at minor intersections.

When improvements to isolated intersections are being considered, but without any change to upstream conditions, the figures in Table 5.1 can be taken as limiting values.

Table 5.1: Typical mid-block capacities for urban roads with interrupted flow

Type of lane	One-way mid-block capacity (pc/h)
Median or inner lane	
Divided road	1000
Undivided road	900
Middle lane (of a 3 lane carriageway)	
Divided road	900
Undivided road	1000
Kerb lane	
Adjacent to parking lane	900
Occasional parked vehicles	600
Clearway conditions	900

Source: Table 5.1 in Austroads (2013).

Peak-period mid-block traffic volumes may increase to 1200 to 1400 pc/h/ln on any approach road when the following conditions exist or can be implemented:

- adequate flaring at major upstream intersections
- uninterrupted flow from a wider carriageway upstream of an intersection approach and flowing at capacity
- control or absence of crossing or entering traffic at minor intersections by major road priority controls
- control or absence of parking
- control or absence of right turns by banning turning at difficult intersections
- high-volume flows of traffic from upstream intersections during more than one phase of a signal cycle
- good co-ordination of traffic signals along the route.

5.2.2 Descriptions of Levels of Service

The traffic conditions associated with the various levels of service for urban and suburban arterial roads with interrupted flow conditions are described in HCM 2016 as follows:

- LOS A describes primarily free-flow operation. Vehicles are completely unimpeded in their ability to manoeuvre within the traffic stream. Control delay at the boundary intersections is minimal. The travel speed exceeds 80% of the BFFS.
- LOS B describes reasonably unimpeded operation. The ability to manoeuvre within the traffic stream is only slightly restricted and control delay at the boundary intersections is not significant. The travel speed is between 67% and 85% of the BFFS.
- LOS C describes stable operation. The ability to manoeuvre and change lanes at mid- segment locations may be more restricted than at LOS B. Longer queues at the boundary intersections may contribute to lower travel speeds. The travel speed is between 50% and 67% of the BFFS.
- LOS D indicates a less stable condition in which small increases in flow may cause substantial increases in delay and decreases in travel speed. This operation may be due to adverse signal progression, high volume, or inappropriate signal timing at the boundary intersections. The travel speed is between 40% and 50% of the BFFS.
- LOS E is characterised by unstable operation and significant delay. Such operations may be due to some combination of adverse progression, high volume, and inappropriate signal timing at the boundary intersections. The travel speed is between 30% and 40% of the BFFS.
- LOS F is characterised by flow at extremely low speed. Congestion is likely occurring at the boundary intersections, as indicated by high delay and extensive queuing. The travel speed is 30% or less of the BFFS.
- LOS F is assigned to the subject direction of travel if the through movement at one or more boundary intersections has a VCR ratio greater than 1.0.

6. Intersections

In general, the more important unsignalised and signalised intersections determine the overall capacity and traffic performance of interrupted flow facilities. Analysis of these types of intersections usually involves consideration of their capacity and of the mid-block or route capacity of the approach roads. Intersection selection and analysis involves consideration of safety, operational performance or other factors in order to maximise safe mobility (i.e. the safest practicable treatment that also provides an acceptable level of mobility).

The capacity analysis of an intersection may be based on existing traffic volumes or on estimated future traffic volumes depending on the particular problem being addressed. For example, if the objective is to correct an existing deficiency or to review existing traffic signal timings, present-day peak-period volumes would be appropriate. However, if the objective is to prepare an ultimate design in order to decide possible future land use requirements, estimated future traffic volumes should be used.

If future traffic volumes are to be used, they may be obtained as follows:

- use traffic modelling techniques to produce estimates of future traffic volumes and turning movements
- alternatively, if sufficient information is not available for modelling, or if it is thought to be inappropriate
  - determine present-day peak-period traffic volumes and turning movements
  - determine the critical approach road
  - estimate the potential future capacity of the critical approach road when fully developed
  - factor up the existing traffic volumes and turning movements at the intersection by a factor equal to the potential future capacity of the critical approach road divided by the existing volume on it.

Even if the first method is used, it should be checked by the second to ensure that estimated future approach-road volumes at the intersection are realistic.

In certain circumstances, the installation of traffic signals may improve capacity, traffic operation and/or safety at an intersection.

Although each case should be considered on its merits, typical guidelines for the installation of traffic signals are discussed in Section 2 Austroads (2017b).

Information on the design and traffic management of both unsignalised and signalised intersections is given in the *Guide to Road Design Part 4* (Austroads (2017d) and *Guide to Traffic Management Part 6* (Austroads 2017b) respectively. Pedestrians are covered as a generic group but should be taken to include all road users that are deemed to be pedestrians under legislation that applies to the particular jurisdiction (e.g. people using wheelchairs).

6.1 Unsignalised Intersections

This section considers the capacity of unsignalised intersections. The capacity of signalised intersections is dealt with in Section 6.4. Refer also to Section 5.5 in *Guide to Traffic Management Part 2* (Austroads 2015a) for example applications of gap acceptance analysis to unsignalised intersections.

The operation of unsignalised intersections was extensively studied in the USA in the 1990s and the results are included in HCM 2016. Unsignalised intersections are one of the most complicated type to analyse and there have been a number of approaches developed over the years. All approaches have their advantages and disadvantages, and in most cases it can only effectively be evaluated with a computer program.

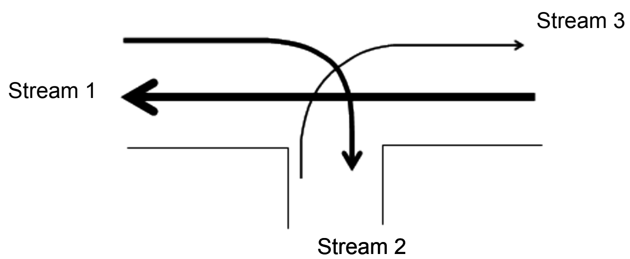
In Australia and New Zealand, the analysis of unsignalised intersections should be evaluated using SIDRA Intersection (Akçelik and Associates 2011) or an equivalent program. This section documents the concepts used in the analysis of unsignalised intersections.

The importance of unsignalised intersections in the context of traffic impact assessment (refer to *Guide to Traffic Management Part 12* (Austroads 2016c)), traffic management for intersections (refer to *Guide to Traffic Management Part 6* (Austroads 2017b)) and the road safety implications should not be overlooked.

Determining the conflicting streams

Unsignalised intersections have a number of conflicting streams that have an order of priority. For instance, at a T-intersection with three vehicular streams (Figure 6.1), stream 1 has the highest priority, being on the major road. Stream 2 has the second priority as drivers in this stream need to give way to stream 1 vehicles. Stream 3 has the lowest priority, as stream 3 drivers need to give way to stream 2 vehicles, which in turn have to give way to stream 1 vehicles.

Figure 6.1: Major and minor streams at a T-intersection

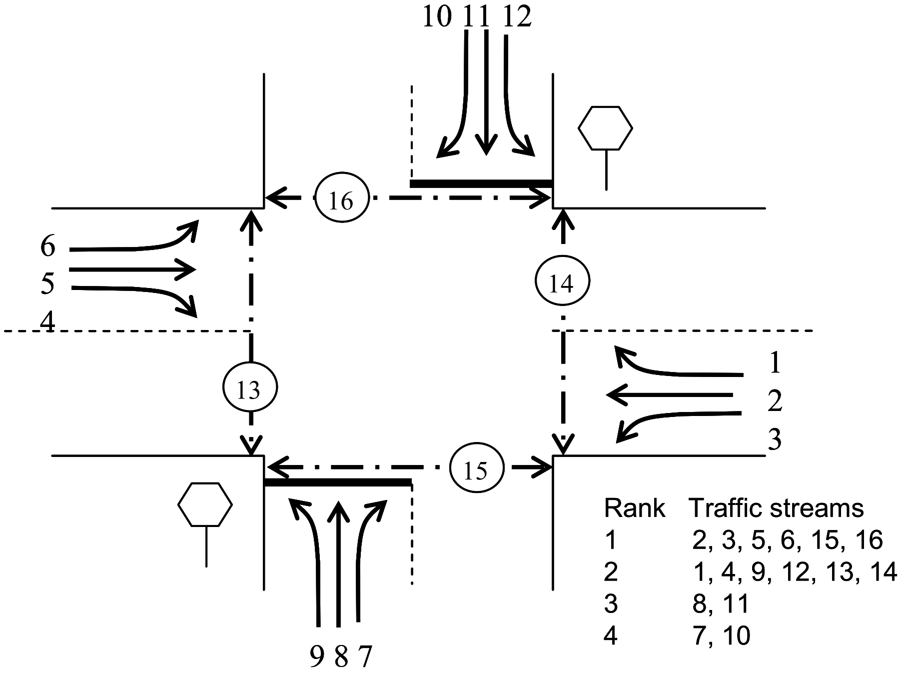


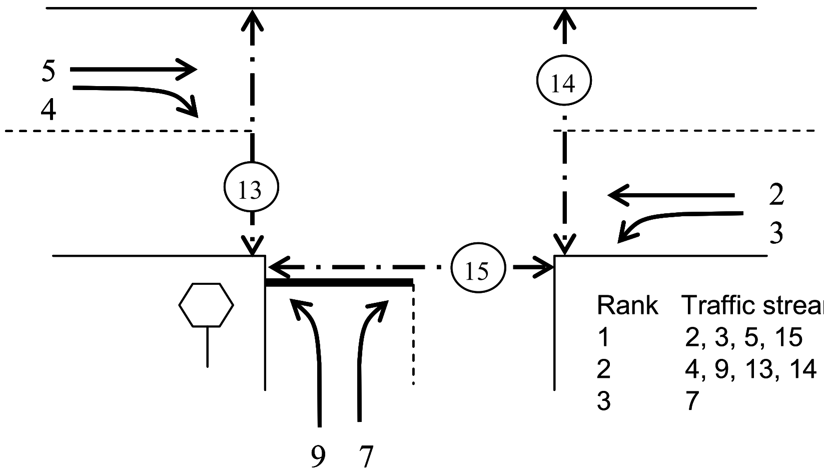
Source: Adapted from Exhibit 20-7 in HCM 2016.

When there are only vehicular movements, then the priority of each stream can be determined by the driving rules. If there are pedestrian movements then the order of priority is more difficult. HCM 2016 refers to the different levels of priority as the 'rank' of the traffic stream. Figure 6.2 illustrates the rank of each stream at a cross-intersection.

In this figure, streams 3 (left turn from the major road) and 15 (pedestrian movement) have equal priority. However, it might be questioned whether the left turners have, or take priority over the pedestrian movement, or whether it is the other way around and pedestrian movement takes priority over left turners. In fact, driver and pedestrian behaviour would suggest that at different times each stream may have priority over the other. Consequently, the order of the priority of streams is not clear and if the analysis is to be as realistic as possible, then there may a need to accommodate a range of behaviours. Brilon and Wu (2002) have developed a probabilistic method that allows for this changing priority phenomenon.

Figure 6.2: Traffic stream ranks at two-way stop-controlled intersections





Source: Adapted from Exhibit 20-1 in HCM 2016.

Critical acceptance headway and follow-up headway

The critical acceptance headway and the follow-up headway are two basic terms which describe how a driver will decide whether to depart or not. The critical acceptance headway describes the assumed minimum headway between conflicting vehicles that is acceptable to an entering minor-stream driver (in a stream of rank 2, 3 or 4). It is assumed that all drivers are consistent and behave in the same way. The follow-up headway is the assumed headway between minor-stream drivers in the one stream that depart in a longer acceptable headway between vehicles in higher-ranking streams.

Values of the critical acceptance headway are difficult to measure in an unbiased fashion and the best method is a maximum likelihood method (Brilon et al. 1999; Troutbeck 1992). The critical acceptance headway and follow-up time values used in HCM 2016 represent well evaluated values based on the observations under the US driving conditions (Kyte et al. 1996). Table 6.1 provides SIDRA intersection default values of t_c and t_f for passenger cars based on values recommended in Austroads *Road Design Guide Part 4A* (Austroads 2017e).

HCM 2016 explains (in HCM 2016 Equations 20-30 and 20-31) the critical headway and follow-up times for movement x as (Equation 15):

$$X = t_c = t_{c,base} + t_{c,HV} P_{HV} + - t_{c,T} - t_{3,LT} \qquad 15$$

and

$$t_f = t_{f,base} + t_{f,HV} P_{HV} \qquad 16$$

where

$t_{c,x}$	=	critical headway (s)
$t_{c,base}$	=	base critical headway
$t_{c,HV}$	=	adjustment factor for heavy vehicles (1.0 for two-lane major streets and 2.0 for four-lane major streets) (s)
P_{HV}	=	proportion of heavy vehicles for minor movement
$t_{c,T}$	=	adjustment factor for each part of a two-stage gap acceptance process (1.0 for first or second stage; 0.0 if only one stage) (s)
$t_{3,LT}$	=	adjustment factor for intersection geometry (0.7 for minor-street right-turn movement at three-leg intersection; 0.0 otherwise) (s)
t_f	=	follow-up time for minor movement x (s)
$t_{f,base}$	=	base follow-up time
$t_{f,HV}$	=	adjustment factor for heavy vehicles (0.9 for two-lane major streets and 1.0 for four-lane major streets)
PHV	=	proportion of heavy vehicles for minor movement.

Table 6.1: Critical gap and follow-up headway parameters for two-way sign-controlled intersections in SIDRA intersection (template setting for 4-way intersections)

Standard driving on the left model						
	2-lane major road			4-lane major road		
STOP sign	t_c	t_f	s	t_c	t_f	s
Minor road: left turn	4.5	2.5	1440	5.0	3.0	1200
• through	5.0	3.0	1200	6.5	3.5	1029
• right turn	5.5	3.5	1029	7.0	4.0	900
Right turn from major road	4.0	2.0	1800	4.5	2.5	1440
GIVE-WAY / YIELD sign	t_c	t_f	s	t_c	t_f	s
Minor road: left turn	4.0	2.2	1636	4.5	2.7	1333
• through	4.5	2.7	1333	6.0	3.2	1125
• right turn	5.0	3.2	1125	6.5	3.7	973
Right turn from major road	4.0	2.0	1800	4.5	2.5	1440

Source: SIDRA Intersection User Guide (Akçelik & Associates 2011).

Impedance factors

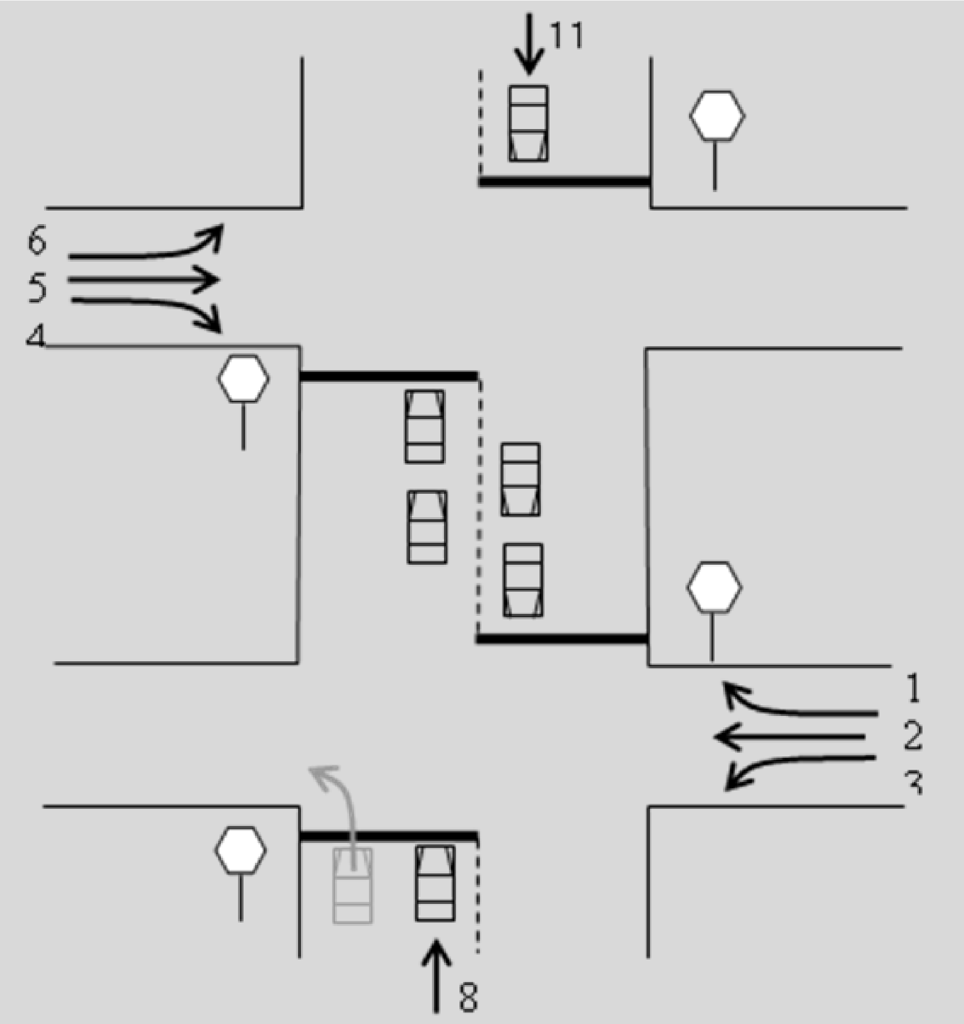
Rank 3 and 4 streams are impeded by the queuing in the streams of higher ranks (1 and 2). A rank 3 vehicle cannot depart unless the queue of rank 2 vehicles has cleared. A process to account for the impedance factors was developed by Bennett in the 1980s and a description is given in Bennett (2003).

Essentially the capacity is first calculated based on the total conflicting flow. This flow is adjusted to account for queuing in the higher-ranking streams (which will have priority). The capacity adjustment factors for rank 3 streams (Figure 6.2) are the product of the probabilities that the conflicting rank 2 streams will not have a queue. The capacity adjustment factor for the rank 4 streams, namely the right turns from the minor street at a cross-intersection, is also given by the product of the probabilities that the conflicting rank 2 and 3 streams will not have a queue, and by an additional adjustment factor. This additional factor, for the impedance to rank 4 streams, is not a simple one and was first found by a review of output from a large number of simulation runs (it has later been established theoretically). There has been some discussion that the impedance factors are not required. Kyte et al. (1996) found that the use of impedance factors improved the estimates of capacity recorded in field observations.

Two-stage crossing

HCM 2016 provides for a two-stage headway acceptance procedure. The two-stage process is illustrated in Figure 6.3. Right turners from the major road and through and right turners from the minor road can cross one carriageway at a time. At each stage, the number of conflicting vehicles is less than it would be if the median was not available for storage. However, the capacity is restricted to some extent as the median has limited storage. The addition of the first space provides the greatest advantage, with each subsequent space in the median providing less additional increase in capacity.

Figure 6.3: Two-stage headway acceptance analysis



Source: Adapted from Exhibit 17-3 in HCM 2000 (TRB 2000).

Levels of service criteria

HCM 2016 judges the LOS on the average control delay as shown in Table 6.2 HCM 2016 states that:

> *The LOS criteria for TWSC (two-way stop-controlled) intersections are somewhat different from the criteria for signalised intersections primarily because different transportation facilities create different driver perceptions. The expectation is that a signalised intersection is designed to carry higher traffic volumes and experience greater delay than an unsignalised intersection.*

Table 6.2: LOS criteria for two-way stop-controlled intersections

LOS	Average control delay (s/veh)
A	0–10
B	> 10–15
C	> 15–25
D	> 25–35
E	> 35–50
F	> 50

Source: Exhibit 20-2 in HCM 2016 (TRB 2016).

Recommended analysis procedures

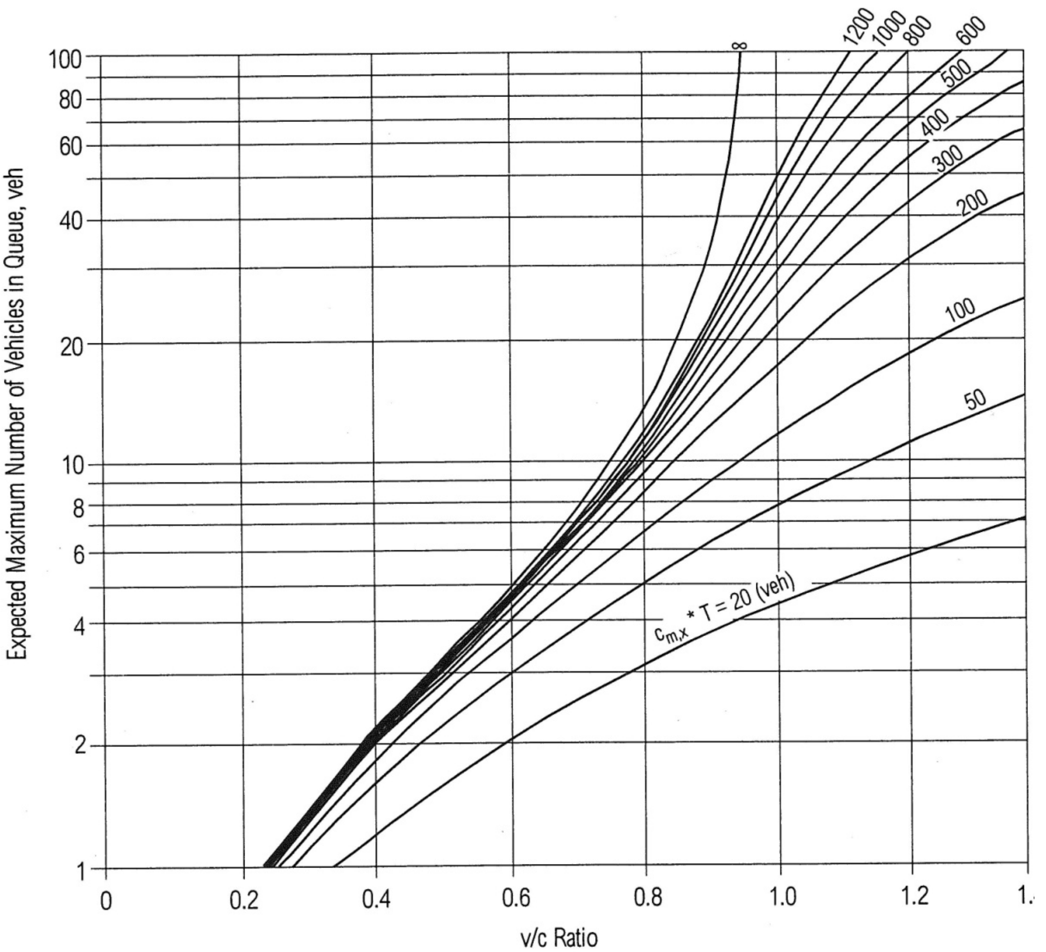
The predicted average control delay is generally used to describe the performance of the intersection and can be used to compare the intersection's performance with other forms of control. The degree of saturation, the flow divided by the capacity, is another measure to quantify the intersection's performance.

The control delay is increased when the conflicting flows are increased and as the critical headway and follow-up times are increased. The critical headway and the follow-up headway tend to be longer as the manoeuvre becomes more difficult and as the subject stream has to give way to more streams.

SIDRA intersection provides a convenient analysis tool. It can account for the effect of short lanes, for environmental effects and for extensive queues. It is recommended that the SIDRA intersection or the HCM 2016 software be used to analyse unsignalised intersections.

HCM 2000 provided a useful graph, and procedure, to estimate the 95th percentile queue length to estimate storage requirements. This figure is reproduced here as Figure 6.4 and uses two basic terms, the VCR ratio and the maximum number of departures, in the analysis period, which is assumed to be 15 minutes.

Figure 6.4: 95th percentile queue lengths



Note: $C_{m,X}$ = capacity of movement x (veh/h), and T = analysis time period (h) (T=0.25 for a 15-min period).

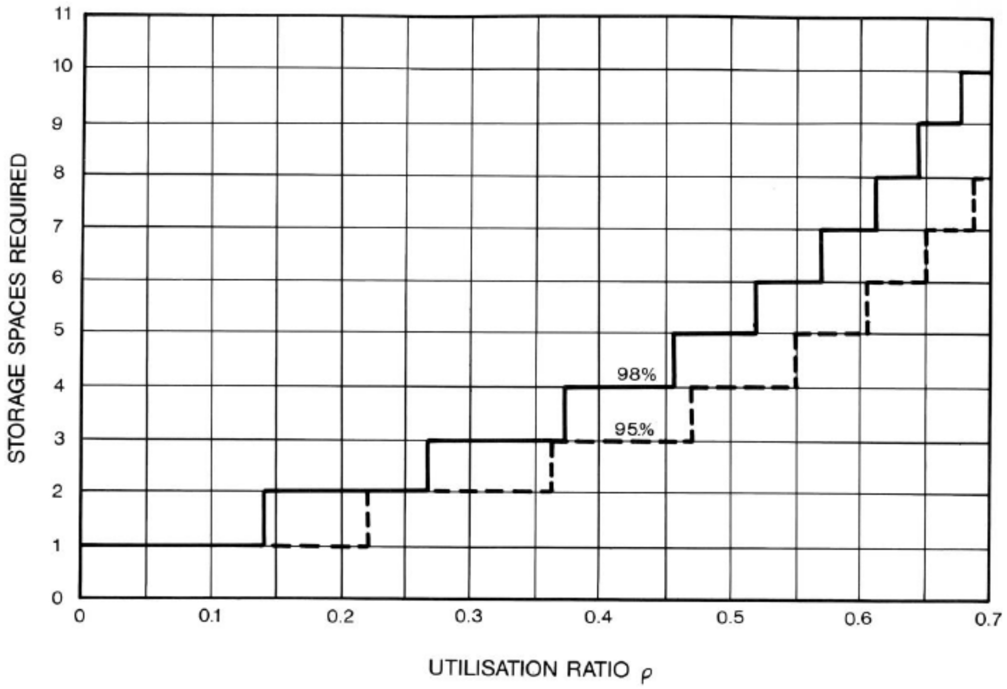
Source:
1. Exhibit 17-19 in HCM 2000 (TRB 2000).
2. Curves follow Equation 20-68 in HCM 2016 (TRB 2016).

Storage requirements

The storage requirements, such as for example the storage length required for an unsignalised right-turn movement may be calculated as follows:

- Determine the minor stream movement lane volume r_m.
- Determine the maximum service rate s for the minor stream movement. Usually this is taken as the absorption capacity C for the movement.
- Calculate the utilisation rate ρ (ratio of the arrival rate to the service rate) i.e. $\rho = r_m / s$.
- Decide on a probability that the mean queue length will not be exceeded. As a minimum 95% should be selected.
- Use Figure 6.5 to determine the maximum queue length appropriate to the selected probability level.
- Allow 6 m for each passenger car and 12 m for each heavy vehicle in the maximum queue length.

Figure 6.5: Unsignalised intersection vehicle storage requirements



Source: Austroads (2013).

Part 6 of *Guide to Traffic Management* (Austroads 2017b) provides guidance on the capacity and flow at unsignalised intersections from a traffic management perspective.

6.2 Roundabouts

Roundabouts are a particular form of unsignalised intersection. Details of their design, operational analysis and traffic management are given in *Part 6* of *Guide to Traffic Management* (Austroads 2017b) and *Part 4* of *Guide to Road Design* (Austroads 2017d). Australian and New Zealand practice for determining the capacity of roundabouts is based on headway acceptance theory. Traffic entering the roundabout gives way to, and accepts opportunities (gaps) in the circulating traffic stream, as depicted in Figure 6.6.

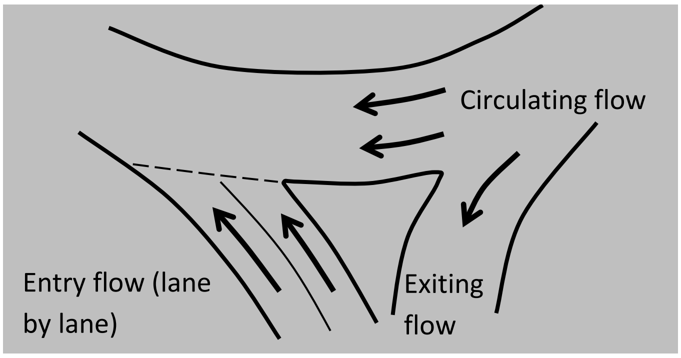
Traditional thought that roundabouts operate as a series of T-intersections may be true under very low demand conditions but there is a high level of interaction between all roundabout entries and circulating flows at medium-to-high-demand flow conditions. The reasons for the interactive nature of roundabout flows are as follows:

- Some entry lanes may be oversaturated, and therefore capacity constraint applies and the downstream circulating flow rate is reduced.
- Unbalanced flow conditions exist, i.e. circulating flow originates mostly from one approach and is highly queued on the approach before entering the roundabout with uniform queue discharge headways, thus affecting the entry capacity adversely.
- Approach lane use conditions of contributing streams from different approaches determine the circulating lane use, and this in turn affects entry capacities of downstream approaches, while at the same time, approach lane use depends on entry-lane capacities.

The current Australian analysis procedure for roundabouts is largely based on field data at Australian roundabouts and analysis procedures developed at ARRB (Akçelik 1981; Troutbeck 1989).

This section describes the basic concepts used in the analysis procedures.

Figure 6.6: Major and minor flows at a roundabout



Source: Austroads (2013).

Previous Austroads guidance on roundabouts based on the research by Troutbeck (1989) recommended that the performance of each lane be analysed separately.

The behaviour of drivers in each entry lane is defined by a critical acceptance headway (previously termed a critical acceptance gap) and the follow-up time. These two terms are correlated and together they are the gap acceptance parameters. The gap acceptance parameters are different for drivers in different entry lanes and are dependent on traffic conditions and roundabout geometry. The lane with the larger flows has shorter gap acceptance parameters and a higher capacity. Equal flows in different entry lanes do not produce equal lane degrees of saturation.

All entering drivers tend to give way to all circulating vehicles. The circulating roadway is modelled by a dichotomised headway model that has a proportion of the circulating vehicles travelling in platoons with short headways of between 1 and 2 seconds. This model is more able to represent the likely platooning in streams.

The capacity and delay estimation procedures are based on traditional gap acceptance equations, which account for a dichotomised headway model for the circulating traffic. The capacity equation is based on the assumption that the major stream, the circulating traffic, does not adjust its speed or relative position when a vehicle enters the roundabout. This approach is still the most common one used in guides around the world (e.g. HCM 2016).

The *SIDRA Intersection User Guide* (Akçelik and Associates 2011) recommended the LOS criteria in Table 6.3 for Australian and New Zealand roundabouts.

Table 6.3: LOS criteria for roundabouts

LOS	Average control delay d (s/veh)
A	d ≤ 10
B	10 < d ≤ 20
C	20 < d ≤ 35
D	35 < d ≤ 50
E	50 < d ≤ 70
F	70 < d

Source: SIDRA Intersection User Guide (Akçelik & Associates 2011).

The gap acceptance parameters would be reduced if the entry flow is much larger than the circulating flow. This effect increases the entry capacity for circulating flows less than 900 pcu/h. Refer to Commentary 2 for a discussion of the development of the analysis procedure.

[see Commentary 2]

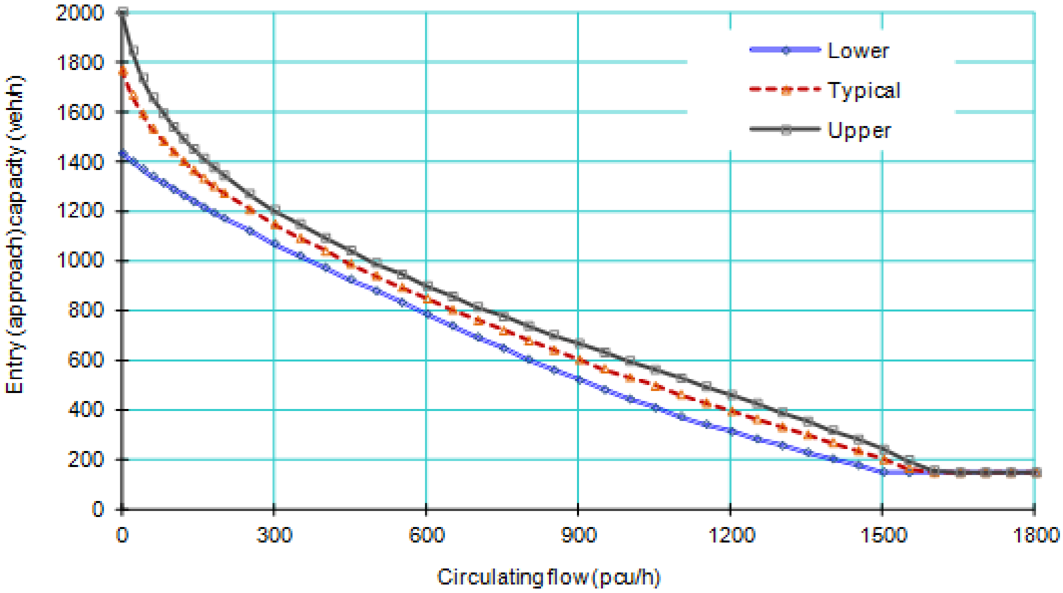
SIDRA intersection produces a range of entry capacities based on the traffic-flow characteristics at other approaches to the roundabout. This range is illustrated in Figure 6.7 (case 1 – single-lane roundabout with an inscribed diameter of 40 m and other roundabout geometry parameters set to default values) and Figure 6.8 (case 2 – two-lane roundabout with an inscribed diameter of 60 m, other roundabout geometry parameters set to default values and ratio of the dominant to the subdominant lane flows of 1.2).

Figure 6.7 and Figure 6.8 include upper and lower-bound capacity estimates as well as a curve representing typical capacity estimates. All graphs were created using an Environment Factor of 1.0 which is default for the 'standard left' model applicable to Australia and New Zealand. All roundabout geometry parameters except inscribed diameter are default values. Figure 6.8 gives the sum of dominant and subdominant lane capacities obtained assuming lane flows which yield equal degrees of saturation. The ranges that define upper bound, lower bound and typical capacity estimates are defined as follows:

- Upper bound: Balanced flow conditions with low levels of queuing on approach lanes, and entry/circulating flow adjustment = high.
- Typical: Moderately unbalanced flow conditions with medium levels of queuing on approach lanes, and entry / circulating flow adjustment = medium (default).
- Lower bound: Highly unbalanced flow conditions with high levels of queuing on approach lanes, and entry / circulating flow adjustment = low.

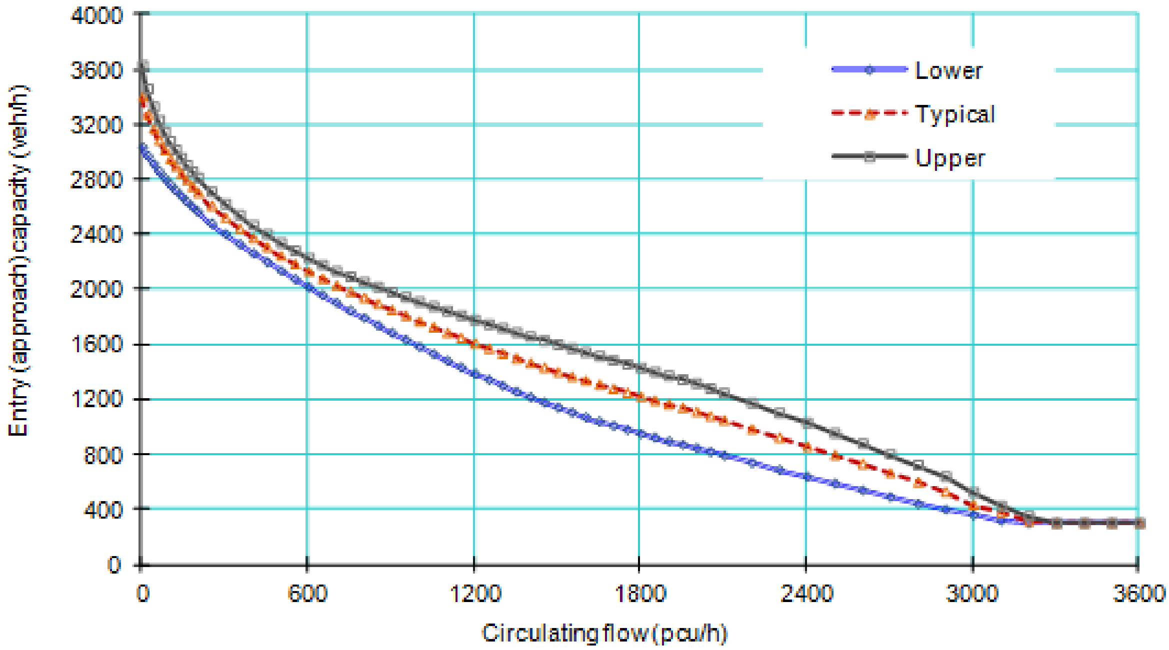
The roundabout capacity model used in SIDRA intersection has been calibrated using extensive field data of critical acceptance gap and follow-up headway values observed at Australian roundabouts. The model is well regarded by the profession and its use promotes consistency in analyses. It is not recommended that the HCM 2016 procedure for roundabout analysis be used at this time.

Figure 6.7: Upper and lower-bound capacity estimates for a single-lane roundabout (with inscribed diameter 40 m and other roundabout geometry parameters set to default values)



Note: Output from SIDRA intersection.

Figure 6.8: Upper and lower-bound capacity estimates for a two-lane roundabout (with inscribed diameter 60 m, other roundabout geometry parameters set to default values, dominant/subdominant lane flow ratio of 1.2)



Note: Output from SIDRA intersection.

6.3 Traffic Simulation and Computer Analysis Models

In addition to the analytical procedures outlined in Section 6.1 and Section 6.2, the performance of unsignalised intersections and roundabouts may be assessed using computer micro-simulation models or analytical tools such as SIDRA intersection. Micro-simulation models are more detailed and time-consuming to users. They require calibration but the modelling of each individual vehicle can provide a more realistic assessment of complex sites. Further discussion on modelling approaches and guidelines can be found in Section 7.

Guide to Traffic Management Part 6 (Austroads 2017b) covers the use, performance, signalisation, road-space allocation, lane management and functional design of a roundabout from a traffic management perspective. Design details are covered in *Parts 4, 4A* and *4B* of the *Guide to Road Design* (Austroads 2017d, Austroads 2017e and Austroads 2015b).

6.4 Signalised Intersections

In certain circumstances, the installation of traffic signals may improve capacity, traffic operation and/or safety at an intersection. Although each case should be considered on its merits, typical guidelines for the selection and installation of traffic signals include consideration of traffic volume, continuous traffic, crashes, pedestrian safety, intersection configuration and traffic management plans. These factors, together with functional layout, road-space allocation, lane management, signal phasing and timings, co-ordination and other information associated with signalised intersections are presented in *Part 6* of the *Guide to Traffic Management* (Austroads 2017b). Austroads (2015e) provides a *Signal Management Toolkit* that includes an indicative impact assessment for a range of signal timing and modal priority techniques. Further information on the standards associated with traffic signals is given in AS1742.14.

6.4.1 Concepts and Definitions

At signalised intersections, capacity calculations tend to focus on individual traffic movements (or lane groups) and individual approaches, rather than on the intersection as a whole. A traffic movement (or lane group) can be defined as one or more lanes on an intersection approach having common right-of-way provision and similar lane-utilisation characteristics.

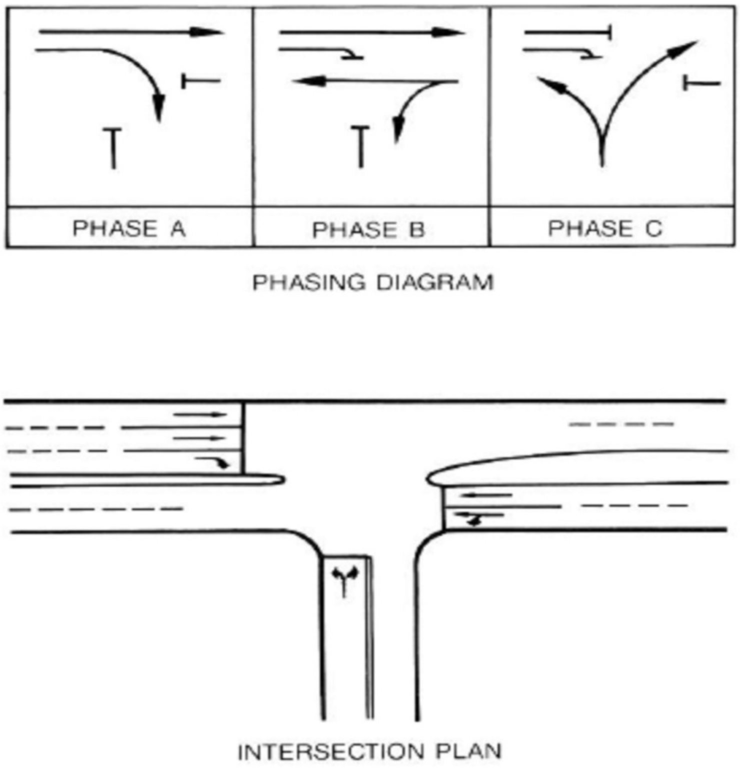
Separate traffic movements (or lane groups) may be as follows:

- an exclusive right-turn lane (or lanes)
- an exclusive left-turn lane (or lanes)
- all straight-through lanes on the approach; all shared lanes (i.e. a combined left-turn and straight-through lane, or a combined right-turn and straight-through lane) may also be included in this group provided they have a similar degree of utilisation – otherwise they can be considered separately.

The allocation of right-of-way to individual movements at an intersection is determined by the traffic signal phasing system. A signal phase is the state of the signals during which one or more movements receive right-of-way. At least one movement receives right-of-way at the start of the phase, and at least one movement loses right-of-way at the end of it. A movement that receives right-of-way in more than one phase is called an overlap movement. One complete sequence of phases is called a signal cycle.

Figure 6.9 shows an intersection and phasing diagram for a three-phase traffic signal installation at a typical T-intersection.

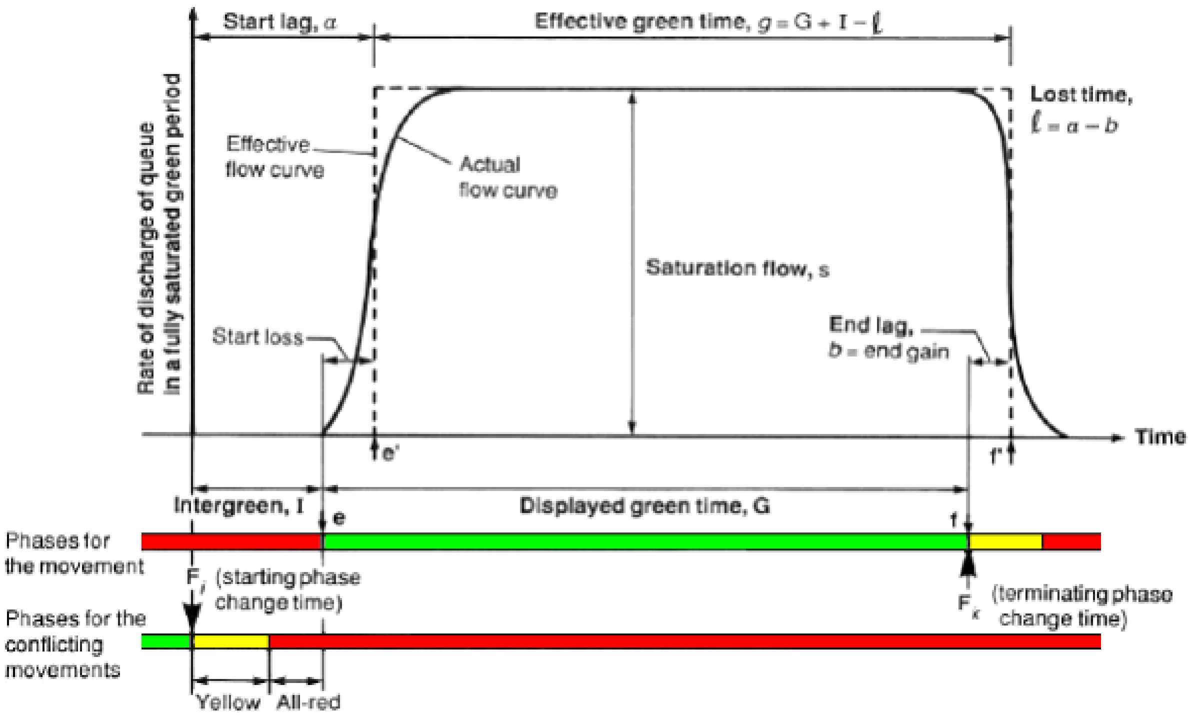
Figure 6.9: Typical intersection plan and phasing diagram



Source: Akçelik (1981).

The basic traffic signal capacity model, which is illustrated in Figure 6.10, assumes that when the signal changes to green, the flow across the stop line increases rapidly to a rate called the saturation flow, which remains constant until either the queue is exhausted or the green period ends.

Figure 6.10: The basic capacity model



Source: Akçelik (1981).

As indicated in Figure 6.10, the basic model replaces the actual departure flow curve by a rectangle of equal area, the height of which is equal to the saturation flow and the width of which is the effective green time. Thus, the area under each curve (i.e. the actual departure flow curve, and the rectangle of equal area) is the maximum number of departures in an average cycle.

As also indicated in Figure 6.10 the inter-green time is the time from the end of the green period for one phase to the start of the green period for the next phase. It is equal to the sum of the amber time plus any all-red time. The cycle time is the sum of all inter-green and green times summed over all phases.

Start and end lag times are also shown in Figure 6.10. The movement lost time is equal to the difference between the start and end lag times, and is equal to the inter-green time plus the difference between the start loss and the end gain. The difference between movement lost time and movement inter-green time will vary from site to site. However, in general, movement lost time can be assumed to be equal to the inter-green time or taken as the inter-green time plus one second.

The movements that determine the capacity and timing requirements of an intersection are called critical movements. If all movements are non-overlap movements, there is one critical movement per phase, i.e. the movement requiring the longest time in that phase. When there are overlap movements, it is necessary to identify the critical movements or those that lie on the critical (or longest) path. Intersection lost time is the total lost time summed over the critical movements.

6.4.2 Capacity Analysis

Most analyses are performed with computer programs which can analyse more complex phasings and intersection layouts. The process presented here is a description of the basic theory and analysis process.

Capacity of a movement

Capacity at signalised intersections is based on the concept of saturation flow. The saturation flow rate may be defined as the maximum rate of flow that can pass through a given traffic movement (or intersection approach) under the prevailing roadway and traffic conditions, usually expressed in vehicles per hour, after taking into consideration the traffic composition and types of turning movements.

The capacity of a movement can be expressed as in Equation 17:

$$C = Sg/c \tag{17}$$

where

C	=	the capacity, in vehicles per hour
S	=	the saturation flow rate, in vehicles per hour
g	=	the effective green time per cycle that is available for the particular movement, in seconds
c	=	the cycle time, in seconds

The ratio of effective green time to cycle time is called the green time ratio, and is denoted by u (Equation 18):

$$u = g/c \tag{18}$$

The ratio of the arrival flow, Q vehicles per hour, to the saturation flow S is called the flow ratio, y, given by (Equation 19):

$$y = Q/S \qquad 19$$

The movement degree of saturation, x, is the ratio of the arrival flow to capacity (i.e. the volume to capacity ratio) and is expressed as in Equation 20:

$$x = Q/C = Qc/Sg = y/u \qquad 20$$

For a movement to have adequate capacity, it is necessary that (Equation 21):

$$C > Q \text{ or } x < 1 \qquad 21$$

Thus Equation 22:

$$Sg > Qc \text{ or } u > y \qquad 22$$

Base saturation flows

Base saturation flows at signalised intersections are normally expressed in through-car units (tcu). A through-car unit is a straight-ahead passenger car. The actual numbers of through and turning trucks and turning cars must be multiplied by adjustment factors (which are given later) to convert them to equivalent through-car units.

Base saturation flow values in through-car units per hour (tcu/h) by environment class and lane type are set out in Table 6.4.

Table 6.4: Base saturation flows in through-car units per hour by environment class and lane type

Environment class	Basic saturation flow in tcu/h		
	Lane type 1	Lane type 2	Lane type 3
A	1850	1810	1700
B	1700	1670	1570
C	1580	1550	1270

Source: Akçelik (1981) Table 5.1.

The environment classes A, B and C listed in Table 6.4 are defined as follows:

- Class A – ideal or nearly ideal conditions for the free movement of vehicles on both approach and exit sides, including good visibility, very few pedestrians, and almost no interference due to loading and unloading of goods vehicles or parking turnover (typically, but not necessarily, on a suburban residential or parkland area).
- Class B – average conditions, including adequate intersection geometry, small to moderate numbers of pedestrians, some interference by loading and unloading of goods vehicles or parking turnover and vehicles entering and leaving premises (typically, but not necessarily, in an industrial or shopping area).
- Class C – poor conditions, including large numbers of pedestrians, poor visibility, interference from standing vehicles, loading and unloading of goods vehicles, taxis and buses, and high parking turnover (typically, but not necessarily, in a central city area).

The lane types 1, 2 and 3 listed in Table 6.4 are defined as follows:

- Type 1 – through lane – a lane containing through vehicles only.
- Type 2 – turning lane – a lane that contains any type of turning traffic, such as an exclusive left-turn lane, an exclusive right-turn lane, or a shared lane from which vehicles may turn left or right or continue straight through. There should be an adequate turning radius, and negligible pedestrian interference to turning vehicles.
- Type 3 – restricted turning lane – a lane similar to a type 2 lane, but with turning vehicles subject to a small turning radius and some pedestrian interference.

Adjustment of base saturation flows

The base saturation flows in Table 6.4 must be adjusted to allow for various factors, namely lane width, gradient, and traffic composition (which takes into account vehicle type and the mix of turning traffic). The adjustment can be applied to each lane separately, or to the movement as a whole after adding the lane saturation flows if the factors are equally applicable to all lanes.

The adjustment is made as follows (Equation 23):

$$S = f_w \, f_g \, S_b \, / \, f_c \qquad 23$$

where

S	=	the estimated saturation flow in vehicles per hour
S_b	=	the relevant base saturation flow in through-car units per lane, from Table 6.4
f_w	=	lane width factor
f_g	=	gradient factor
f_c	=	traffic composition factor

The lane width factor, w, is as follows:

- 0.55 + 0.14w for lane widths between 2.4 and 3.0 m
- 1.00 for lane widths between 3.0 and 3.7 m
- 0.83 + 0.05w for lane widths between 3.7 and 4.6 m.

For a varying lane width, use the width at the narrowest point within 30 m of the stop line. The exit lane must be at least as wide as the approach lane – if it is narrower use its width.

The gradient factor is given by Equation 24:

$$f_g = 1 \pm 0.5 \text{ (per cent gradient) } 100 \qquad 24$$

with + being for a down grade, and – for an upgrade.

The traffic composition factor is given by Equation 25:

$$f_c = \Sigma\, e_i\, Q_i\, /\, Q \qquad 25$$

where

Q_i = flow in vehicles per hour per vehicle type and movement i

Q = total movement flow in vehicles per hour

e_i = through-car equivalent of vehicle traffic and movement, from Table 6.5

As indicated in Table 6.5 vehicles are classified as cars or heavy vehicles, a heavy vehicle being defined as any vehicle with more than two axles or with dual tyres on the rear axle. All other vehicles are regarded as cars.

Also, Table 6.5 classifies turns as unopposed or opposed. An opposed turning vehicle is one that has to give way to, and seek gaps in a higher priority opposing movement, whereas an unopposed turning vehicle can turn without restriction as on a green arrow signal. There are two types of unopposed turns, namely:

- Normal unopposed turns. These may be either left or right turns, where the radius of the turn is reasonably large, i.e. at least 15 m, and where there is little or no interference by pedestrians.
- Restricted unopposed turns. These may be either left or right turns, where the radius of the turn is less than 15 m, and where turning vehicles are subject to some interference by pedestrians. Turns in which vehicles are subject to interference by heavy pedestrian flows may be treated as opposed turns.

Table 6.5: Through-car equivalents (through-car units per vehicle) for different types of vehicles and movements

Vehicle	Through	Unopposed turn		Opposed turn
		Normal	Restricted	
Car	1	1	1.25	e_o
Heavy vehicles	2	2	2.5	e_{o+1}

Source: Akçelik *(1981) Table 5.2.*

For the case of long combination vehicles, the figures for heavy vehicles in Table 6.5 would need to be extended. As a broad guide, National Road Transport Commission (1998) has determined general passenger car equivalents for Austroads vehicle classes (Appendix A.5) as follows:

- vehicle classes 2 to 5: 2 passenger car equivalents
- vehicle classes 6 to 9: 3 passenger car equivalents
- vehicle classes 10 to 11: 4 passenger car equivalents
- vehicle class 12: 5 passenger car equivalents.

With opposed turns, the value of the opposed-turn equivalent e_o in Table 6.5 depends on signal timings and opposing movement characteristics. Opposed turns may include:

- filter right turns giving way to vehicles in an opposing stream
- left turns or right turns from one-way streets giving way to pedestrians
- filter left turns under ‘left turn at any time with care’ rules giving way to pedestrians or to vehicles in the opposing through and right-turn streams
- filter left turns under ‘left turn on red’ rules.

A detailed procedure for determining e_o for both exclusive lanes and shared lanes is given in Akçelik (1981). The parameter, e_o is estimated in an iterative fashion, as e_o is affected by signal phasings and timings and e_o affects the signal timings. A good first approximation is to use a value of e_o equal to 3.

If relatively precise calculations are desired in any particular case, adjustments can be made for lane underutilisation, for short lanes and for lane blockages. This involves relatively complex analysis and is best done through the use of computer programs such as SIDRA intersection. For details of these adjustments, see Akçelik (1981).

The process outlined by Akçelik (1981) includes the use of the practical degree of saturation. This value is chosen by the user and is the target degree of saturation for the critical movements. The practical degree of saturation is usually within the range of 0.8 to 0.95 and is used to find the practical cycle time.

Once the saturation flow rate for a particular traffic movement has been calculated, the capacity and degree of saturation can be determined using Equation 17 and Equation 20 given earlier. Also, by using the results for each of the separate traffic movements on any given approach, the approach capacity and its degree of saturation can be calculated.

After evaluating the signal timings, a degree of saturation should range from zero when the arrival flow rate is zero to the practical degree of saturation for the critical movements. If a cycle time is too short or there are insufficient lanes to handle all traffic (that is the saturation flows are insufficient for the demand) then the degree of saturation can be greater than the practical degree of saturation. A worked example is given in Commentary C1.2.1.

The procedures given here have been refined over years for use in the SIDRA intersection software. Further details of the procedures can be found in Akçelik and Associates (2011).

6.4.3 Measures of Intersection Performance

Capacity calculations at signalised intersections have most meaning when applied to a traffic movement or to an intersection approach. However, it is desirable to express the operational efficiency of a signalised intersection in terms of various measures of performance.

A basic measure of signalised intersection performance is the intersection degree of saturation, which is defined as the largest of the individual movement degrees of saturation. The minimum intersection degree of saturation for a given cycle time is obtained when the critical movement green times are proportional to the corresponding flow ratios. This in fact is an equal degree of saturation situation in which the degrees of saturation for all critical movements are equal. In this situation (Equation 26):

$$X = \frac{Yc}{c - L} \tag{26}$$

where

X = intersection degree of saturation

Y = intersection flow ratio

= Σy for the critical movements

c = cycle time

L = intersection lost time

= sum of the critical movement lost times

Because the intersection degree of saturation is defined as being equal to the largest movement degree of saturation, if the condition $X < 1$ is satisfied, then $x < 1$ applies for all movements.

It should be noted that equal saturation of all critical movements is not always achieved or desirable and hence intersection degree of saturation may not accurately represent the capacity or saturation of the intersection.

When X for a signal controlled intersection is less than one, the intersection is said to be under-saturated. When X = 1, the intersection is saturated, or operating at capacity. When X > 1 the intersection operation is described as oversaturated. As both queue length and delay increase rapidly as X approaches 1, it is usual in design to attempt to keep X to less than 0.9.

Other measures of intersection performance include delay, number of stops and queue length, and some equations to enable these measures to be approximately estimated for a given vehicle movement at an isolated fixed-time signal are given below.

Akçelik (1981) comments on these equations as follows. Each of these equations (which are taken from this source)) can be considered as having a uniform and an overflow component. The uniform component is based on the assumption of regular arrivals and is expressed mainly in relation to the red time. The overflow component is expressed as a function of the average overflow queue, i.e. the average number of vehicles left in the queue at the end of the green period. Overflow queues are due to oversaturation which may last only for a few signal cycles, (i.e. low to moderate degrees of saturation), or which may persist for a long period of time (i.e. degrees of saturation > 1).

The average overflow queue length for a movement at isolated fixed time signals for a degree of saturation $x > x_o$ is given by Equation 27:

$$n_0 = 0.25CT_f\left[Z + \sqrt{Z^2 + \frac{12(x - x_0)}{CT_f}}\right] \qquad 27$$

where

n_0 = average overflow queue in vehicles (where there are several lanes of vehicles, this is the total number of vehicles queued in all lanes)

C = movement capacity in vehicles per hour

T_f = the flow period, i.e. the time interval in hours during which an average arrival demand Q persists

Z = x–1

x = movement degree of saturation = arrival flow Q/capacity C

x_0 = movement degree of saturation below which the average overflow queue is approximately zero

= 0.67 + Sg/600

S = saturation flow in vehicles per second

g = effective green time per cycle in seconds that is available for the particular movement

The approximate value of total delay for a movement at isolated fixed time signals is given by Equation 28:

$$D = \frac{Qc(1-u)^2}{2(1-y)} + n_0 x \qquad 28$$

where

D	=	total delay (vehicle-hours per hour)
x	=	flow (vehicles per second)
c	=	cycle time (seconds)
u	=	green time ratio = g/c
y	=	flow ratio = Q/S
n_0	=	average overflow queue (vehicles)

The average delay per vehicle is given by Equation 29:

$$d = D/Q \qquad 29$$

where

d	=	average delay per vehicle (seconds)
D	=	total delay (vehicle-hours per hour)
Q	=	flow (vehicles per second)

The average number of complete stops per vehicle h at isolated fixed time signals is called the stop rate (Equation 30):

$$h = 0.9\left[\frac{1-u}{1-y} + \frac{n_0}{Qc}\right] \qquad 30$$

where 0.9 is a reduction factor to allow for vehicles which are delayed but which do not come to a complete stop. The number of complete stops per hour H experienced by a movement with a flow rate of Q vehicles per second is given by Equation 31:

$$H = 3600Qh \qquad 31$$

An estimate of the maximum back-of-queue length n_m at an isolated signal for an average cycle is given by Equation 32:

$$n_m = \frac{Q(c-g)}{1-y} + n_0 \qquad 32$$

Delay per vehicle at an isolated fixed time signal may be estimated using Equation 29. However, delay is a complex variable that is sensitive to a variety of factors, and when evaluating existing conditions at an intersection it is preferable to measure actual delays wherever possible.

Table 6.6 provides the LOS criteria for signalised intersections in HCM 2016 and SIDRA intersection (Akçelik & Associates 2011).

Table 6.6: LOS criteria for signalised intersections

LOS	*Control delay (s/vehicle)*
A	0–10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

Source: Exhibit 19-8 in HCM 2016 (TRB 2016).

6.4.4 Signal Calculations

Various computer models have been developed for analysing the operation of a traffic signal, and to assist in calculating signal timings, capacity, delays and queue lengths. One of the most commonly used models is SIDRA intersection. For details of this model and for information on its use in practice, see Akçelik (1987, 1993) and Akçelik and Associates (2011). Other computer-based traffic simulation models (e.g. Paramics, VISSIM) may also be used. The use of these models in network traffic operations is outlined in Section 7 and in Austroads (2016b). Appendix E in Austroads (2016b) also includes an extensive worked example of signalised intersection design utilising SIDRA intersection.

6.4.5 Summary of LOS Criteria Using Delay

Table 6.7 summarises the LOS criteria for intersections using average delay per vehicle (d). The Roads and Traffic Authority (RTA) (2002) method has been used in practice and is included in the table for reference.

Table 6.7: Summary of LOS criteria using delay

LOS	**Average delay per vehicle (d) in seconds**			
	Unsignalised intersections	**Roundabouts**	**Signalised intersections**	**All intersection types**
	HCM 2000 and 2016; SIDRA intersection	**SIDRA intersection Recommended values**	**HCM 2000 and 2016; SIDRA intersection**	**RTA (2002)**
A	d ≤ 10	d ≤ 10	d ≤10	d ≤ 14
B	10 < d ≤ 15	10 < d ≤ 20	10 < d ≤ 20	15 < d ≤ 28
C	15 < d ≤ 25	20 < d ≤ 35	20 < d ≤ 35	29 < d ≤ 42
D	25 < d ≤ 35	35 < d ≤ 50	35 < d ≤ 55	43 < d ≤ 56
E	35 < d ≤ 50	50 < d ≤ 70	55 < d ≤ 80	57 < d ≤ 70
F	50 < d	70 < d	80 < d	70 < d

Source: Adapted from TRB (2016), Akçelik and Associates (2011) and RTA (2002).

7. Modelling Guidelines

7.1 Selection of Appropriate Modelling Approach

7.1.1 Major Categories of Modelling and Jurisdictional Guidelines

Transport and traffic modelling techniques are tools that help planners and traffic engineers to examine the outcome of traffic measures. There is a variety of tools available and Austroads jurisdictions generally categorise the modelling as follows:

- Macroscopic models, also known as strategic/network models, and macro-models commonly utilise the four-step model and/or land use models to predict the volume of demand and travel patterns. At the aggregate level, macro-models have been shown to be accurate, however, their accuracy diminishes when focussing on a specific route or site. Macroscopic models are mostly used by transport planners.
- Mesoscopic models, also known as macrosimulation, are a type of simulation where vehicles are represented as a traffic stream or platoon. Vehicle speed is governed by the level of congestion on the road and the presence of traffic signals. Unlike macro-models, the mesoscopic model is able to deal with queues and intersection delays. However, it has limitations in cases where vehicle-to-vehicle interaction is a critical factor.
- Hybrid models are the hybrid of macroscopic and microscopic models and generally cover areas smaller than those in strategic models but include intersection details to more accurately reflect intersection delay. Hybrid models can use a combination of volume delay functions, shockwave and queue-back algorithms as well as intersection delay calculations when assigning demand to the network. Hybrid models can be lane based or link based (depending on the level of detail required) and can use a dynamic (where paths change throughout the model period) or a fixed stochastic assignment technique.
- Microsimulation is a type of simulation wherein individual vehicle units are traced through the network. Unlike macrosimulation, the movement of an individual vehicle is governed by how it interacts with vehicles in its proximity (i.e. car-following models, lane- change models, and gap-acceptance models). This allows microsimulation to factor in vehicle-to-vehicle interactions not possible in macrosimulation.
- Nanosimulation is the most refined level of traffic modelling, seeking to replicate the behaviour of individuals using different modes of travel. It is particularly concerned with modelling waiting times, interaction between individuals, etc. The model requires network description similar to microsimulation, however enriched with data on pedestrian spaces and corridors. They can also be used in the design of transport terminals (such as railway stations) and access to buildings and other facilities.
- Intersection models are commonly used for analysis of an isolated site, such as an intersection. Because they are based on predetermined equations, their accuracy is limited to conditions within the assumptions used in the formulation of these equations.

Examples of software packages for each of the above techniques can be found in Appendix M. However, software packages often include a combination of techniques and some packages can be considered as applying to two or more modelling levels (e.g. VISUM, CUBE and OmniTRANS), therefore the classification of software into one modelling category could be subjective.

Practitioners operate transport models at slightly different levels. For example, VicRoads (2012a) illustrates the general use of the different models at four levels, namely strategic (VicRoads 2011), mesoscopic (VicRoads 2012b), microsimulation and local area modelling. Roads and Maritime Services (2013) provides general guidelines for modelling (Part A) and specific guidelines for the modelling development and application at the following five levels (Part B): strategic, highway assignment models covering strategic and meso-highway assignment, microsimulation, corridor and single intersection modelling.

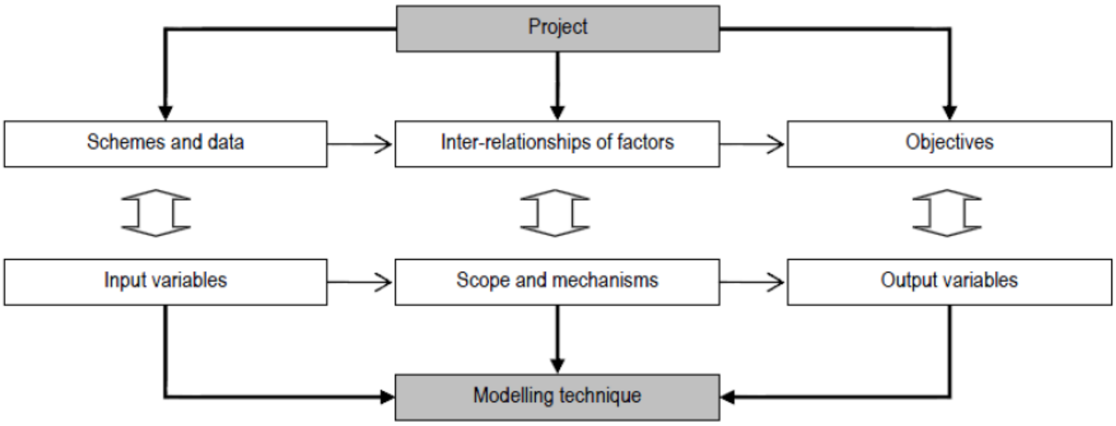
7.1.2 Selection of Modelling Techniques

Selecting a suitable modelling technique is important. Selection of modelling tools appropriate for the project needs to be based on a fit-for-purpose principle, matching project needs (traffic problems being investigated and project objectives) with the modelling method and software tool capability. The selected tool needs to be sensitive to the relevant issues of a project. Austroads (2010c), Transport and Infrastructure Council (2016) and Transport for London (2010) provide guidelines on how to select the most suitable modelling technique for individual projects.

To assess the suitability of recommended modelling techniques, the challenge is to match the context of a project to the strengths and weaknesses of each technique. The first step is to contextualise the project by identifying its key elements. These key elements should be able to cover the schemes and data, inter-relationships of factors, and objectives of the project. These elements can then become specifications of the model as follows (shown in Figure 7.1):

- input variables, which list the data that are available or going to be required for the study such as transport demand and the network
- scope and mechanism, which list the functionality of the model necessary to deal with the project which covers geographic and temporal scope, traveller response and model functionality
- output variables, which describe the preferred indicators relevant for the project; the outputs are categorised as indicators, accuracy and cost, and audience.

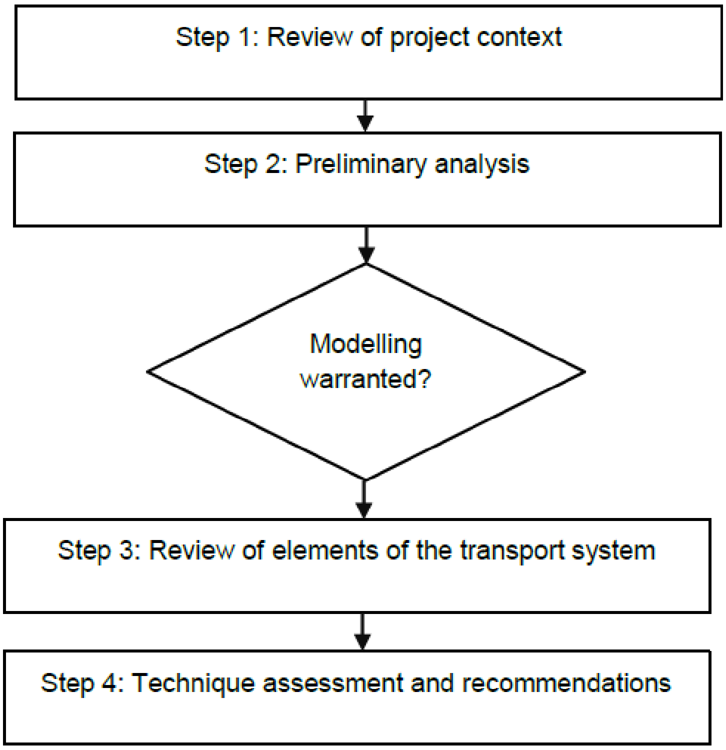
Figure 7.1: Elements of the traffic system model



Source: Austroads (2010c).

No clear science exists for selecting the most appropriate transport modelling technique. The requirement for rigorous analysis and acceptable cost needs to be balanced in the selection. In order to concentrate the resources on the projects that require modelling, it is also essential to filter out case studies that do not require modelling. Guidelines are therefore needed to facilitate the selection process. Austroads (2010c) set out four steps as shown in Figure 7.2. The first step involves careful consideration of the project objectives. The high-level preliminary analysis advocated in step 2 considers the nature of outcomes and impacts that may be expected from which the analyst can form a view on the extent to which a model can quantify these outcomes and the importance of this information in making decisions on whether and how to proceed with the project. Given an initial decision that some modelling is justified, the subsequent step 3 is to consider the aspects of the transport system that require modelling. Finally, step 4 is to consider the methods that can best be applied to represent these aspects, reflecting the nature of the project and the expected outcomes.

Figure 7.2: Steps in modelling technique selection



Source: Austroads (2010c).

7.1.3 Alternatives to Modelling

In general, modelling is an expensive and time-consuming exercise. It is therefore recommended to undertake a preliminary analysis of alternatives before applying the modelling approach. Transport and Infrastructure Council (2016) and HCM 2016 suggest some alternatives such as sketch planning techniques and qualitative comparison of the alternatives.

Sketch planning methods generate only rough indicators and an enumeration of factors and their potential impact on the schemes being examined. These methods produce only rough indicators and a list of factors and their possible impact on the schemes being examined. Generally, sketch planning is used to reduce the number of alternatives being considered for future analysis. While undertaking the sketch planning exercise, an alternative may clearly stand out or all other alternatives are eliminated. If this is the case, then no further analysis may be required (Van Hecke et al. 2008).

An example of sketch planning is the use of prescribed warrants in recommending signalisation of an intersection. If an intersection satisfies a certain number or combination of warrants the intersection will be recommended for signalisation (VicRoads 2012a). The warrants include traffic volume, pedestrian volume, accident exposure, and others. Modelling is not necessary to conclude whether an intersection should be signalised or not. In some cases, basic calculations can be sufficient, and past projects or studies with similar characteristics could also be helpful to come to a decision without modelling.

Qualitative comparison of the alternatives is also a useful approach. Environmental, social, strategic planning, and economic considerations are some of the factors that should be considered. A review of the advantages and disadvantages of possible alternatives could be enough to reach a decision. In some cases, qualitative comparison could highlight important factors outside the scope of the modelling such as social impact, strategic impacts, and complex behavioural responses.

There are three conditions under which a decision to proceed to more a demanding analysis using modelling techniques should be conducted:

- The preliminary analysis did not identify the best course of action.
- The project requires a rigorous analysis to be approved by decision makers.
- There are substantial risks associated with getting recommendations wrong.

It is recommended to only proceed to modelling when a need for modelling has been identified clearly after applying various alternatives to modelling.

7.2 Organising a Modelling Study

The key steps in undertaking a modelling study are as follows (Austroads 2006):

- identifying study objectives and project scoping (Section 7.2.1)
- selecting the right software platform for microsimulation (Section 7.2.2)
- developing a base model (Section 7.2.3)
- model calibration and validation (Section 7.2.4 and Section 7.2.5)
- auditing model output results (Section 7.2.6).

7.2.1 Study Objectives and Scoping

A modelling study can take up a lot of resources and it is advisable that the analyst (modeller), the project manager and decision maker have a clear understanding on what needs to be achieved. It is worth emphasising that each modelling technique would have different capabilities of modelling a suitable range of the network. For example, microsimulation is more suitable for site-specific or small-area analysis (e.g. 5 km × 5 km) over a relatively short time period (e.g. a few hours). It is acknowledged that the advance in the processing power and memory availability in a PC has enabled the simulation of larger networks over longer time periods. Some important questions to ask include:

- What is the project objective?
- Why is the analysis needed?
- What are the characteristics of the project being analysed?
- What questions should the analysis answer?
- What are the scenarios (alternatives) to be studied?
- Who are the recipients for the results?
- Have all stakeholders involved been consulted?
- What are the performance indices required to evaluate the scenarios?
- What resources are available?
- What is the scale of the study, both in time (temporally) and in space (geographically)? What is the peak period (or representative time period that covers project criteria) to be modelled?
- Are traffic problems identified?
- What are the design years?
- What is the extent of the model required?
- What data and information are required for the model and are these data available, affordable and practical?

- What modelling outputs are required?
- What are the potential benefits/uses of the model beyond the project?
- What previous modelling work has been undertaken in the vicinity and is it appropriate to re-use it?
- What are the risks and implications of likely errors in the modelling results?

7.2.2 Selecting a Software Platform

The use of the appropriate level of modelling and suitable package is critical in determining the success of a modelling project. Inappropriate software can lead to poor project outcomes as well as significant costs and delays. When assessing the level of modelling to be used, consideration must be given to traffic, on-road public transport, pedestrians, cyclists, road network features, traffic control systems, ITS applications and the environment.

The selection of a software platform for a particular problem depends on the nature of the problem. Appendix M provides general background information on example software packages that have been considered by Austroads jurisdictions. Each of these packages has its own strengths and weaknesses and it is beyond the scope of this report to compare them. It is, however, important to consider both non-technical and technical factors in choosing a package.

Non-technical factors include the following:

- level of expertise within a project team and the road/transport agency
- level of support from the software supplier
- training required to get a base model developed
- level of transparency of the package structure and outputs so that meaningful interpretation of model results and hence decision making are possible.

It is rare that a single package will fit all types of simulation needs. Some technical issues relevant to the choice of a package are:

- experience in applying a package for different network sizes, i.e. the scale of application
- suitability of the facilities and parameters in a package to simulate the phenomenon that an agency wishes to investigate, e.g. pedestrian movements
- sensitivity of the parameters to specific features to be analysed in the proposed scenarios
- accuracy of vehicle movement logic such as lane changing and car-following manoeuvres.

7.2.3 Base Model Development

Modelling of the existing base-case situation is essential for the vast majority of projects. The base model is required for two key reasons:

- to enable accurate verification, calibration and validation of model parameters to ensure that the model reflects current traffic conditions and provides accurate forecasts of future performance
- to provide a benchmark against which the effectiveness of the proposed designs can be compared.

Like most traffic studies, a successful modelling study requires clear objectives and full scoping of the work and schedule. Milestones need careful monitoring and deliverables should be closely reviewed. Typical key milestones and deliverables for a base model development are summarised in Austroads (2006) as shown in Table 7.1.

Table 7.1: Milestones and deliverables for modelling development (1)

Milestone	Deliverable	Contents
Study scope	• Study scope and schedule • Proposed software package • Proposed data collection plan • Proposed calibration plan • Coding quality assurance plan	Study objectives, time and space domains of study, alternative data collection plan, error-checking procedures on model coding, calibration and validation plans
Data collection	• Data collected • Data collection report	Data collection procedures, quality assurance, summary of data collection results
Model development	• 50% coded model	Software input files
Error checking	• 100% coded model	Software input files
Calibration and validation	• Calibration test report • Validation test report	Calibration results, adjusted parameters and rationale, achievement of calibration and validation targets
Auditing or alternative analysis	• Auditing report	Broad-level checking (sanity check); alternative analysis results
Final report	• Final report • Technical documentation or modelling report	Compiling previous reports; documentation of model development and calibration; software input files

1 *Focused on base model development but also applicable to general modelling development.*

Source: Adapted from Austroads (2006).

The base model should be developed for the existing traffic conditions using recently collected data. Verification of the model network coding should be undertaken to ensure that the modelled network is correctly representing the physical and operational characteristics of the real-world network (this is undertaken by comparing and refining the model network inputs to better reflect network characteristics).

7.2.4 Calibration Procedures

Calibration is the process of changing the parameter values in a model in order to achieve agreement between modelling results and observed data. It is necessary to calibrate a model if the results are to be trustworthy and used to support decisions in traffic management. The importance of calibration cannot be overemphasised. FHWA (2004) reported tests of six different software programs for predicting freeway speeds – calibration differences of 13% in the predicted freeway speeds for existing conditions increased to differences of 68% for future conditions.

The objective of calibration is to improve the model’s ability to reproduce driver behaviour and traffic performance characteristics such as travel time, delay or queue length by varying model parameter values from the default values supplied by the software supplier. Many parameters are often involved in a model. It is a good practice for a user to adopt a calibration strategy. For example, Austroads (2006) recommends key principles of microsimulation traffic modelling (MSTM) calibration as follows:

- accept those default parameters that can be used with confidence
- limit calibration to a workable set of parameters
- global parameters are those that affect the operation of a network model as a whole and are calibrated first
- local or site-specific parameters are then calibrated, e.g. those for a road link
- a smaller time step for simulation gives more accurate results, e.g. a time step of 0.2 s allows more accurate simulation of driver reaction than a time step of 1 s, but requires more time to run a simulation; the driver reaction time is also preferably an integral multiple of the time step, e.g. reaction time = 0.8 s and time step = 0.2 s or 0.4 s
- allow the model to settle down (e.g. by filling a network with vehicles) before initiating calibration; a rule of thumb is to have a warm-up period equal to twice the travel time for a vehicle to traverse from one end of a network to the furthest destination at a FFS
- undertake sufficient runs using different seeds for the random number generators; five to six runs are recommended.

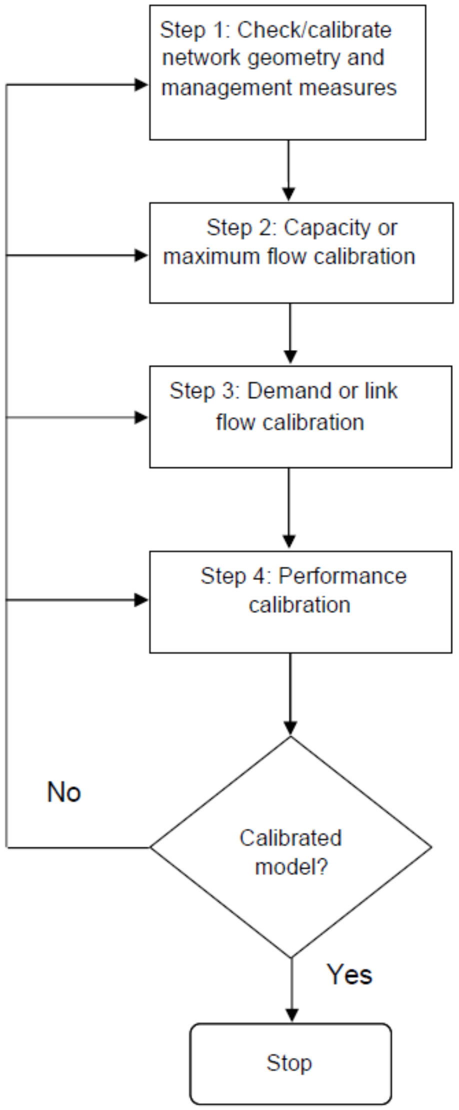
The calibration process involves varying operational parameter values within specified ranges. This is an iterative process until the modelled and observed outputs agree to an acceptable level of accuracy. Austroads (2006) reviewed various target accuracies adopted by road agencies together with overseas practices and suggested a simple principle for MSTM calibration accuracy, that is, the following model outputs should be within 5% of observed values:

- maximum flow at a stop line by vehicle types
- capacity per intersection approach
- maximum queue length per lane
- average delay per vehicle per lane including buses
- travel time for buses and the general traffic.

This calibration practice is recommended in a microsimulation study as it is adequately comprehensive without the need for 100% accuracy in all the model outputs.

Good practices for calibrating MSTMs as recommended by Austroads (2006) include network depiction, and calibrating capacity, demand and performance (Figure 7.3). Each of these core areas should be investigated and addressed and feedback loops be established as a part of the overall model calibration.

Figure 7.3: An example of model calibration process (microsimulation)



Source: Austroads (2006).

Further detailed guidelines for different types of modelling approaches are provided in Transport and Infrastructure Council (2016) and Road and Maritime Services (2013).

7.2.5 Validation Procedures

Validation can be defined as a comparison of model outputs with observed data independent from the calibration procedure. It is common to collect sufficient input data such that a portion of the input data is for calibration and the rest is for validation. The performance outputs can be travel times that are either link-specific (in seconds, minutes or hours) or flow-weighted to give a network-based index (for example, in veh-h/h). Delay or queue lengths can similarly be compared. Most network models produce both link and network-based results for analysis.

The process is far simpler than model calibration since it does not require any adjustment to parameters or inputs. The modeller simply runs the simulation and compares a set of model outputs with an equivalent set of observed site data (this must be data that have not been used during the calibration process).

Because of the randomness of both observed and simulated data, validation should be carried out on a statistical basis. Observed data are random due to the stochastic nature of traffic. Simulated data are random due to the many random numbers involved in simulating traffic operations.

A key statistical consideration in model validation is the use of confidence limits. Let the sample mean and standard deviation of the observed data be *x* and σ respectively.

Assuming normality of data, the confidence limits (CL) for a sample size of *n* and a (1-α) probability are given by Equation 33:

$$CL_{1-\alpha} = (\bar{x} - z_{1-\alpha/2}\frac{\sigma}{\sqrt{N}}, \bar{x} + z_{1-\alpha/2}\frac{\sigma}{\sqrt{N}} \qquad 33$$

where

$z_{1-\alpha/2}$ = the number of standard deviations that have 1-α/2 area on either side of the mean of a normal distribution curve. For a 95% confidence level, α = 0.05 and $z_{0.975}$ = 1.96 (the Student's t-test can also be used especially for small sample sizes).

$\bar{x}$ = sample mean

σ = standard deviation

N = sample size

In any complex model, it is likely that certain areas or functions will be better validated and calibrated than others. It is essential that confidence intervals be determined spatially (link, corridor and whole network), temporally (e.g. a.m. peak, p.m. peak) and functionally (e.g. cycle time, degree of saturation) within the model. These should be presented graphically. If certain areas of the network are less accurate than others or certain time periods or certain vehicle groups exhibit less than desirable levels of calibration or validation, they should be documented within the validation report.

Should the validation process imply that the model is not yet at a sufficient level of accuracy then the specific areas of concern should be identified and analysed. The model therefore returns to the calibration stage such that the parameters can be adjusted to address the relevant issues.

7.2.6 Model Auditing

Auditing a modelling is broadly defined as a process to verify the results from the model. It can be carried out as a peer review within a road agency or through the services of a consultant. This auditing process can be conducted by means of the following:

- general error checking by an independent analyst
- an independent reviewer who can provide a 'sanity check' on model outputs, especially on the warm-up period and whether the outputs have reached steady-state
- a comparative study of model outputs from several other models if time and budget are available, e.g. this could be benchmarking an MSTM with an analytical or macrosimulation model
- more statistical analysis
- alternative analysis using different scenarios.

7.3 Applications of Microsimulation Modelling

MSTM has been of interest to road and transport agencies to analyse complex, congested situations that normally are beyond the domain of analysis using conventional analytical or macroscopic modelling procedures. Microsimulation packages now provide a range of functions from visualisation and simulation to the emulation of a single operation in a network, which is achieved by interfacing a single system (like SCATS and STREAMS) to a microsimulation software package. The Roads and Maritime Services product SCATSIM allows SCATS to be run in a microsimulation environment. Visualisation with quality three-dimensional graphics is particularly useful for presenting solutions that address sensitive issues.

The areas that are generally regarded as appropriate for analysis using MSTM are (Austroads 2006):

- complex traffic operation schemes, e.g. bus priority, advanced signal control, incident management, different modes of toll collection
- significant conflicts between road users, e.g. pedestrians, cyclists, buses
- the effects of major roadworks on traffic movements, e.g. lane closures, one-way systems, toll plazas
- politically sensitive projects that could benefit from visualisation
- planning and design of high-value projects with potential large savings if detailed models are prepared
- emulation of the operation of a dynamic signal control system, with a simulated network driven directly by the control system, with significant savings in signal timing preparation and optimisation e.g. traffic management plans
- town-centre studies
- tram and light rail operations.

While the capabilities of current MSTM procedures and computer technology surpass those available in the past, it still holds that there is no such thing as a perfect model. It is imperative that the practitioner be aware that all models are built on assumptions and rules and, in the real world, there will always be exceptions.

The objective of microsimulation modelling is to analyse complex traffic conditions. This requires more resources and costs more than conventional modelling techniques. For model development, static and dynamic input data requirements for MSTM are well-defined. The calibration and validation procedures are reasonably standardised. It has been identified that the simulation of lane-changing phenomena is a critical issue that would affect the accuracy of model outputs. Therefore, it is essential to ensure that model outputs at a link level such as delay and travel time be carefully audited.

A typical problem in a microsimulation study is the lack of input data for establishing a base model and calibration. Sufficient resources should be allocated for this phase of work. As a guide, about 50% of the budget is for tasks leading to the coding and development of a base model, 25% for calibration and validation, and 25% for scenario analysis and documentation.

As most calibration parameters are specific to each software program, it is generally difficult to adopt a common set of operational parameter values commonly used for microsimulation packages. Additionally, the software program is unlikely to use the same set of parameters in different applications or models developed using the same package. Table 7.2 provides principles for choosing MSTM input parameters as proposed in Austroads (2006).

Table 7.2: Principles for choosing input parameters

Parameter	Recommended principles
Reaction time	Avoid large values; if excessive delay occurs in the model
Lane change/selection	Signposting or looking ahead distances should be sufficiently long for simulated drivers to be more responsive to road hazards Maximum driver awareness and aggressiveness also help the model to be more responsive Headway with next vehicle in new lane should have a minimum separation before a lane change occurs
Acceleration	A minimum value should be set
Headway	Check model headways at local sites reflect on-site headways whether a global headway is adopted in a package or not
Ratio of reaction time to simulation time step	An integer larger than 1; higher accuracy can be achieved with a reaction time that is two to three times the simulation time step

Source: Austroads (2006).

Austroads has developed a set of standardised parameter values for the consistent use of MSTM across jurisdictions. Commentary 3 provides a list of suggested values for input parameters that have been produced by Austroads project NS1229: *Microsimulation Standards* in 2007.

[see Commentary 3]

Another critical problem is unreleased trips (also called latent demand), a common phenomenon associated with microsimulation modelling in congested conditions. It describes those vehicles which are unable to enter the modelled network, typically due to localised congestion near their point of release. Because these vehicles sit outside of the modelled network for some of (and occasionally all of) the simulation period, the unrealised potential travel distance and travel time relating to their desired trips are not fully reflected in the various metrics typically produced by modern microsimulation packages (i.e. vehicle hours travelled (VHT), vehicle kilometres travelled (VKT), number of stops, fuel consumption and emissions). As a result, incorrect inferences can be drawn from the results of modelling runs (which may represent different peak periods, modelled years or options) where different levels of latent demand exist. In order to permit modelling results to be used credibly and effectively for project selection, justification, optimisation and prioritisation, it is imperative that latent demand be addressed so as to ensure ‘like with like’ comparability across different scenarios.

Commentary 4 illustrates two methods that Roads and Maritime Services has suggested to address latent demand problem.

[see Commentary 4]

7.4 Use of Modelling Outputs

All modelling approaches have their own limitations. For example, the precise microsimulation model techniques are more useful for specific assignment and operational objectives but they can tell little about the overall performance of the transport system and the opportunities available for policies and strategies to meet wider objectives. Therefore, great care is needed to avoid assignment and operational modelling being undertaken solely for policy and strategy development.

HCM 2016 noted that model outputs are subject to three main sources of uncertainties:

- uncertainty in model inputs such as variability in measured values and measurement error, uncertainty inherent in future volume forecasts and arising from the use of default values
- uncertainty in the performance measure estimate produced by a model, which in turn may rely on the outputs of another model that has its own uncertainty
- imperfect model specifications – a model may not fully account for all the factors that influence the model output.

Although uncertainty cannot be eliminated, it effects can be reduced to some extent. For example, the LOS concept helps to dampen the effects of uncertainty by presenting a range of service-measure results as being reasonably equivalent from the travellers' point of view. Where uncertainties arise about model inputs (e.g. land use forecasts) then application of a scenario-based approach is appropriate. Alternatively, sensitivity testing can be used to explore the impact of uncertainty about model parameters (VicRoads 2012a). Furthermore, sources of error and uncertainty should be recognised explicitly in the modelling process. For example, traffic forecasting can never be precise, and should not be presented as such, because it involves predicting the future.

The operation and limitations of a macrosimulation or microsimulation package should be understood in detail for modelling results to be interpreted reliably and appropriately. The modelled operation and performance of all aspects of the model must be checked carefully during and after a simulation so that accuracy and realism is satisfactory. This may require a review of the way individual aspects of driver behaviour are represented, including consideration of the suitability and robustness of default parameter values. Significant effort would be required for model auditing or validation before the model results can be appropriately reported and used.

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Appendix A Traffic Volume Surveys

This appendix provides an introduction to traffic volume counting and the estimation of associated traffic parameters. Road agencies and other private service providers carry out most traffic counting in Australia and New Zealand. It is essential to the many users that such data are presented on a consistent basis, and uniformity of methods and procedures is desirable. The appendix describes a basis for the design of traffic counting systems that provide traffic data at a consistent and specified level of accuracy and with a consistent coverage of the road network.

Table A 1: Types of traffic volume data required for different purposes

Purpose	Type of traffic data required
1. Road financing and budgeting	Total vehicle kilometres of travel (VKT) for different classes of road
2. Classification of roads and road network planning	AADT, traffic composition and trends
3. Classification of traffic for noise and environment studies	AADT, traffic composition and speed
4. Validation and adjustment of trip data collected in transportation studies	AADT, AAWT, 24 hour or peak hour across cordon or screen lines or at major intersections
5. Development and maintenance and improvement programs and economic evaluation of alternatives	AADT, VKT, DHV; traffic composition and growth trends for important corridors
6. Selection of design standards for road geometry and pavement thickness	AADT, DHV, traffic composition and growth trends for a particular road; possibly bicycle and pedestrian data
7. Interchange and intersection deign	Turning movements, traffic composition, peak-hour volume and trends; possibly bicycle and pedestrian data
8. Evaluation of LOS	DHV, hourly flow variation patterns; AADT may be adequate for low-volume rural roads
9. Supply of exposure data for determining accident rates	AADT, VKT, traffic composition
10. Implementation and appraisal of safety programs	VKT on different road sections and areas, AADT and peak-hour volumes on particular roads, movements at intersections; sometimes, traffic composition and pedestrian movements
11. Establishment of warrants for traffic control devices	AADT, peak-hour volumes; sometimes traffic composition and pedestrian movements
12. Estimating the loading on pavement and bridges(1) (in the absence of weigh-in-motion equipment)	Classified vehicle counts and axle configurations

1 *The selection of design standards for pavement thickness is now largely determined from weigh-in-motion equipment that provides a direct measure of pavement loading by vehicle types.*

Note: AADT = annual average daily traffic; AAWT = annual average weekday traffic; DHV = design hour volume; VKT = vehicle kilometres of travel. See the Austroads Glossary (Austroads 2015f) for full definitions.

A.1 Methods of Counting

Systematic counting programs have been undertaken for many years by Austroads jurisdictions.

The results of these programs are still published at regular intervals. Models for estimating design traffic volumes exist from these data sources particularly for rural areas where there are often long historical records. The science of systematic traffic counting is well advanced, but still an active area of research. Road agencies have also developed software for the correction of corrupt or missing counts recorded by permanent traffic counting stations (e.g. Commonwealth Scientific and Industrial Research Organisation (CSIRO) 2000; Section 2.3).

Computer-based traffic information systems are being developed. These systems provide a common platform to retrieve and store traffic flow and other data from freeway, signal and other emergency management systems. An example is the Transport Management Integration System of Roads and Maritime Services, NSW. The technology for measuring traffic counts consists of vehicle sensors and data loggers. When a vehicle appears at an observation station, the sensor generates a signal that is received and stored inside the logger. The data within the logger can be downloaded on site to a portable computer at regular intervals, or uploaded through telemetry to a central computer.

Traffic volume counts can be carried out manually or by automatic traffic counters.

A.1.1 Manual Traffic Counting

Manual counts are usually carried out at intersections where turning movement volumes are required or at sites where detailed classification data are needed, e.g. number of passengers or vehicle body types. Mechanical hand-counter and PC-based data loggers are available to facilitate data logging. Manual counting is much more expensive than automatic counting and is generally used for short-3period studies only. The number of persons required to count a particular site will be governed by the amount of data required and the experience of the personnel involved. For example, an experienced person can record up to 12 items, say, three traffic movements of four vehicle classes under light to moderate traffic flow.

A.1.2 Automatic Traffic Counting

Automatic counters are normally used for recording 24-hour counts and the hourly, daily or seasonal variations in traffic volume. The equipment normally consists of a data logger and an axle or vehicle sensor, or both.

Automatic axle counts

Axles are generally detected by a pneumatic rubber tube stretched across the road surface. The pulse generated in the tube when an axle crosses it closes the contact at the connected air-switch, and so the axle is registered. A typical response from a tube axle detector is shown in Figure A 1 for the case of a two-axled vehicle. Pneumatic tubes are susceptible to damage and have relatively short lifespans. They need frequent checking for splits or breakages and therefore are mainly suitable for short-term traffic studies.

Special electrical cables such as 'triboelectric' (friction sensitive) and 'piezoelectric' (pressure sensitive) cables may also be used as axle detectors (Stewart et al. 1986; Luk & Brown 1987). They work by generating a detectable electric charge as a vehicle passes over the cable. Piezoelectric cables are normally set in grooves cut in the surface, making them permanent. They are used in the CULWAY weigh-in-motion systems described in Appendix G. In-ground piezoelectric cables are durable, but more expensive than above-ground pneumatic tubes. One type of piezo-resistive material changes the resistance of the sensor under pressure and is suitable in detecting pedestrians at zero or low speeds. Appendix G.3 also describes the use of quartz piezo-cable for weigh-in-motion applications.

When tube or cable detectors are used, it is important that they are laid perpendicular to the flow of traffic to reduce the chance of multiple counts. A notable exception is in the case where detectors installed in a 'zed' pattern are used to measure lateral position also.

Automatic counting equipment that uses pneumatic tubes and other axle detectors either provides counts of 'axle pairs' from one detector, or classifies the vehicles according to axle spacings using two detectors. If it provides counts of axle pairs only, an appropriate correction factor needs to be used. This factor is usually derived from vehicle classification counts – generally taken manually at representative sites. Alternatively, a correction factor may be obtained from a direct correlation between an axle detector count recording at sites also equipped with inductive loop detectors that register the passage of a complete vehicle unit.

Figure A 1: Typical response from an axle detector when crossed by a two-axled vehicle

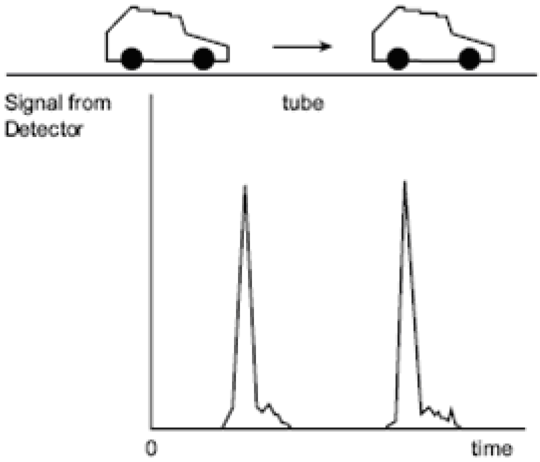
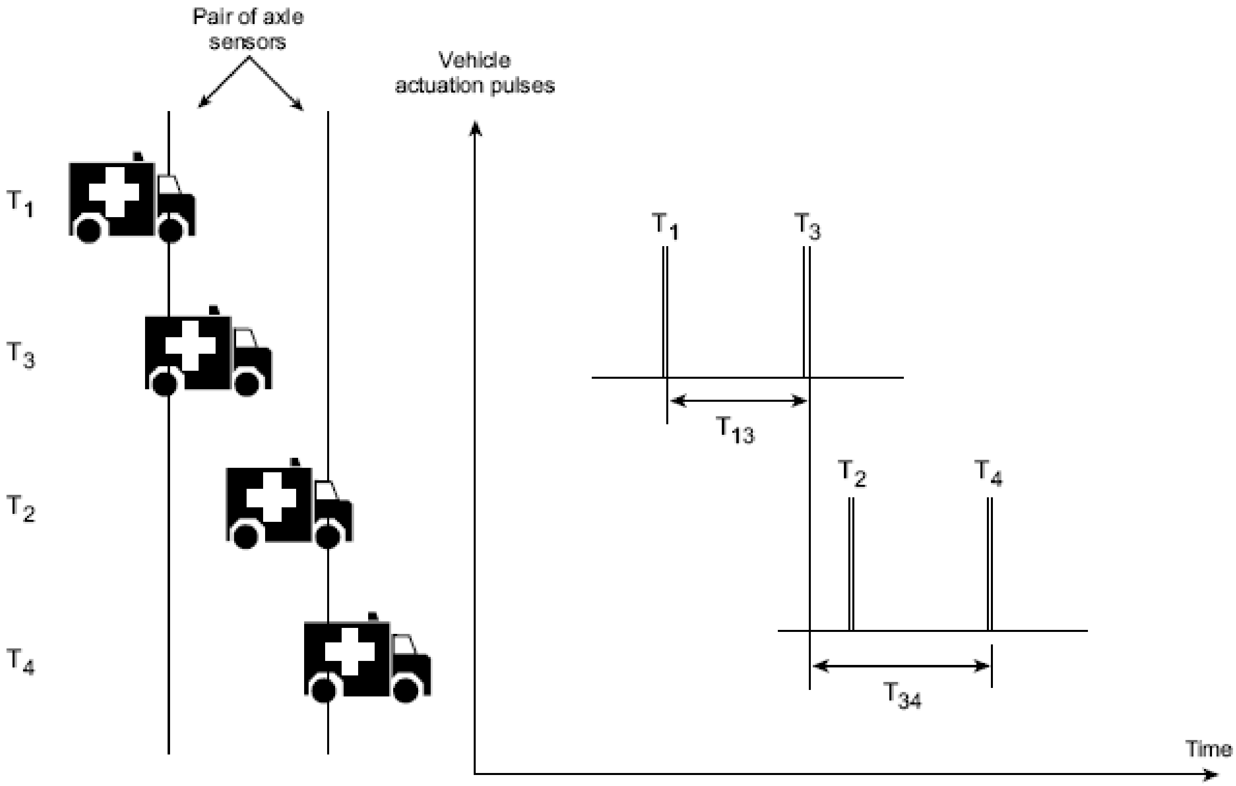


Figure A 2: Four positions of a two-axled vehicle passing over two-axle sensors



The principle of using a pair of axle sensors for vehicle classification is illustrated in Figure A 2. Two axle sensors (e.g. pneumatic tubes) are spaced at a known distance L metres apart. Four vehicle actuation pulses (T_1, T_2, T_3, T_4) are recorded when a two-axled vehicle passes over the sensors. The speed (v) and wheelbase (w) are determined using the following Equation A1 and Equation A2:

$$\text{Speed} = v = \frac{L}{T_{34}} \text{ (m/s)} \qquad \text{A1}$$

$$\text{Wheelbase } w = v\,T_{13} \qquad \text{A2}$$

where

T_{13} = time difference between T_1 and T_3

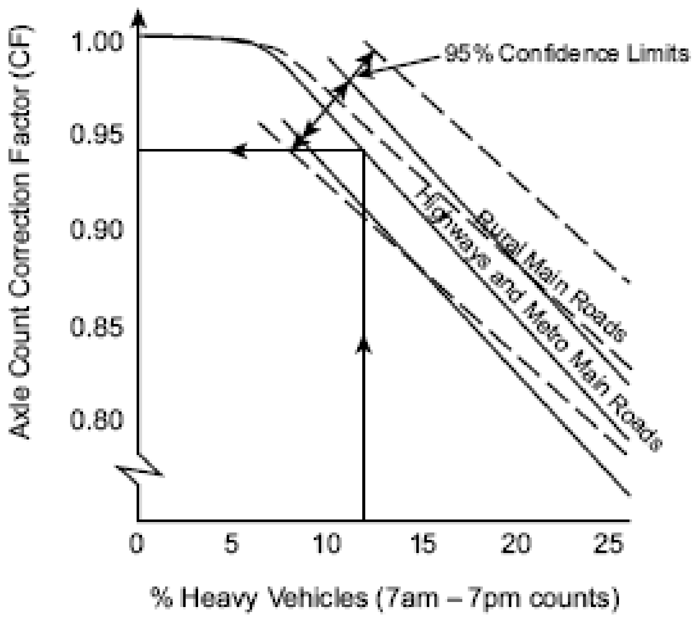
T_{34} = time difference between T_3 and T_4

The correction factor (CF) for converting axle-pairs to vehicle numbers is defined as:

CF = (total (manual or automatic) vehicle count)/('axle pair' count)

Figure A 3 shows typical correction factor graphs for highways and main roads in Victoria (similar graphs would be available in most other jurisdictions).

Figure A 3: Axle count correction factors



Automatic vehicle counts

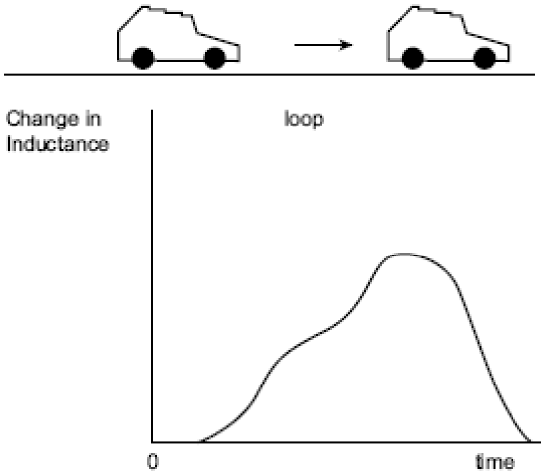
An alternative form of traffic detection is to register the passage and/or presence of a vehicle. The most widely used vehicle presence detector is the inductive loop. Other technologies include microwave or radar scanning, infrared, acoustic, magnetic and video imaging devices.

Inductive loops

The inductive loop sensor is by far the most accepted and widely used vehicle count technology by Australian road agencies. It consists of several loops of wire embedded in the pavement, or attached to the road surface, as a temporary detector. Loop detectors embedded in the road pavement are more durable under normal traffic conditions, but they are more expensive than surface-mounted rubber-tube detectors.

An alternating current is passed through the inductive loop. When a mass of metal (such as a vehicle chassis or an engine) passes through the electromagnetic field of the loop, the inductance of the loop changes. These changes are used to indicate the passage or presence of a vehicle. Figure A 4 shows the typical response of a loop detector when crossed by a passenger car. The reduction in loop inductance depends on the size and metallic content of the vehicle. For example, a passenger car generates a change of about 5% for the loops typically used in traffic counting. The inductive loop detector is used extensively for automatic traffic counting, traffic surveillance and traffic signal control. The analysis of inductive profiles is also used for vehicle classification.

Figure A 4: Typical response of a loop detector crossed by a passenger car



For accurate counting, care needs to be taken in selecting the size, shape and positioning of the loop within the traffic lanes. Over-counting can occur when loops in adjacent lanes are too close and the same vehicle is detected in both lanes. On the other hand, motorcycles or small vehicles straddling both lanes, may be missed by loops placed too far apart.

Dual loop configurations can capture three or four classes of vehicle. Some recently developed systems can capture up to eight classes by detecting the induced magnetic profiles of vehicles travelling over the loops. This enables a distinction between trucks and buses which have the same axle configuration but differ in mass. There are also available specialised loop configurations to capture bicycle movement by direction.

Two types of loop designs are commonly used by road agencies for vehicle counting on a lane of traffic flow. A loop for counting at mid-block is often a square loop of 2 m x 2 m, and a pair of these loops at a known distance (about 4 m) apart is usually used for speed measurement and vehicle classification. A second loop design is the SCATS loop for signal operation and traffic counting. The SCATS loop has a width of 2 m and an overall length of about 4.5 m and is located near a stop line of an approach. It provides reasonably accurate counts over a time period of, say, 15 minutes but the counting accuracy is less over the length of a signal cycle due to the loop location and length.

The SCATS loop does have some shortcomings as a vehicle counter. Because of the size of the loop and its location just before the stop line, at the start of the green signal it is possible for vehicles to ‘bridge’ the loop- the rear of one vehicle having not left the loop’s field of influence before the front of the following vehicle enters it. Under those circumstances the two vehicles will be counted as only one. The impact that this effect would have on the accuracy of a count would vary from site to site. It might also be more significant on approaches with short greens and at sites with short cycle lengths.

In general, inductive loop detectors are not suitable for unsealed roads. Pneumatic tubes may be used as axle sensors on unsealed roads provided the pavement is hard and smooth. Their life is short where pavement deformation or surface erosion occurs under traffic.

Microprocessor data recorders log traffic volumes at pre-determined intervals (e.g. daily, hourly, 15 minutes, 5 minutes) or alternatively they simply log individual axle event data for later downstream processing. They can store large amounts of data that can be retrieved also from a remote site using a data modem for subsequent analysis.

Other Automatic Counting Technologies

Video image detection

Video image detection (VID) systems use software to count and also classify vehicles from video recordings of vehicle movements. Advanced systems can record traffic data, detect incidents and classify vehicle by length. However, vehicle classification is generally limited to daylight hours, unless the recording site is well lit. Night-time headlight detection algorithms can measure vehicle numbers but not classify vehicles

Video imaging technology has advanced rapidly in recent years, especially as an enforcement and monitoring device for tolled roads such as the CityLink in Melbourne. There, vehicle registrations can be identified from an image of the numberplate as the vehicle passes through a gantry where the video camera is mounted. Video cameras have long been used for monitoring a freeway or other traffic facilities. Some of these cameras are dedicated for collecting data such as volume, occupancy and speed, which are suitable for automatic incident detection and as speed and red- light cameras for law enforcement.

Image processing technologies have enabled the automatic recognition of vehicle numberplates from video images. The automatic number plate recognition (ANPR) technology is often used at electronic tolling and other sites for enforcement. In NSW, it is also used in the Safe-T-Cam system to monitor heavy vehicle travel times across the state-wide road network. It can be used to obtain travel times and origin-destination information by the matching of numberplates at various monitoring points in a road network. Keilthy (2008) reported that the accuracy of modern ANPR equipment was in the range 90%–94% under ideal conditions. Its accuracy has been improved over time.

Microwave radar

Microwave radar devices use high-frequency radio waves to measure the traffic flow rate, including vehicle class. Microwave technology has the advantage of operating at high-frequency (gigahertz) wavelengths, unimpeded by climatic conditions, e.g. rain, snow, and fog, and is equally able to operate during daytime and at night. At high frequency, the technology can trace vehicle profiles, thereby enabling vehicle classified count data. Limitations of the technology include vehicle occlusion-i.e. vehicles being missed because they are partly or wholly occluded by a larger vehicle closer to the detector.

Lidar sensors

Lidar sensors use ultraviolet, visible, or near infrared light to image objects, including vehicles. Like microwave radar, the technology can be used to determine physical vehicle profiles with very high resolution, enabling both total and classification-based vehicle counts. Like microwave radar, the main limitations of lidar are vehicle occlusion and, in some cases, road profile occlusion.

Magnetometers

Magnetometers are mounted under the road pavement and detect the change in the earth's magnetic field caused by the passage of a vehicle. These are typically battery powered and connect to roadside equipment wirelessly. Magnetometer-based systems are able to count vehicles and produce occupancy estimates at the same level of accuracy as inductive loops and they are easier to install (and cause less pavement damage).

Acoustic sensors

Acoustic traffic sensors monitor vehicle noise, primarily tyre noise, and can count vehicles and measure vehicle speeds across multi-lane roads in high-traffic environments. While some researchers have found the accuracy of acoustic sensor counts and estimated vehicle speeds vary, they may be a more cost-effective alternative to conventional in-pavement sensors where road and traffic conditions limit sensor service life, or where there are significant safety concerns associated with conventional sensors. Acoustic sensors may have higher set-up costs and may be less suitable for non-permanent count applications.

Infrared

Like lidar sensors, infrared vehicle detectors operate by transmitting a high-energy beam across or down on the roadway and can detect vehicle numbers, height, width and length from the returned particles. The technology can provide measures of vehicle counts, speeds and classification.

An infrared beam acts as an axle sensor by registering the on and off times that the wheel of a vehicle crosses the path of the beam. The TIRTL equipment (CEOS n.d.) has been commercially available since 2002 after a period of research funded by Roads and Maritime Services and the federal government.

TIRTL consists of transmitter and receiver units on opposite sides of a carriageway and uses two parallel and two cross-beams below axle height to measure and classify passing vehicles. The system has a speed measurement accuracy with less than 1% error at up to 250 km/h. It also has the accuracy to classify vehicles on multi-lane highways according to the Austroads 12-bin system by measuring the number of axles, axle separation, wheel width and the front to back wheel width ratio. TIRTL has an expected product life of 20-plus years.

A.2 Sources of Counting Error

Errors in traffic counts may arise from the collection of raw count data, the estimation of a mean count from a series of observed (sample) counts, and the period over which continuous counts are taken. Counting errors would also depend on the type of equipment and method of counting (the pneumatic method would yield an error type different from that of the inductive loop method). It is always essential to follow the equipment manufacturer's instructions to minimise the counting error.

A fundamental observation about all traffic count surveys is that errors and biases are inevitable and must be allowed for. Neither manual nor automatic counts are free of possible errors. Although the magnitudes of specific errors at any one site are difficult to ascertain, some indications of likely errors are available if the site can be compared to other surrounding sites. The following rules are known:

- Completely accurate counts do not exist due to human errors, mechanical failures, environmental factors such as rain and magnetic fields, vehicles straddling two lanes, interference from opposing traffic, misinterpreted multi-axled vehicles (e.g. vehicles following others too closely or changing speed), vehicles turning across axle detectors, and other factors which affect the reliability of raw traffic counts.
- Manual counts tend to underestimate traffic volumes, due to missed observations. These errors of omission are likely to occur more frequently on heavily-trafficked roads or when inexperienced personnel are used. A rule-of-thumb is that undercounting of about 5% or higher may be expected from manual traffic count surveys.
- Automatic counters using single-axle detectors may be expected to overestimate traffic volumes, unless an axle count correction is applied to the raw data. The magnitude of this error will depend on the proportion of multi-axled or heavy vehicles in the traffic (Figure A 3).
- If the axle sensors of a classifying counter are placed closer together than the specification, vehicle speeds are over-stated and vehicles could be wrongly classified.
- Time-clock errors in automatic counters may result in apparent 'time shifts' of peak demand periods. Counts should be taken over intervals of at least 24 hours to permit corrections to be applied. Long-term records must also be corrected for daylight saving time.

The counting personnel should be well prepared in terms of counting equipment, suitable seats, appropriate clothing including contingency provisions for adverse weather, amenities (food, toilet), and carry authority certificates. Conspicuous signage is not recommended as it may affect count results.

A common and useful assumption is that traffic count estimates have a normal probability distribution about the true traffic volume. The assumption of statistical normality means that the methods of statistical inference can be applied to generate confidence intervals for traffic volume estimates i.e. a likely range of values of the true traffic flow.

A.3 Estimation and Accuracy Requirements

Frequently used traffic characteristics include AADT, DHV, VKT and peak-hour volume. DHV and VKT can be derived from AADT and therefore the measurement and estimation of AADT is important. Global estimates of VKT over the total road network are also estimated by other methods, e.g. the Survey of Motor Vehicle Usage of the Australian Bureau of Statistics (ABS from 1963 onward). In NSW, vehicle odometer readings are recorded at annual inspections and the information if transferred could provide useful VKT data in that jurisdiction.

The use of AADT assumes that weekend traffic remains fairly constant over time, although less so than weekday traffic. However, a much larger component of weekend traffic is discretionary and factors such as rising fuel prices might well impact on the underlying assumptions. The AAWT might well become a better indicator for planning purposes.

The accuracy of an AADT estimate depends mainly on the period of time during which traffic was counted. It also depends on the accuracy of the seasonal adjustment factor used to convert the average daily traffic (ADT) to AADT as described in the following section.

Similarly, the accuracy of the estimation of VKT depends primarily on the number of road links on which AADT is measured and the accuracy of the estimates of AADT. If every link with relatively uniform traffic cannot be sampled, it also depends on the dispersion or ‘scatter’ of the estimates. Accuracy can be improved by grouping together road segments with similar traffic characteristics. These groups may be geographic regions, functional categories, or volume strata, depending on the data sought. For example, seasonal variations have the greatest impact on AADT estimates and regional groupings are usually preferable. On the other hand, volume stratification is used to improve the accuracy in estimating VKT.

Thus, there is a clear distinction between the estimation of AADT and VKT:

- AADT is estimated by making short-term counts, usually on a rotational basis, at selected sites, calculating ADT and applying an adjustment factor. This factor is derived from a seasonal (12 month) count on the road in question, or on a road exhibiting similar seasonal traffic variations.
- VKT on a network is estimated by measuring or estimating AADT on selected links, and making an estimate of the total network travel on the basis of travel on each link. The accuracy of the result will depend on the number of links selected and the dispersion of AADT on the network. It can be improved by adopting statistical sampling procedures.

Accuracy requirements for both AADT and VKT are set out in Table A 2 and Table A 3.

Table A 2: Traffic characteristics' accuracy requirements – individual sites or road sections(1) (2)

Traffic characteristic	Maximum error requirements
AADT < 100	50%
101–300	35%
301–1100	25%
> 1100	15%
Trend in AADT	10%
Traffic composition	20%

1 *Accuracies for individual sites at the 68% confidence limits.*

2 *Traffic composition errors are expressed as percentage errors of the vehicle class proportions.*

Source: Austroads (2013).

Table A 3: Traffic characteristics' accuracy requirements – groups of roads(1)

Traffic characteristic	Australia	State	City(3)	Rural area(3)	AADT range(2)
VKT – Annual total	3%	5–7%	10%	10%	25%
VKT – Annual change	1%	3%	5%	5%	10%
VKT – Annual composition	3%	5%	10%	10%	15%

1 *Accuracies for groups of roads are expressed as errors at the 95% confidence limits.*

2 *Recommended AADT ranges are given in Table A 5.*

3 *If these accuracies are achieved, those for jurisdictions and Australia will also be obtained.*

Source: Austroads (2013).

A.4 Measurement and Estimation of AADT

Some r jurisdictions carry out comprehensive area traffic volume surveys in which traffic volumes are usually expressed in terms of AADT. These are measured on all important roads in a state/territory or region comprising several local government areas. By definition AADT is measured at sites where counts can be taken continuously for a full year. A sample count method, however, may be used to estimate AADT based on the assumption that the seasonal variation in daily volumes remains stable over a long period of time and over a long length of road.

Other approaches that are being used by Austroads member agencies include those reported in Transfund New Zealand (2001) and by the US Federal Highway Administration (FHWA 2016).

A.4.1 Daily and Seasonal Variations in Traffic Volume

Traffic volumes vary between hours of the day, days of the week, and between weeks or months of the year.

The variations are cyclical and repetitive year after year under normal conditions. The traffic flows at a site can be regarded as a time series, with variations occurring as a result of the combination of random, cyclic and trend effects. Some of the variations are due to:

- Daily variations – stem from hour-by-hour changes in levels of traffic demand. Figure A 5 shows typical examples for urban and rural arterial roads. Distinct peaks may be observed, corresponding to peak levels of travel-related activities by people. In rural areas, a single peak in mid-afternoon is common, while urban roads tend to have two peaks (morning and evening). Urban roads also tend to show directional differences in flows.

- Weekly variations – differences especially between weekday and weekend may be observed as shown in Figure A 6. Typical weekday flows on arterial roads in urban areas are higher than weekend flows, while the rural variations are less well-defined and may reflect the influences of recreational traffic.
- Seasonal variations – tend to be fairly consistent for a given type of route and location. This observation forms the basis of the methods for estimating AADT and DHV from weekday counts (Appendix A.4.3). Figure A 7 indicates typical patterns of variations over the months of the year. Urban roads generally show small variations, whereas rural roads may show significant changes, usually as a function of the level of recreational and vacation traffic.
- Trend effects – arise from the changes in the general levels of traffic activity at a site over an extended period, usually as a reflection of changes in land use, population and economic activity in a region.

Figure A 5: Typical hour-by-hour variations in two-way flow on arterial roads

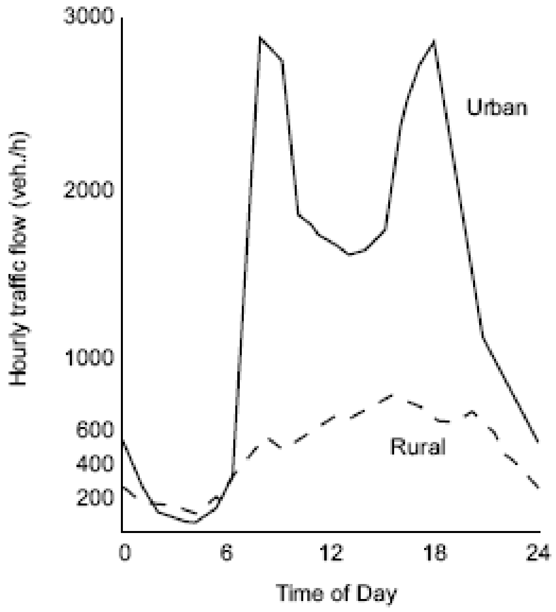


Figure A 6: Typical variations in traffic flow over days of the week on arterial roads

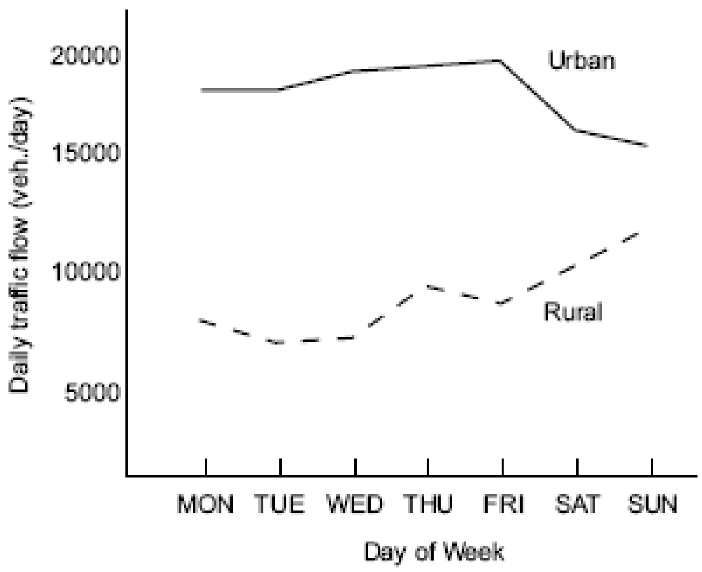
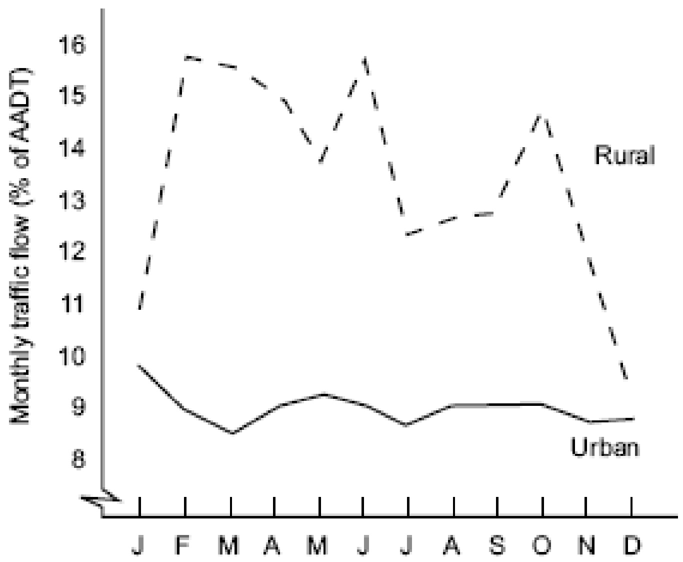


Figure A 7: Typical variations in traffic flow on arterial roads over months of the year



A.4.2 Counting Stations

Roads having similar traffic patterns may be grouped together, and different types of counting stations used to monitor traffic volumes in each group. These groups can remain substantially intact for several years, even though the AADT values on different road sections within the pattern group may vary over wide limits.

Counting stations are categorised as pattern stations or short-term stations according to whether counting proceeds long enough to establish seasonal patterns or not.

Pattern stations

Pattern stations may be either:

- Permanent stations, which are monitored continuously
- Seasonal stations, which may be continuously counted or frequently sampled, but usually in a particular locality for 12 months.

At permanent or continuously counted seasonal stations, the AADT may be calculated directly. This represents a measured value. Where less than a full year's data is available, techniques for synthesising missing data are used (CSIRO 2000). However, at Seasonal Stations where counting is not continuous, AADT is estimated by using appropriate adjustment factors. The adjustment factor for a particular Seasonal Station is obtained by comparing its output with that from a Permanent Station which exhibits a similar pattern.

It should be noted that seasonal volume variations are much more closely related to climatic and geographic characteristics than to AADT. Consequently, the application of a seasonal variation factor derived at a Permanent Station in the same AADT stratum, but in a different geographic area, is less likely to yield an accurate result than if the Permanent Station was similarly located geographically.

Short-term (coverage count) stations

At Short-Term Stations, traffic volumes are measured for only a brief period (usually less than a week), once during a survey period. As at a Seasonal Station, AADT may be estimated by multiplying the ADT obtained at the Short-Term Station by a Seasonal Adjustment Factor derived at a suitable Permanent Station (Appendix A.4.3).

The adequacy of this approach depends upon the assumption that each Short-term Station is well represented by the Permanent Station chosen. The selection of Pattern Station locations is therefore very important (Transfund New Zealand 2001). If the choice is strictly random, it is unlikely that adequate coverage will be obtained. There is also a danger that permanent sites, placed at positions of known congestion or other vital interest, may not be indicative of the traffic pattern in the vicinity.

To increase the confidence in the pattern information gained, it is recommended that the collection of data at all Short-term Stations in a pattern group be made at the same time as the related Seasonal Station is being sampled. This should be possible with careful deployment of available short-term count equipment and staff, provided the number of stations in each pattern group is not excessive. An example is given in Appendix A.4.3 to illustrate the error in estimating AADT using short-term counts.

Location and density of counting stations

The accuracy of an AADT estimate depends partly on the reliability of grouping Short-term Stations with Pattern Stations, and the method of grouping is important. Areas with similar economy, culture and development tend to show similar traffic patterns. The most effective grouping is therefore generally by geographical region as already mentioned.

Pattern Stations are best located by randomly selecting the number of sites on roads from each of a number of AADT strata, within each geographical region. As a general rule, the density of stations in each stratum should be proportional to the product of the total length of road and the square root of the mean AADT in that stratum. This will provide a weighting factor in favour of low-volume roads, which tend to exhibit more daily variation in traffic volumes.

Usually the number of Short-term Stations is determined by the extent of the road system and the availability of funds. It is usual to count on each leg of all major intersections, and at identifiable sites remote from major intersections. The variations in AADT at these sites are expected to differ from those measured at major intersections.

The number of Pattern Stations, and the ratio of Pattern Stations to Short-term Stations, cannot be rationally assessed in advance, because the assessment will depend on the regional grouping and the reliability of matched patterns. The achievement of consistent and specified accuracy levels in AADT estimation is essentially an iterative process. Table A 4 can be a guide on the densities of pattern stations).

Table A 4: Densities of pattern stations

Type of road	Counting station density (km/station)		
	Pattern stations		Short-term stations
	Permanent[1]	Seasonal	
Rural			
Freeways, arterials	100–200	200–300	Up to 25 (usually 13–17)
Local Roads	5000–9000	1000–2000	Short term stations may be characterised by one Pattern Station
Urban			
Freeways, arterials	20–50	30–50	
Collectors, distributors	100–150	50–100	
Local roads	> 1000	4000–6000	

1 *Lower figures (or higher densities) applicable if relatively few Seasonal Stations used.*

Source: Austroads (2013)

A.4.3 Estimation of AADT

To measure the AADT on a particular road segment, it is necessary to locate a counting station on that segment for a year, then divide the total counted traffic by the number of days in the year.

If traffic were counted on a random day in the year, the result would only approximate the AADT. The accuracy of the approximation would depend on the weekly and seasonal pattern for the road segment, the day of the year the count was made, and on the ADT counted because daily variations tend to decrease with increasing traffic volumes.

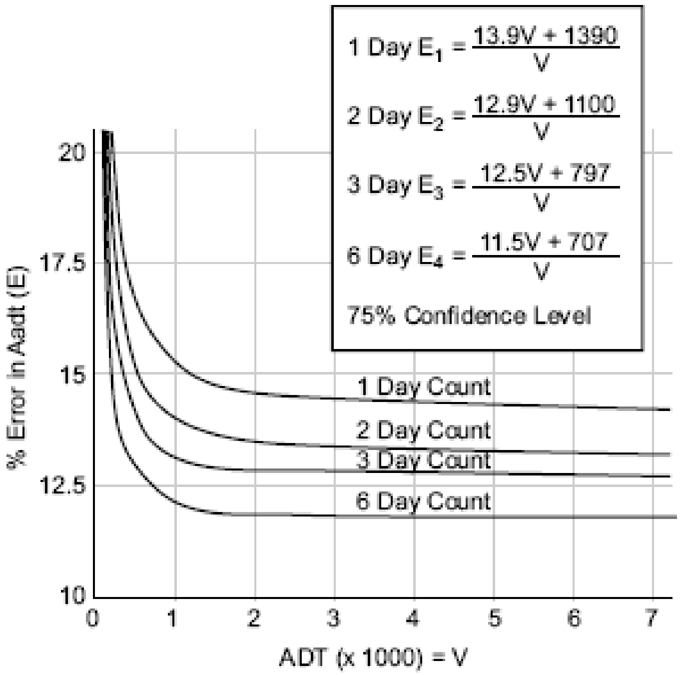
The longer the period during which traffic is counted, the closer the ADT obtained will approximate the AADT. For example, the ADT derived from a seven-day count will tend to be a more accurate estimate of AADT than a one-day count. Similarly, the ADT derived from several short-term counts throughout the year will more closely approximate the AADT, especially if the particular road segment exhibits a high seasonal variation. A typical relationship between count duration, ADT and estimate accuracy is illustrated in Figure 4.8 (Austroads 2013; Michael 1976). Where a broad counting program has been established and seasonal patterns identified, the AADT at a particular location may be estimated by multiplying a sample count (say, two-to-six days duration) by the seasonal adjustment factor derived from a Pattern Station representative of the required location as in Equation A3:

$$X = (AADT)_j = ADT_{i,j} \times (SAF)_{i,k} \qquad \text{A3}$$

where

$(AADT)_j$	=	the AADT at the required location j
$(ADT)_{i,j}$	=	the sample count in the season (month, week, etc.) i at the location j
$(SAF)_{i,k}$	=	the seasonal adjustment factor for the season (month, week, etc.) i at a Pattern Station, k, representative of the required location j.

Figure A 8: Expected percentage error for AADT from ADT and count durations (n-day counts) for a 75% confidence level



An example below illustrates the estimation of AADT and the error due to count duration using Figure A 8.

Let the ADT obtained from 6-day counts at a Short-Term Station X be 6335 vehicles. Let the ADT obtained from 6-day counts at a Permanent Station Y be 13 221 vehicles. Let the AADT obtained from one-year counts at a Permanent Station Y be 14 176 vehicles.

Then the seasonal adjustment factor = AADT/ADT for Permanent Station Y:

= 14 716/13 221 = 1.113

AADT for Short-Term Station X = ADT x seasonal adjustment factor:

= 6335 x 1.113 = 7051 vehicles

Error due to the count duration using 6-day counts (Figure A 8:) is given by:

E_6 = (11.5 x 6335 + 707)/6335

= 11.6% at 75% confidence level

Absolute error from count duration:

= ± 11.6% x 7051

= ± 818 vehicles.

The accuracy of an AADT estimate therefore depends on the accuracies of both the short-term count and the seasonal adjustment factor. The latter depends on the accuracy with which the Pattern Station reflects the fluctuations at the Short-Term Stations. The estimation of error in the seasonal adjustment factor and its combination with the error due to count duration are described in Appendix J.

Transfund New Zealand (2001) provides the factors to convert short-term counts to estimates of AADT. These factors are categorised by road types, day-of-week, time periods of day (part-days) and vehicle types. The day and part-day conversion factors are shown in Table A 5 for Auckland and non-Auckland roads in New Zealand. These factors could be validated by using local data from other places, note that error for AADT estimates using part-day factors can be greater than 30%.

Some examples of using Table A 5 are as follows:

- If the short-term day count on a Wednesday on a rural urban fringe site is 10 000 vehicles per 24 hours, then the estimated AADT is 1.10 x 10 000 = 11 000 vehicles.
- If the short-term part-day (7 am – 9 am) count on a Wednesday at the same rural urban fringe site is 1300 vehicles per 2 hours, then the estimated AADT is 8.61 x 1300 = 11 193 vehicles.

Table A 5: Some conversions factors from short-term counts to AADT estimates

Road types	Day factors						
	Mon	Tues	Wed	Thu	Fri	Sat	Sun
Urban arterial 1 (Auckland)	0.98	0.95	0.93	0.92	0.88	1.17	1.37
Urban arterial 1 (non-Auckland)	1.01	0.98	0.95	0.94	0.89	1.09	1.24
Urban arterial 2 (Auckland)	0.99	0.96	0.95	0.93	0.89	1.13	1.31
Urban arterial 2 (non-Auckland)	1.00	0.97	0.94	0.93	0.88	1.11	1.30
Urban CBD (Auckland)	1.02	1.00	0.97	0.94	0.86	1.07	1.34
Urban industrial (Auckland)	0.85	0.85	0.84	0.84	0.84	1.84	2.66
Rural urban fringe	1.14	1.14	1.10	1.07	1.07	0.94	0.86
Rural strategic 1	1.05	1.01	0.99	0.97	0.97	1.09	1.10
Rural strategic 2	1.11	1.14	1.10	1.05	1.05	1.03	0.91
Rural recreation summer	1.07	1.17	1.13	1.05	1.05	1.11	0.88
Rural recreation winter	1.15	1.25	1.21	1.12	1.12	1.03	0.82

Road types	Part-day factors for typical Monday–Thursday			
	7–9am	9am–12pm	1–4pm	4–6pm
Urban arterial (Auckland)	5.63	5.66	5.10	5.78
Urban arterial (non-Auckland)	7.31	5.17	4.60	6.00
Urban CBD (Auckland)	7.86	5.77	5.22	6.36
Urban industrial (Auckland)	5.57	4.13	3.72	5.97
Rural urban fringe	8.61	5.59	5.49	6.64
Rural strategic 1	8.40	4.74	4.41	5.99
Rural strategic 2	10.20	5.17	4.89	6.88
Rural recreation summer	16.90	4.84	4.08	7.77
Rural recreation winter	13.80	5.24	4.76	7.91

Source: Transfund New Zealand (2001).

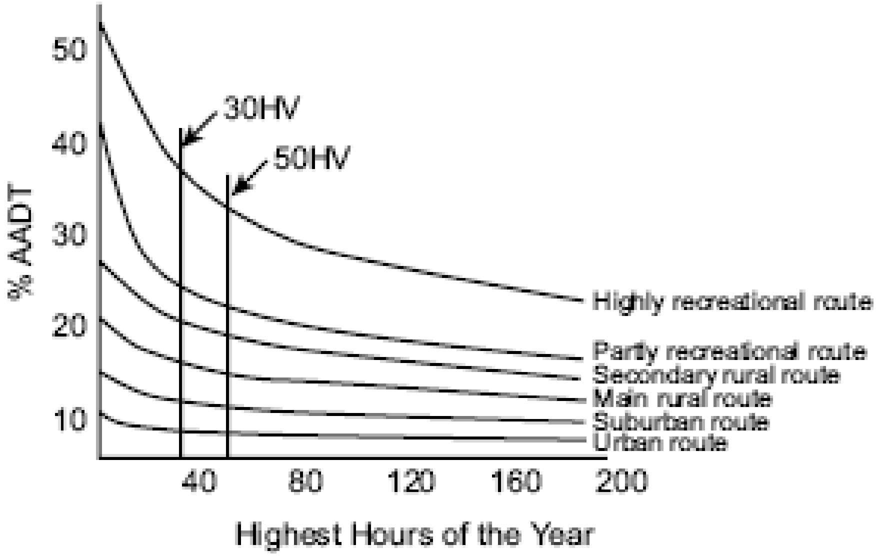
A.4.4 Estimation of Design Hourly Volume from AADT

Choice of design hourly volume (DHV)

When designing a road, a balance must be achieved between the construction cost and the LOS. The objective of the designer is generally to achieve the desired LOS at acceptable costs. Traffic demand usually varies widely in a day and throughout a year, and it would be uneconomic to design a road for the maximum hourly volume that could be expected. Instead, a lower volume is chosen which will be exceeded for a particular number of hours during the year. During this time congestion may be noticeable, and a lower LOS experienced by motorists.

The 30th highest hourly volume (denoted as 30 HV) is often used in designing rural roads, taking into consideration the traffic growth over a design period. Figure A 9 illustrates the relationship between hourly volume and annual traffic for various types of roads. If a DHV higher than the 30 HV is chosen, a considerable increase in road construction costs will be incurred for a relatively small decrease in the number of congested hours of operation. On the other hand, choosing a DHV less than a 50 HV saves relatively little in expenditure per extra congested hour. Nevertheless, the 30 HV may be inappropriately high for predominantly recreational routes, and an 80 HV or 120 HV may be chosen. In these cases the choice of DHV often needs special economic consideration.

Figure A 9: Typical relationships between hourly volumes and AADT



Determination of design hourly volume

If hourly volumes are available for the whole year, the *n* HV can be found directly by sorting the data. Similarly, if intermittent counting has been adopted (say, one week in four), a value can be readily obtained, although with less accuracy.

To determine the *n* HV for a Short-Term (Coverage) Count Station, where little or no hourly data is available for the whole of a year, it is necessary to obtain a relationship between *n* HV and AADT. The simplest method is to select a representative Pattern Station (Permanent or Seasonal) where full hourly data is available, and use the ratio of *n* HV/AADT from that location for the Short-Term Station. In practice, a number of Short-term Stations are often allocated to a particular representative Pattern (Permanent or Seasonal) Station. Where a number of Pattern Stations are available in the one pattern group, regression equations may be used to get *n* HV from AADT.

On urban roads which are subject to pronounced peaks, it may not be appropriate to establish capacity analysis or design on a full peak-hour flow. This is because higher flows for shorter periods (e.g. 15 minutes) may result in unacceptable congestion for even a short period. The peak 15-minute flow rate, converted to an equivalent hourly rate, may be used in such situations. Alternatively, a peak-hour factor (PHF) can be specified for the site and the full-hour flow is divided by PHF to give a higher design volume to cater for possible heavier congestion. Further details of the method of estimating DHV can be found in Vaughan (1968).

A.4.5 Estimation of Vehicle Kilometres of Travel (VKT)

The total daily travel (the daily VKT) on all segments in a road network is the sum of the product of AADT on each segment and the segment length. The yearly VKT or total yearly travel is therefore the total daily travel multiplied by the number of days in that year (365 or 366 days), and care must be exercised in comparing yearly VKT. Measurements of VKT and trends in VKT are required for Australia or New Zealand as a whole, for states and territories, or for regions within states, e.g. capital cities, major provincial cities and the remaining rural areas. Within these regions, VKT may be required by the class of road, individual road or road section.

VKT may be estimated by determining the mean AADT of a sample of road segments representing the system under consideration, and then multiplying this by the total length of roads in the system. The accuracy of this estimate depends on the sample size and the dispersion of AADT in the network. If the distribution is skewed, the median AADT is more appropriate for estimating VKT.

Stratification

The AADT dispersion may be reduced by restricting the group to road segments carrying a limited range of AADT. These groups, or 'strata', will each include a relatively large number of segments which can be regarded as a 'population' for statistical sampling. The relative proportions of roads within various traffic volume strata will differ in various regions throughout a state/territory or an area. Stratification between urban and rural areas is of prime importance, and is also desirable within urban and rural regions because of differences in the economy and development.

Most road segments carry relatively low volumes. Strata limits should be adjusted to provide similar amounts of travel in each stratum, and the strata limits therefore show a bias towards low- volume roads. Table A 6 provides some stratification for urban and rural roads.

Stratification by AADT volume requires a preliminary estimate of the AADT on each segment so that it can be allocated to a group. Existing data may be used for this purpose. Stratification can also be by traffic patterns, which are dependent on economic-geographical factors.

Table A 6: Suggested volume stratification of road segments

Stratum	Suggested range for urban roads (AADT)	Suggested range for rural roads (AADT)
1	0–300	0–100
2	301–1 100	101–300
3	1 101–2 000	301–700
4	2 001–4 000	701–1 100
5	4 001–7 000	1 101–2 000
6	7 001–16 000	2 001–4 000
7	16 001–20 000	4 001–7 000
8	20 001–35 000	7 000 plus
9	35 001–50 000	
10	50 000 plus	

Number of AADT observations

The recommended method of estimating the number of AADT observations required to calculate VKT_{ij} in any region i and in a stratum j is as follows:

Calculate the following estimates from the most recent available data (Equation A4):

$VKTi$ = amount of travel in region i

$$= \sum_{j=1}^{n_i} VKT_{ij} \qquad \text{A4}$$

where

SD_{ij} = standard deviation of the AADT observations in region i and stratum j

L_{ij} = total length of road in region i and stratum j

Ni = the number of strata in region i

Calculate the required number of AADT observations in region i and stratum j using the following Equation A5 (Austroads 2013):

$$N_{ij} = \frac{L_{ij}SD_{ij}}{(0.05VKT_i)^2}\sum_{j=1}^{n_i} L_{ij}SD_{ij} \qquad \text{A5}$$

where

N_{ij} = the required number of AADT observations in region i and stratum j

An example calculation to illustrate the procedure is shown in Table A 7 with eight strata (n_i = 8). The site is on a rural road. The estimates of VKT_{ij}, L_{ij} and SD_{ij} are from existing inventory data in region i.

Table A 7: Calculation of the number of VKT stations

AADT stratum *j* = 1 to 8	Inventory data in region *i*			L*ij* x SD *ijl* (veh-km)	No. of stations N *ij*
	VKT ij (x10³ veh-km)	L *ij* (km)	SD *ij* (veh)_		
0–100	2 586	94 821	29	2 749 809	14.07 (14)
101–300	2 750	15 275	60	916 500	4.7 (5)
301–700	3 277	6 931	111	769 341	3.94 (4)
701–1 100	2 409	2 699	110	296 890	1.52 (2)
1 101–2 000	3 496	2 310	279	644 490	3.30 (4)
2 001–4 000	4 013	1 428	543	775 404	3.97 (4)
4 001–7 000	2 574	496	848	420 608	2.15 (3)
7 000 plus	2 703	275	2 459	676 225	3.46 (4)
Sum	23 808	–	–	7 249 267	40

The total VKT_i for this region *i* using eight strata

$$= \sum_{j=1}^{8} VKT_{ij} \quad = 23\,808 \text{ x } 103 \text{ veh-km}$$

The summation

$$= \sum_{j=1}^{8} L_{ij}SD_{ij} \quad = 7\,249\,267 \text{ veh-km}$$

The number of counting stations for each stratum (N_{ij}) is calculated using for example, L_{i1} = 94 821 km and SD_{i1} = 29 veh for *j*=1 and

$$Nij = \frac{94{,}821 \text{ x } 29}{(0.05 \times 23{,}808 \text{ x } 10^3)^2} \times 7{,}249{,}267 \quad = 7\,249\,267 \text{ veh-km.}$$

Location and density of AADT observations

The methods of estimating AADT are as discussed in Appendix A.4.3. The two main sources of error in estimating AADT, instrument error and seasonal factors, do not have a significant effect on the accuracy of VKT for a region provided an adequate sample of AADT observations is obtained.

Some member jurisdictions have counting systems designed to estimate AADT. Where these AADT estimates are sufficient in number, and meet the stipulated accuracy levels of Table A 2, it may be possible to use these as AADT observations for VKT estimation. This can be checked by comparing the mean and standard deviation of the AADT estimates for all roads in the stratum, making use of existing traffic volume data in a member agency. If there is a significant difference, it will be necessary to include additional ADT observation points.

Testing of this has shown that problems may be experienced in the lower-volume categories, particularly in urban areas. Some extra counting may have to be arranged in this area. Short-term counts on local government roads adjusted appropriately could be used, and estimation techniques based on housing density or other parameters might also be useful.

There is a need for further research in this area. Where no traffic counting is currently being undertaken to the required degree of accuracy, observation points should be randomly selected. The recommended procedure would be to divide the length of road in each stratum into equal intervals, the number of such intervals equalling the number of AADT observations required. The observation points could then be located at the same relative position within each interval. Software tools such as the geographical information systems (GIS) are available to facilitate this task.

Trends in VKT

Trends relate to extended periods, and are accordingly less vulnerable to inaccuracies than the values of the characteristics to which they refer. Consequently, the number of measurements necessary to establish values of VKT, with the required accuracy, will also be adequate to provide the accuracy for trends, as prescribed in Table A 2.

A.5 Vehicle Classification

Vehicle classification data are important for all transport engineering applications. Comprehensive classification data are now possible using new technologies. Vehicle classification surveys are concerned with the distribution of traffic data into different types of vehicles. Traffic engineers are interested in the composition of a traffic stream, for example, in terms of flows of cars, heavy vehicles, motorcycles and bicycles for the planning, operation and assessment of a traffic system.

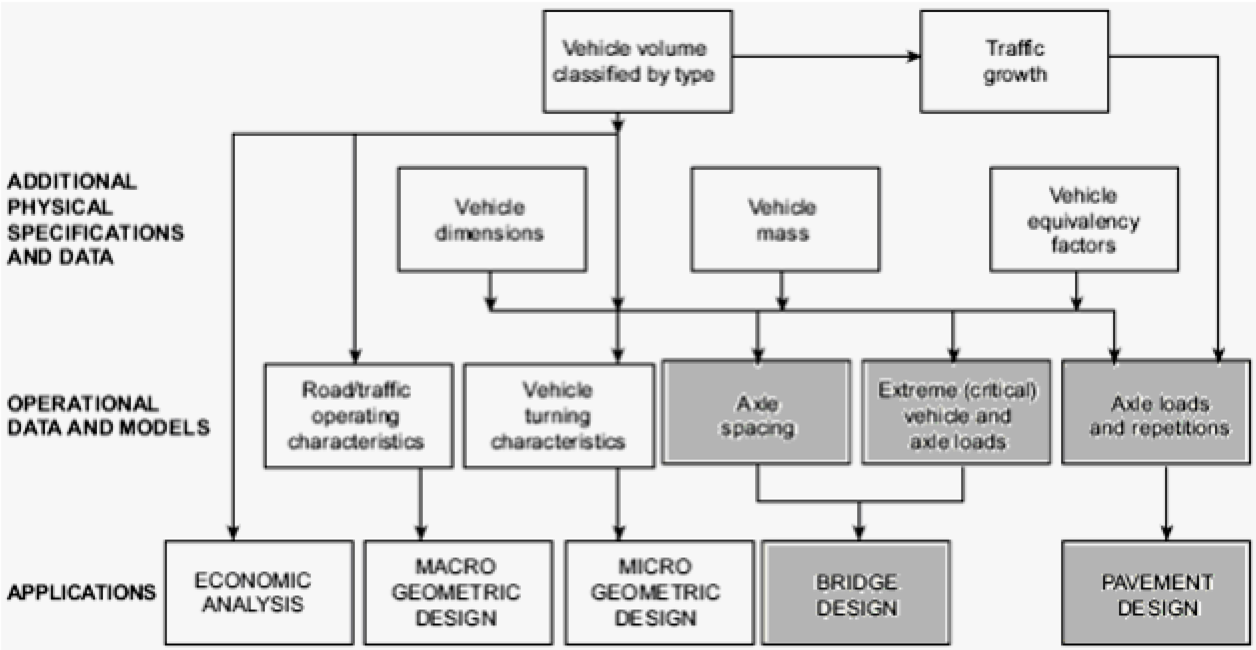
A.5.1 Needs for Vehicle Classification Data

Six broad study areas may be identified in which vehicle classification data are essential. These study areas include:

- design and performance of road pavements and bridges
- traffic capacity and operations analysis
- vehicle categories for traffic legislation and regulation
- road safety studies
- parking system design
- economic studies.

The first two items above are of immediate concern to the traffic engineer, and brief descriptions of the needs for classification data in these two areas are given below. Figure A 10 also provides a framework on the various uses of classified counts (Vincent 1986).

Figure A 10: Usage of vehicle classification data in transport engineering



Source: Vincent (1986).

Road pavements and bridges

The total loads borne by a road element, and the distribution and frequency of individual loads are important in predicting and assessing the performance of that element. The structural design of pavements and bridges is a primary use for vehicle classification data. Structural design requires data on the distribution of axle loads, the frequency of axle loads, and the configuration of axles (hence vehicle types). Note that road pavement design and thereby costs are sensitive to the presence of outliers in the patterns of axle groups taken as a traffic stream, i.e. not necessarily individual vehicles but groupings of vehicles. Traditionally, axle-load data have been collected as by-products of legal load enforcement procedures. Present needs require direct attention to the actual loads using a particular facility such as a bridge, and weigh-in-motion technology is available to efficiently obtain this data (Appendix G).

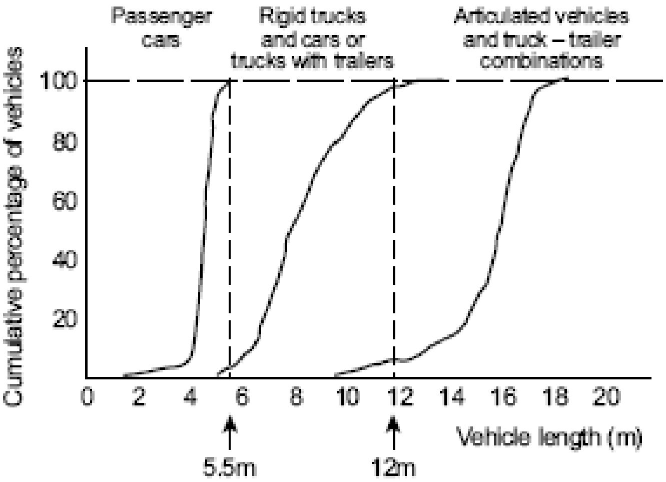
Traffic capacity and operations

The ability of a road system to cope with the demands imposed on it depends to a large degree on the dimensions and performance characteristics of the vehicles using it. Low-powered and/or large vehicles may find difficulty in climbing grades, thereby reducing the LOS for a given traffic flow, unless special provision is made to allow other vehicles to overtake. Standards for intersection design and channelisation, and road geometry, are largely defined in terms of the ability of a design vehicle to negotiate curves and corners, and comply with traffic regulations without demolishing roadside furniture. The design vehicle is usually taken as one having a swept path envelope representing at least 85% of the population of that type of vehicle. Estimates of travel and operating costs for different vehicle classes are necessary in economic studies of road transport needs.

A.5.2 Vehicle Classification Systems

A number of classification systems are currently in use. A system that groups vehicles into three categories is illustrated in Figure A 10. This involves the automatic classification of vehicle lengths using inductive loop sensors. The system identifies distinct break points in vehicle length distributions for cars, rigid trucks and articulated vehicles. Figure A 11 shows the distribution of vehicle lengths in these three broad classes, indicating the break between cars and rigid trucks occurring at a length of 5.5 m, and between rigid and articulated trucks at 12.0 m. Overlaps or uncertainties always exist at each break point.

Figure A 11: Length distributions of vehicles on Victorian highways in 1985



Source: Vincent (1986).

Table A 8 shows the current Austroads vehicle classification system that was updated in 1994. This system provides for three levels of vehicle classification – Level 1 by overall vehicle length, Level 2 by number of axles, and Level 3 by axle configuration and vehicle type. It is illustrated in Figure A 12. The system allows flexibility for any user to aggregate vehicle classes in various ways to suit particular purposes and economies of data collection.

The classification data can be used in conjunction with axle-load data from strategically located weigh-in-motion sites to develop load factors. Heavy vehicle counts can be converted to equivalent standard axles (ESA) for road pavement design (Appendix G). Three-bin and four-bin systems are being used for classification of vehicles by length. At present, the Roads and Maritime Services Incident Management System employs a three-bin system and VicRoads employs a four-bin system, both of which are different from the Austroads five-bin system classified by vehicle length. The Australian Bureau of Statistics provides data related to heavy vehicle activity using different classification categories (Appendix G).

Table A 8: Austroads vehicle classification systems (updated in 1994)

Level 1	Level 2		Level 3	Austroads classification	
Length (indicative)	Axles and axle groups		Vehicle type		
Type	**Axles**	**Groups**	**Description**	**Class**	**Parameters**
	Light vehicles				
Short Up to 5.5 m	2	1 or 2	Short Sedan, wagon, 4WD, utility, light van, bicycle, motorcycle, etc.	1	$d_1 \leq 3.2$ m and axles = 2
Medium 5.5 m to 14.5 m	3, 4 or 5	3	Short-towing trailer, caravan, boat, etc.	2	groups = 3, $2.1 \text{ m} \leq d_1 \leq 3.2$ m $d_2 \geq 2.1$ m, and axles = 3, 4 or 5
	Heavy vehicles				
	2	2	Two axle truck or bus	3	$d_1 > 3.2$ m and axles = 2
	3	2	Three axle truck or bus	4	Axles = 3 and groups = 2
	> 3	2	Four axle truck	5	Axles > 3 and groups = 2
Long 11.5 m to 19.0 m	3	3	Three axle articulated or rigid vehicle and trailer	6	$d_1 > 3.2$ m Axles = 3 and groups = 3
	4	> 2	Four axle articulated or rigid vehicle and trailer	7	$d_2 < 2.1$ m, or $d_1 < 2.1$ or $d_1 > 3.2$ m Axles = 4 and groups > 2
	5	> 2	Five axle articulated or rigid vehicle and trailer	8	$d_2 < 2.1$ m, or $d_1 < 2.1$ or $d_1 > 3.2$ m Axles = 5 and groups > 2
	6 > 6	> 2 3	Six axle (or more) articulated or rigid vehicle and trailer	9	Axles = 6 and groups > 2; or axles > 6 and groups = 3
Medium combination 17.5 m to 36.5 m	> 6	4	B Double or heavy truck and trailer	10	Axles > 6 and groups = 4
	> 6	5 or 6	Double road train or heavy truck and two trailers	11	Axles > 6 and groups = 5 or 6
Long combination over 33 m	> 6	> 6	Triple road train or heavy truck and three trailers	12	Axles > 6 and groups > 6

Definitions:

Group: (axle group) – where adjacent axles are less than 2.1 m apart

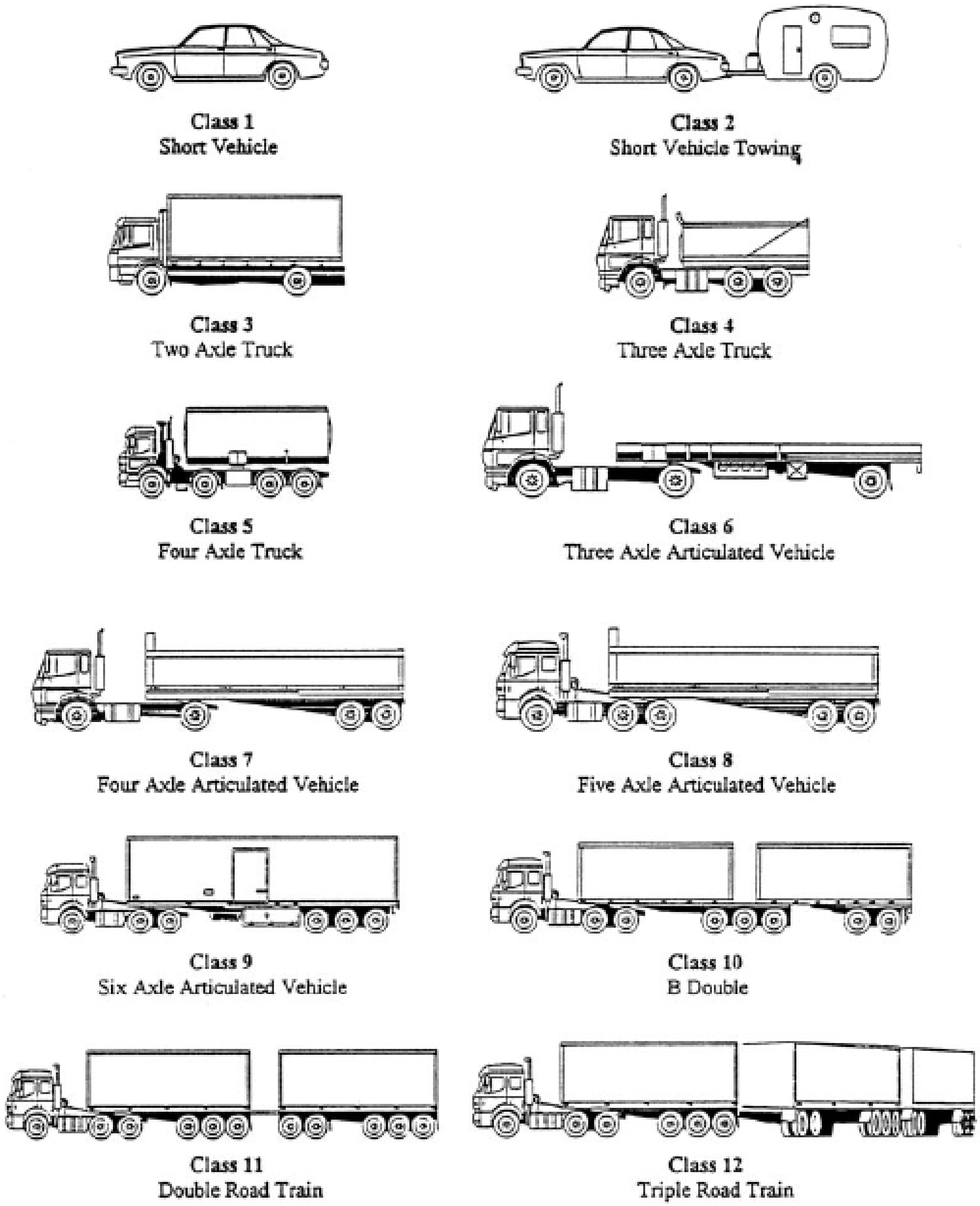
Groups: number of axle groups

Axles: number of axles (maximum axle spacing of 10 m)

d1: distance between first and second axle

d2: distance between second and third axle.

Figure A 12: Representative vehicles in Austroads 12-bin classification system



A.5.3 Methods of Collecting Vehicle Classification Data

Manual vehicle classification methods, based on either vehicle body type (e.g. surveys by the Australian Bureau of Statistics) or axle configurations (e.g. Austroads), have been used for many years. Manual methods are now largely confined to intersection turning movement counts. As these surveys require considerable human resources, they are costly and generally limited to short period counts – generally up to 12-hours duration.

Automatic vehicle classifiers are now readily available. They usually use a pair of axle sensors for classification by axle configurations or a pair of inductive loops for classification by vehicle lengths (Appendix A.1). More accurate classified counts are possible by using an extra sensor, e.g. two loops plus an axle sensor. However, uncertainties are present in vehicle counting and classification, especially under congested flow conditions. Modern vehicle classifiers are programmable to suit different classification systems and are available at low cost. They should replace simple vehicle or axle-pair counters when these are due for replacement.

As mentioned in Appendix A.1, lidar, VID and infra-red technologies are also able to provide vehicle classification information. In recent years, electronic tolling systems have employed video imaging for law enforcement and charging tolls. Numberplates and vehicle profiles are used for vehicle identification and classification. Video imaging is therefore a potentially useful vehicle classification technology with a high accuracy (up to 99.5%). It is expected to be more commonly employed as the cost of implementing the technology decreases.

A new non-intrusive technology based on intersecting infra-red beams crossing a carriageway is gaining extensive use (i.e. TIRTL). As discussed in A.1.2, TIRTL has the capability of providing accurate classified counts on multi-lane highways according to the Austroads 12-bin system by measuring the number of axles, axle separation, wheel width and the front-to-back wheel width ratio.

Research on corrective software to improve the accuracy of vehicle classification counts (using loops or axles) has progressed well. Roads and Maritime Services has employed such software, which needs to be calibrated against sample video clips of traffic flow for vehicle classification surveys.

A.6 Data Analysis and Presentation

Traffic data should be presented in ways that make them easily and quickly understood, by people who are unfamiliar with the methods used to collect data. Large volumes of precise data must generally be presented as numerical tabulations. Graphical presentations permit rapid visual interpretation, particularly for variations in time and space, and are sufficiently accurate for most purposes. Modern computer graphics systems can readily generate time profiles and flow maps.

It is important to precisely record and identify the location of counting sites. This may be done either on the basis of a permanent reference system devised for general inventory purposes, by verbal description, or by making a locality plan. The availability of GPS devices and GIS software has greatly facilitated data presentation. Unusual variations from such causes as extremes of weather, public holidays, crashes or the like should be recorded.

A.6.1 Estimated AADT

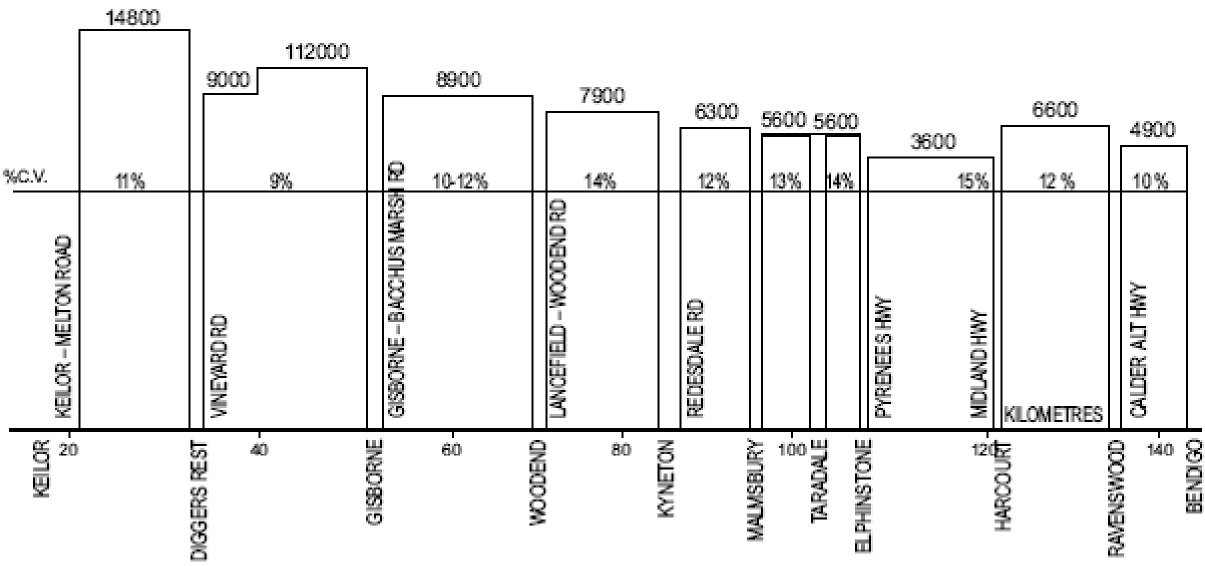
Estimated AADT profiles for roads and highways are normally obtained from short-term counts, using the methods outlined in Appendix A.4.3. They represent the bulk of published data, particularly for rural areas. A common method of presentation is to show the observed AADT at each counting station on the map. Figure A 13 shows a typical AADT map. Alternatively, flow bands proportional to the AADT volumes are drawn along the surveyed routes.

Figure A 13: Typical map presentation of traffic count data

Source: DPTI, South Australia.

Another method is to plot AADT profiles for only selected routes, and is particularly useful in urban areas where there may be many routes close together so that flow-bands overlap. Here the route is represented schematically by a straight line (the abscissa), and the AADT value for each route segment is shown as an ordinate. Figure A 14 shows an example.

Figure A 14: Typical AADT profile for a route



Source: VicRoads.

The duration of the counts and the adjustment factors used in the estimation of AADT may be of interest to those familiar with traffic counting methods. These data are usually presented in tabular form.

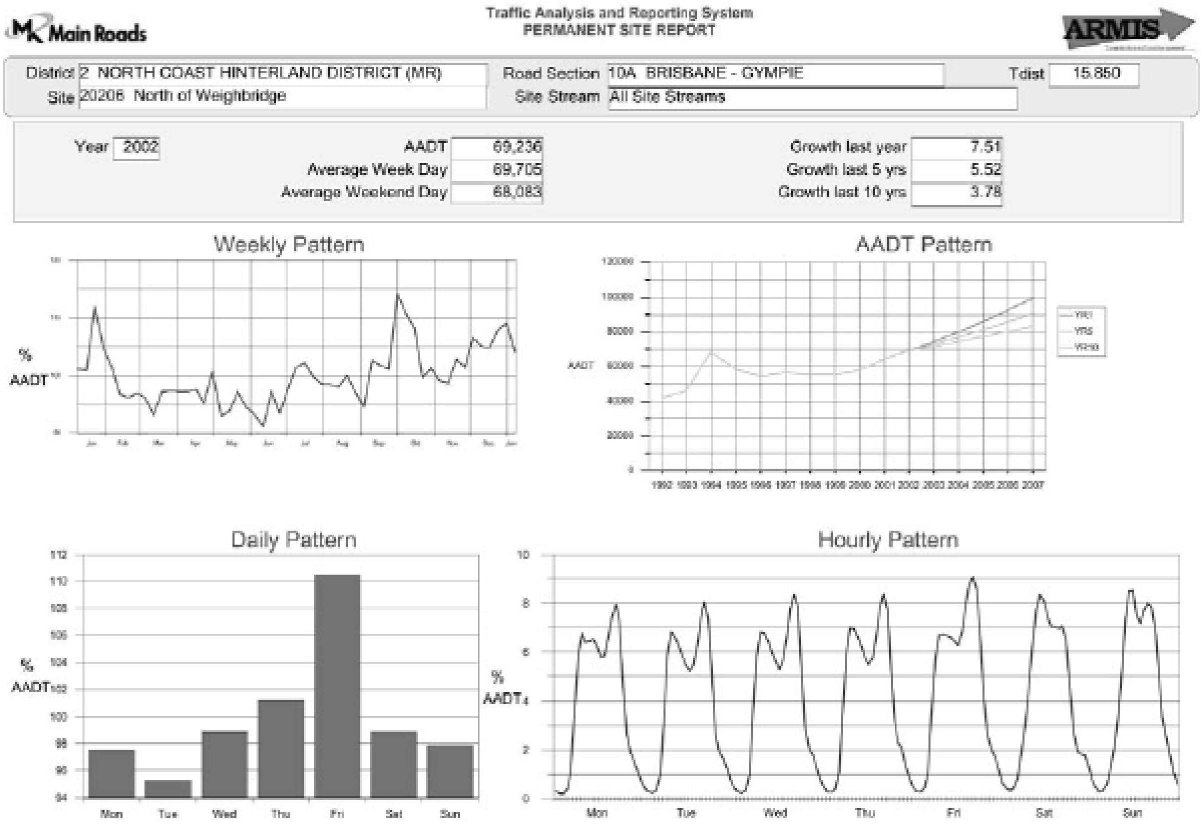
A.6.2 Seasonal Patterns and Trends

Pattern stations provide information on daily and seasonal variations, while permanent stations also provide indications of trends.

Graphical and tabular presentations may be used to show monthly, weekly and daily variation patterns, often illustrated as proportions of average values. Graphs showing such variations arranged in decreasing order are also useful, as they show information on variations about the mean.

A typical pattern station report is shown in Figure A 15.

Figure A 15: Typical pattern station report

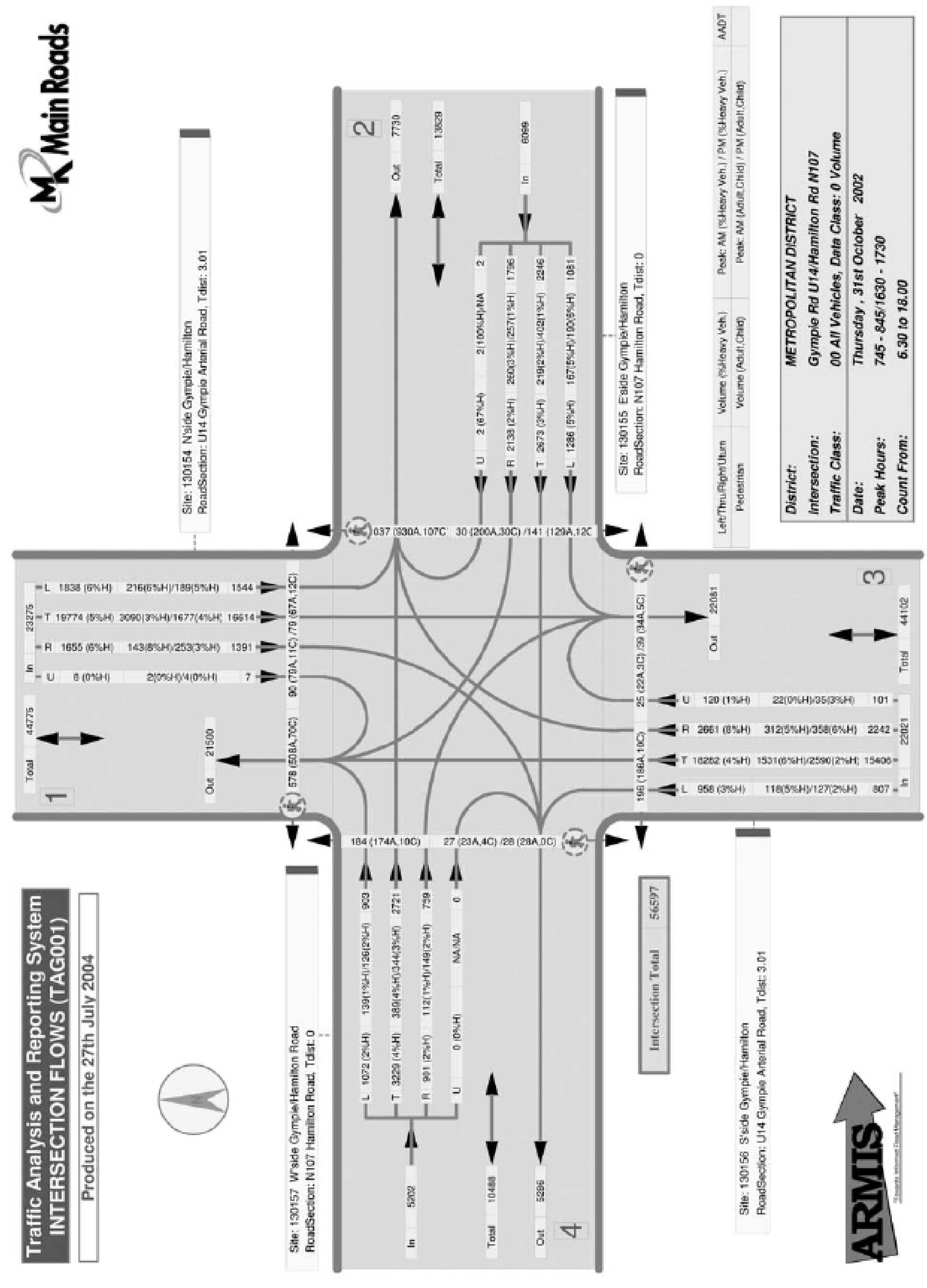


Source: Department of Transport and Main Roads, Queensland.

A.6.3 Turning Movements at Intersections

Information on turning movements is usually obtained from manual counts over relatively short periods, typically the peak hour or over 12 hours from 7.00 am to 7.00 pm. These data are essential for intersection and traffic signal design. Turning movement counts may be segregated into two vehicle classes – cars and heavy vehicles – and these may be displayed, each as a number of cars and heavy vehicle component or as a percentage of the particular total traffic movement. Figure A 16 shows a typical example.

Figure A 16: Typical intersection count report



Source: Department of Transport and Main Roads, Queensland.

Appendix B **Speed Surveys**

B.1 Definitions

There are three speed classifications that are of interest in traffic engineering:

- **spot speed** – the instantaneous speed of a vehicle at a specified point on a road
- **journey or space speed** – the effective speed of a vehicle on a trip between two points obtained from the ratio of distance travelled to the time taken
- **running speed** – the average speed over a trip while the vehicle is moving, given by the distance travelled divided by the time the vehicle is in motion.

A comparison of journey speed and running speed for a road provides a measure of the congestion on that road.

The average speed of a traffic stream can be defined in two ways:

- **time mean speed** – the arithmetic mean of the measured spot speeds of all vehicles passing a fixed roadside point during a given time interval
- **space mean speed** – the harmonic mean of the measured speeds of all vehicles in the stream which are within a specified length of roadway at a given instant of time.

For a given time period, a roadside observer tends to see more cars that are at higher speeds and hence spot speeds are biased in favour of the faster vehicles on the road. The time mean speed is therefore slightly greater (typically 2–3% higher) than the space mean speed. The relationship between time mean speed and space mean speed is described in Austroads (2015a) and in other texts such as Taylor et al. (2000).

Space or journey speed belongs more properly to travel time investigations (journey speed is the inverse of travel time per unit distance) and is discussed in detail in Appendix C. This section concentrates on spot speeds, their measurement, analysis and uses. For both spot speeds and space speeds, further disaggregation of the concept of speed is possible on the basis of driver behaviour. A common system is:

- **actual speeds** – the speeds that would be observed for all vehicles on a road
- **desired speeds** – the hypothesised speeds which drivers will adopt when free from all other influences
- **free speeds** – the observable part of the total population of desired speeds.

B.1.1 Speeds on Curves

Advisory speeds are the desirable speeds for horizontal curves, intersections or other locations where design standards or physical conditions restrict safe operating speeds to a value below the posted regulatory speed limit on a roadway.

The speed at which a driver can negotiate a horizontal curve is related to the superelevation of the pavement, and the friction developed between the tyres and the road. An equation based on the acceleration of a body travelling a circular path applies. The acceleration is measured away from the centre of the path and is given by Equation A6:

$$\frac{v^2}{R} = (e + f)g \qquad \text{A6}$$

where

v = speed

R = radius

e = superelevation

f = friction coefficient

g = acceleration due to gravity

For convenience, this equation can be expressed as Equation A7:

$$v = \sqrt{127R(e + f)} \qquad \text{A7}$$

where

v = speed (km/h)

R = radius (m)

If a driver attempts to negotiate the corner faster than the speed calculated there is a greater risk of a crash. Drivers would typically prefer to use a friction coefficient which is significantly less than the maximum possible value at a particular site.

Consider a rural road with R = 166 m, e = 0.04 and f = 0.16. The advisory speed is therefore 65 km/h.

AS 1742.4 describes the use of the ball-bank indicator to establish appropriate advisory speeds on curves.

Variable speed limits and warning displays have been used by member jurisdictions to match traffic and environmental conditions including fog and icy pavement conditions. It is known from experience in countries with foggy weather conditions that displaying lower speeds on motorways when warranted can reduce road crashes, especially those that occur after the initial or primary crashes.

B.2 Methods of Speed Data Collection

Measurements of spot speeds are generally made from a specific location on the road. Various approaches may be used to collect spot speed data:

- methods involving timing
- microwave radar using the Doppler effect
- direct measurement using laser gun
- methods involving video
- global positioning system (GPS).

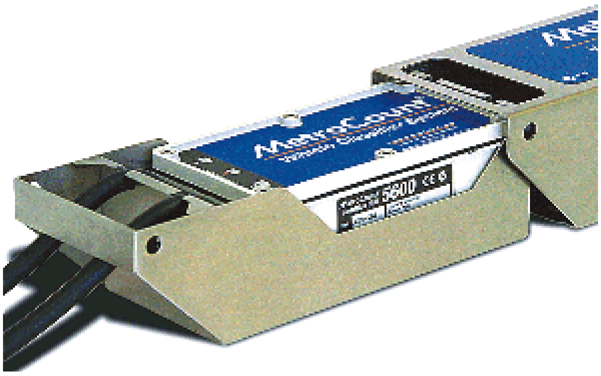
B.2.1 Methods Involving Timing

The increasing availability of electronic time and data recorders has meant that manual timing of vehicles using a stopwatch is now used only as a last resort. The passage time of a vehicle between two detectors, a measured distance apart, can easily be recorded. Detectors can be placed close together (a few metres apart) as high levels of accuracy are obtainable. Thus a more representative spot speed can be measured over that short distance and acceleration and deceleration effects will be minimal. Data are recorded and can be uploaded at regular intervals either on site or remotely. Additional data, such as headway and volume data can also be collected simultaneously.

Various detectors can be used in this type of system, as discussed in Appendix A.1.2. These include pairs of pneumatic tubes, tribo and piezoelectric cables, switch tapes, inductive loops and photo-electric or electromagnetic beams.

An example of such a measuring system is the MetroCount vehicle classifier system (MetroCount n.d.) as shown in Figure B 1.

Figure B 1: The MetroCount vehicle classifier system



B.2.2 Microwave Radar Gun Using Doppler Effect

The earlier generations of microwave radar guns for speed measurements make use of the Doppler effect. A microwave beam is sent to the target vehicle, which reflects a signal back to the receiver in the radar gun.

The moving vehicle affects the frequency of the returned signal. The shift in the frequencies between the emitted and received microwave signals is called the Doppler effect. By measuring the amount of frequency shift and the duration of the time interval, the speed of the targeted vehicle can be determined. A microwave radar gun has a wide cone of detection, which is about 70 m at a range of 300 m. It cannot be relied upon to give an accurate speed measurement of a particular car, and has been replaced by laser devices for law enforcement.

B.2.3 Direct Measurement Using Laser Guns

The laser infra-red gun uses the higher optical frequency and has a small detection cone of about 1 m in diameter at a distance of 300 m between the laser gun and the targeted vehicle. An example of a hand-held laser gun is shown in Figure B 2.

Figure B 2: An example of a laser speed measurement device

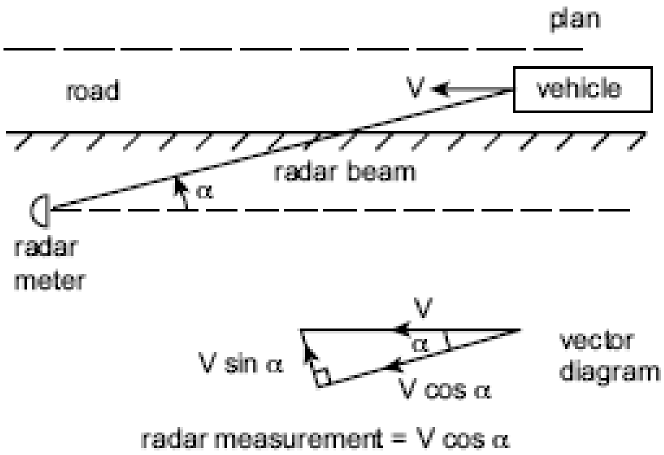
The equipment employs a direct method because it relies on the measurement of the round-trip time of the infra-red light beam to reach a vehicle and be reflected back. The gun can accurately count the number of nanoseconds the light takes for the round trip, and making use of the speed of light at 300 000 km/s, several samples of the distance are obtained in a fraction of a second. The changes in distance and hence the velocity as the targeted vehicle moves can therefore be monitored accurately.

Two types of error may be found when radar or laser speed meters are used to measure the speeds of isolated vehicles:

- **equipment error**, i.e. the rounding-off in the displayed speed; typically this is (–1, +0) km/h, e.g. for an actual speed of 100 km/h, the meter reading would be between 99 and 100 km/h
- **angle error**, which is related to the actual angle of incidence of the radar beam to the direction of travel of the vehicle.

Figure B 3 shows a typical configuration. Unless the meter can be placed in the path of the vehicle, there will be a finite angle α between the beam and the vehicle path.

Figure B 3: Angle of incidence (α) of a radar beam to traffic



The speed reading from the meter is thus v. cos(α) where v is the actual vehicle speed. Typically α will be quite small, so this error will also be small and is given by [100(1 – cos(α)) percent].

Note that this is a consistent error, and that the observed speed reading will always be less than v. Some units have a built-in correction for angle error, usually for a predefined value of the angle α. As the mass market for the units is law enforcement, the correction is often ignored. The rationale for speed limit enforcement is that the observed speed will never exceed the actual speed of the vehicle.

Both types of error mean that observed readings will always be slightly less than actual speeds (a margin of leniency in speed limit enforcement). Because the laser equipment measures speed directly (with a set absolute error in reading speed, depending on the set-up position of the unit), the relative error will decrease as speed increases. This is the opposite result to that noted for indirect speed measurements based on observed travel times.

B.2.4 Methods Involving Video

Video can be used to determine vehicle speeds and is becoming increasingly cheaper to use and operate. The general method involves recording the distance moved by a vehicle in a short period (perhaps a couple of frames), then computing the speed. An example is the Video Analysis Data Acquisition System (VADAS) reported in Troutbeck and Dods (1986), which is now replaced by new imaging equipment at ARRB (e.g. Roper 2004). The advantages of using video include:

- the provision of a complete, permanent record of the traffic flow, which can always be re-analysed and re-examined at a later stage
- additional information (e.g. vehicle classification, flows, headways, special phenomena, overtaking, etc.) can be obtained.

Video imaging techniques are used on freeways in various cities to extract speed data from fixed-position cameras. The operator defines virtual loops on a carriageway and superimposes them on the video frames captured by the camera. The luminance of the marked area changes as one or more vehicles travel over the area. These changes are detected and provide information such as speed, volume, headway and occupancy. The accuracy of speed measurement using video is affected by shadows of adjacent lane traffic and weather conditions, and is expected to be less than that obtained using inductive loop sensors. Video techniques offer the significant advantage of remote sensing and do not require cutting loops on road pavements – an important issue for privately operated tolled roads.

B.2.5 Global Positioning System

Vehicles can be fitted with receiver units that pick-up signals from the global positioning system (GPS) satellite network (Figure B 4). GPS was the first satellite navigation system launched in 1978. It is technically known as NAVSTAR GPS (NAVigation Satellite with Timing And Ranging Global Positioning System). The system is designed, maintained and operated by the US Department of Defence. Among all the extra-terrestrial positioning systems available today, GPS is the most popular and widely used positioning system. It is a worldwide navigation tool that provides three-dimensional position, velocity and time information under all weather conditions for 24 hours a day (Khoo & Luk 2002).

Figure B 4: An example of a GPS receiver unit

The complete constellation of GPS consists of 24 satellites with three operational spares circulating the earth in six orbital planes. The GPS was previously set up for military purposes. Civilians are allowed to use most facilities of the full system.

Two techniques are used to determine distances or ranges. They are the code-based and the phase-based measurements. The code-based measurement makes use of the coarse acquisition code (CA) and can achieve a maximum positioning accuracy of about 3–5 m. The phase-based measurement attempts to improve this accuracy by considering further the information contained in the phase of the carrier and is an active area of research.

Satellite positioning using one receiver is known as single-point positioning or simply point positioning. This is the most basic usage of satellite systems for positioning and navigation. It is widely used in recreational navigation like hiking, sailing and hunting. A point positioning accuracy of about 10 m with a probability of 95% is now possible with code-based GPS. The accuracy of single-point positioning is too low for serious navigation, surveying and other vehicle location applications.

A relative or differential positioning technique is necessary to improve the accuracy. It involves the use of two receivers, one stationary at a reference or base station and the other (called a rover) moving or simply located at a place usually in the vicinity of the reference station. Both receivers simultaneously track the same set of satellites. The errors common to both receivers can be mitigated and hence the accuracy obtainable will be higher. Note that four satellites are needed to fix a position with certainty because two circles representing the two ranges from two satellites intersect at two points, and a third satellite is needed to give a third range to determine which of the two intersecting points is the correct distance. Four satellites are therefore necessary to give a correct three-dimensional position fix.

The accuracy of code-based differential GPS (DGPS) is about 2–3 m with a baseline distance (i.e. range of coverage) of 100–200 km. A phase-measuring DGPS is expensive with a much higher accuracy in the range 20–50 mm, but the baseline distance has been found to be less than 10 km.

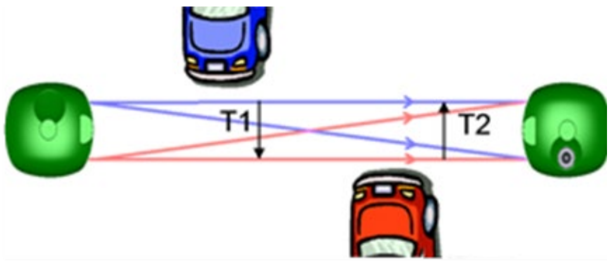
The GPS receiver, whether in point processing or differential mode, keeps track of its position and time. From time and position, it can also calculate its speed when in motion. Modern receivers now make use of a combination of time, distance and frequency-shift signals to calculate speeds and other parameters. The speed values can be useful as an indicator of the road congestion condition. In Singapore, such data are transmitted from GPS-equipped taxis to a traffic control centre. The data are processed and then made available in real-time on the Internet as congestion indicators (Luk & Yang 2001).

B.2.6 Method Using Infrared

Infrared vehicle detectors operate by transmitting a high-energy beam across or down on the roadway and can detect vehicle numbers, height, width and length and accordingly provide vehicle counts, speed and classifications. For example, TIRTL uses two electronic devices (receiver and transmitter units) mounted on either side of a road. It uses two infra-red light beams passing above the road surface to detect and record vehicles and their respective attributes. The two beams are separated by 153 mm and span a road at wheel height.

As a vehicle moves along the road its tyres block and unblock each of the beams (Figure B 5). The TIRTL receiver detects and records the times that the first and second parallel beams are blocked and unblocked by the tyres of a vehicle. Using these times, combined with the distance between the beams, the TIRTL receiver calculates the speed of each vehicle. TIRTL calculates the speed of a vehicle using the times that the beams are blocked and unblocked thereby recording up to 4 speeds for a 2-axle vehicle.

Figure B 5: Illustration of TIRTL devices



Source: CEOS (n.d.)

TIRTL has been certified by National Association of Testing Authorities as a high-accuracy speed measurement device (0.3-0.5 error level, 1.2 km/h uncertainty), and has been widely used as an auditor for speed compliance and tolling systems.

B.3 Sample Selection

Section 2 describes the processes for defining study objectives, selecting survey measures, and selecting target populations. This appendix describes the procedures and methods for determining an appropriate sample size for a proposed speed survey.

B.3.1 Selecting a Target Population

The vehicle speeds to be recorded, and hence the vehicles to be observed, must depend on the specific objectives behind the study. This is of particular importance given the discussion of different concepts of speeds (e.g. actual, desired or free speeds).

If actual speeds are of interest, measurements can be taken from all of the vehicles on a road. With modern data loggers, large volumes of vehicles can be recorded with ease in comparison to older manual recording methods.

Traditionally, free-speed data have been collected by measuring the speeds of **isolated vehicles** and **bunch leaders.** This will yield results biased in favour of the slower vehicles. McLean (1978) introduced a method for correcting this bias on the assumption that free speeds are normally distributed. Brilon (1977) suggested that the speeds of isolated vehicles will be biased in favour of the faster vehicles, so that the desired speed distribution would lie somewhere between that for the isolated vehicles alone and that for the free-moving vehicles (i.e. both isolated vehicles and bunch leaders). Hence Brilon’s method is to measure both the distributions of speeds of isolated vehicles and bunch leaders, and to use these as upper and lower bounds on the distribution of free speeds.

B.3.2 Behavioural Factors

Because of the enforcement of speed limits, drivers are naturally wary of any apparent speed measuring system on or near the road. Detectors on the road surface may cause drivers to slow down. Some drivers may have radar detection apparatus mounted in their vehicles. These devices are now illegal in many places. Speed studies for traffic engineering purposes are intended to provide information about the normal behaviour of drivers. Speed surveys should therefore be conducted in as unobtrusive a manner as possible.

B.3.3 Sample Size for Speed Studies

Most investigations of vehicle speed relate to the range of speeds adopted by vehicles in the traffic stream, rather than a single observation such as the mean speed. Thus the overall distribution of speeds is of interest, and samples must be selected so that useful data on the speed distribution can be obtained.

For most purposes, the minimum sample size *n* required for a speed survey may be determined as in Equation A8 (Oppenlander et al. 1961):

$$n = \frac{V^2 s^2 (2 + U^2)}{2d^2} \quad \text{A8}$$

where

V = normal deviate corresponding to the desired confidence level

s = standard deviation of the sample

U = normal deviate corresponding to the percentile being estimated

d = permitted error in the estimate

A normal deviate is a random variable which follows a normal distribution. For convenience, values of $V^2(2 + U^2)$ are given in Table B 1 for common values of *V* and *U.*

Table B 1: Values of V^2 (2 + U^2) for use in sample-size determination for speed surveys

Percentile being estimated	Desired confidence level (%)		
	90	**95**	**99**
15	B.4 8.3	B.5 11.8	B.6 20.5
50	5.4	7.7	13.3
85	8.3	11.8	20.5

For example, if *d* is defined as ±3 km/h, and the mean speed needs to be determined (i.e. 50th percentile speed, assuming speed distribution is normal) at a 95% confidence level, then from Table B 1, $V^2(2 + U^2)$ would be equal to 7.7 and therefore the minimum sample size *n* required to determine the mean speed, is given by Equation A9:

$$n = \frac{s^2 (7.7)}{2(3^2)} \quad \text{A9}$$

And therefore

$$n = 0.428\, s^2$$

where the standard deviation of speeds s is in km/h

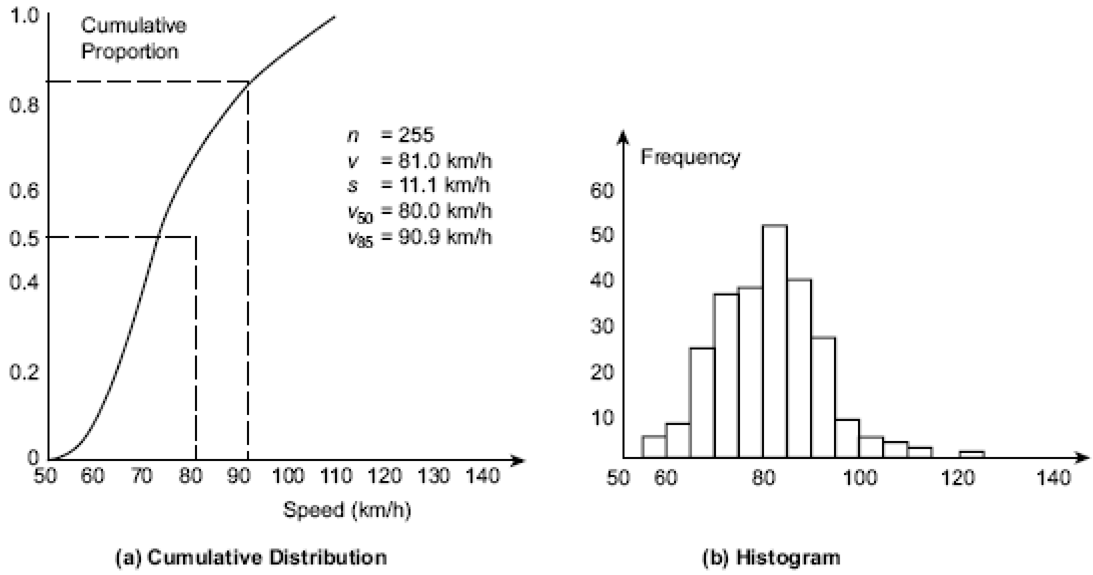
According to McLean (1989), the standard deviation of free speeds on rural highways in Australia is about 14% of the mean speed on the highway, for mean speeds of 90 km/h or more. For mean free speeds less than 90 km/h, this percentage (the ‘coefficient of variation’ of the speed distribution) decreases by about 1% for every 10 km/h decrement in the mean. Thus, if the mean free speed is 80 km/ h, the standard deviation *s* will be about 13% of 80 km/h or 10.4 km/h. This result is for all vehicles in a traffic stream. The standard deviation for trucks would be somewhat less, perhaps 11% of the mean truck speed (in level terrain).

B.4 Data Analysis and Presentation of Results

Speed data typically need analysis of their distributions. Measures only of their central tendencies (e.g. the mean speed) are seldom sufficient indicators of the overall speed situation. The extremes of observed speeds set a range of conditions, which is useful in traffic design. For this reason the results of a speed distribution are usually expressed in graphical terms either as a histogram or (more commonly) as a cumulative distribution. Measures such as the mean, median and 85th percentile speeds, and the standard deviations are then extracted from the distribution. A measure such as the 15th percentile speed might be used to indicate difficulties for some vehicles in maintaining travel speeds (e.g. for trucks climbing grades). Jurisdictions (e.g. TMR) also require the 15 km/h pace speed and percentage within pace to be extracted for speed analysis in speed limit reviews.

Figure B 6 shows a typical cumulative speed distribution curve. Median and 85th percentile speeds are illustrated, and compared to the mean speed. This figure is the typical output from a spot-speed survey.

Figure B 6: Typical cumulative speed distribution and histogram from a spot-speed survey



In before and after analysis, statistical tests are used to compare the observed distributions and their parameters, in a search for significant differences which might indicate changes in driver behaviour. Other variables (e.g. flow) must be held constant or multivariate analysis carried out.

B.5 Conclusions on Speed Surveys

Speed surveys are an important part of the traffic engineer’s investigations. Although they often appear to be routine exercises, care is always needed in defining the types of speeds to be observed and in selecting the experimental techniques to be used. There are definitional problems to be resolved in deciding just what speed data are appropriate to a particular study objective. Speed surveys are, therefore, prime examples for the application of the systems planning approach to traffic studies outlined in Section 2. Speed data can be collected using both indirect and direct methods, many utilising new technologies. The most suitable survey method for a particular study will depend on many factors, including the study objectives, the characteristics of the observation site, the data collection and analysis resources available, and the expected range of speeds to be observed. Other data collection methods which focus on the variability of travel times are available, and should also be considered. These methods are described in Appendix C and Appendix K.

Appendix C Travel Time, Queuing and Delay Surveys

All studies of travel times in traffic systems need to be approached carefully. There are many parameters and definitions to be considered, and the nature of the underlying variable – the travel times of individual flow units in the traffic – is complex. In particular, travel time variability is recognised as an important parameter in travellers' route choice decisions, and also provides information on the congestion levels of a traffic network (Austroads 2011, Section 6.5; Taylor 1982).

There are a large number of definitions and components in travel-time investigations, and the following definitions are commonly used:

- free-flow time – the time required by an unimpeded vehicle to traverse the survey section
- free-flow speed – the length of the survey section divided by the free-flow time
- travel time – the actual (observed) time taken to traverse the test section
- delay – the difference between the travel time and the free-flow travel time
- stopped time – the period for which a vehicle is stationary while in the survey section
- running time – the period of time for which a vehicle is in motion while in the survey section; (total) travel time is then the sum of stopped time and running time
- running speed – the total sectional distance divided by the running time; sometimes running speed is used as an estimate of free-flow speed.

C.1 Travel Time Surveys

Travel times may be observed in two different ways. The first way involves 'moving observer' methods where the observer drives a test car in the traffic stream and records its passage time. The second way involves 'stationary observer' methods, where observers placed at fixed points in the network record the times that a sample of vehicles take to travel the length of the survey section. Table C 1 lists the various travel time survey methods described in this appendix, and also indicates the data items that may be found using each method.

Table C 1: Travel-time survey methods

Basic approach	Survey method	Application	Data obtained
Moving observer	Floating car	Long road sections	Mean travel time, stopped time and running time; number of stops; flows
	Chase car	Long road sections	Mean and variance of travel times, stopped time and running time; number of stops
Stationary observer	Number plate	Road sections, junctions, traffic control devices	Mean, variance and distribution of travel times and flows
	Input-output	Road sections, junctions, traffic control devices	Mean travel time and flow
	Path trace	Short road sections, junctions, traffic control devices	Mean and variance of travel time, free-flow time, running time; number of stops

C.1.1 Moving Observer Methods

Moving observer methods enable estimates of parameters such as mean travel times to be made fairly easily and quickly. The basic resources required are a test vehicle, two people (driver and observer), and a data recording system. Data recording may be by paper, pencil and stopwatch, by hand-held or laptop computer, or by an instrumented vehicle. An instrumented vehicle is one equipped with an 'in-built' computer data recording system. Two moving observer techniques are the floating car and chase car methods. The development of more sophisticated technology has facilitated the use of these methods, e.g. the GPS described in Appendix B.2.

Floating car method

The floating car method measures the mean travel time along a test section. A test vehicle attempts to simulate an 'average' vehicle in the traffic stream, by noting the number of vehicles that overtake the test car, and the number of vehicles that the test car overtakes. The driver tries to equate these two numbers of overtakings, and the test vehicle is said to be 'floating' in the traffic if the two numbers are the same. If it is not possible to equate the numbers of overtakings by the end of the test section, a correction to the observed travel time can be applied (Chapter 9 in Institute of Transportation Engineers 2010a).

The floating car method is appropriate for travel time surveys on long and perhaps complex routes. The main advantage of the method is that data can be collected easily and quickly over a number of routes. The main disadvantage is that it is difficult to collect large samples of data, as the test vehicle has to traverse the route and then return to the starting point for the next run. This all takes time, and in a dynamic environment where travel conditions (e.g. flow rates and traffic control systems) are changing rapidly, repeated runs may form biased samples of travel times from any one set of conditions. Further problems exist for the technique on arterial roads with significant levels of platooning, in which the test vehicle may have great difficulty in floating in the stream.

Johnston et al. (1982) and Luk et al. (1983) reported the experience of using the floating car technique in balanced experimental designs to determine the performance of different signal coordination methods in reducing travel time, fuel consumption and other performance indices. A fully balanced, Latin Square design is not always possible because of resource constraints but meaningful results are still possible by considering most of the factors involved.

Cowan and Eriksson (1972) evaluated the floating car method and demonstrated that it could provide an accurate estimate of the mean section travel time with only a slight bias. They also showed that the bias decreases as the flow rate increases. Appendix C.1.3 describes recent travel-time studies on bus and transit lanes in Sydney with the floating car method using vehicles equipped with GPS receivers.

Chase car method

Here the survey vehicle follows a randomly selected vehicle in the traffic stream, copying as closely as possible the manoeuvres of the chased vehicle. For travel time studies it is necessary to keep track of the time performance of the vehicle (e.g. section travel times, time stopped). Care is needed to ensure that vehicles are selected at random, or strictly according to a predetermined strategy (e.g. by tracking a certain number of particular vehicle types) – otherwise significant biases may result.

One obvious problem with the method is that the chased vehicle may leave the survey route at any time. The decision then has to be made as to whether the survey vehicle aborts the run and returns to pick up a new vehicle, continues and picks up a nearby vehicle to follow, or 'floats' in the traffic until the end of the test section. The decision will largely depend on the sampling unit for the survey. If the unit is the individual vehicle, another vehicle would have to be found and the collected data would still be useable. If the route is the sampling unit, the run would have to be aborted.

C.1.2 Stationary Observer Methods

Travel time observations by teams of observers, situated at a series of fixed points on the road network, may be used to collect data on the travel times of individual vehicles in the traffic stream. Three methods – numberplate, input-output and path-trace surveys – are described below.

Some technologies available for managing traffic operations may be adapted for traffic survey monitoring. For example, the installation of electronic toll tags in an increasing proportion of the vehicle fleet provides an opportunity to use probe vehicles as sensors to measure speeds and travel times. Further discussion is provided in *Part 9* of the *Guide to Traffic Management* (Austroads 2016b).

Numberplate survey method

The most common use of numberplate surveys is in the collection of origin-destination data (see also Appendix D). These surveys may also be used to collect data on the distribution of travel times in a section of the road network. Typically, the section will be a length of road or a single route in the network, but travel times across an area might also be collected. The survey operates by having observers positioned at selected points on the road: they record the numberplates (or partial numberplates) and arrival times of a sample of the vehicles that pass them. Setting the survey area, time and sampling rate is discussed in detail in Appendix D.2. When the data are consolidated in the office, numberplates can be matched between upstream and downstream locations, and travel times for individual vehicles computed from the difference in arrival times. The principles of the survey method are quite straightforward, but certain complications can arise in practice.

The first consideration is to ensure that the workload placed on any one observer is not too great. Beard and McLean (1974) suggested that the practical maximum recording rate for conventional data recording methods was 600 recordings per hour. The selection of appropriate recording equipment and proper training of observers are essential.

The second problem is one of natural attrition of the observed traffic stream. Vehicles entering the test section may turn off the road, or finish their journeys upstream from the exit station. Similarly, other traffic will enter the road from side streets and not pass through the entry station. The longer the section of road, the lower is the through-traffic proportion. As a result of this second problem, there is the need for a trade-off between section length and accuracy of estimating travel time and speed.

Another problem relates to start-up and shutdown errors. Vehicles will be within the survey section at the time the survey commences, and will only be observed on departure. Similarly, vehicles entering the survey section just prior to the end of the survey may not be observed departing. This bias is less significant as the duration of the survey increases (Appendix D.2.3).

The traditional surveys have been gradually replaced by automatic technologies such as automatic number plate recognition (ANPR) etc.

Input-output survey

This survey method has some similarities with the numberplate method, but is simpler in operation. It does not try to match individual vehicle observations at different stations, and thus can only provide estimates of mean travel times. The technique is based on the idea that the difference between the means of two sets of observations is equal to the mean of the differences of the two sets. Input-output surveys find the mean arrival time and the mean departure time of the traffic stream in the test section, and calculate the mean travel time by subtracting mean departure time from mean arrival time. The data collected at each station are the number of arrivals in successive time intervals. Either automatic data loggers or human recorders can collect the data. The shorter the time intervals, the more accurate is the estimate, but also the higher the workloads imposed on field staff and consequently, the less accurate the observations. A compromise for human recorders seems to be an interval of about 10 seconds, with longer intervals giving comparable results – as long as the mean travel time is longer than the interval and at least 30 intervals or 5 minutes are observed.

It is important to check that the same general group of vehicles will be recorded at each station. Allowances need to be made for start-up and shutdown errors as already mentioned. Input-output analysis is best suited to closed systems, in which vehicles entering the survey zone can only leave it via the observation points. Freeway traffic is one such system. Another application for the technique is in recording the duration of stay of vehicles in an off-street car park.

Luk (1989) reported the use of the input-output technique to measure the travel time on a 350 m section of an arterial road. The upstream and downstream measurement stations employed a pair of treadle axle detectors at each station to monitor vehicle counts and wheelbase lengths. Data loggers were used to log and process the data at a resolution of 1 second because of the relatively short travel time. The results suggested that it was viable to monitor travel times by correlating input and output traffic count time series, and that wheelbase lengths do not add significant extra information for travel-time estimation.

Travel times from traffic control systems

With a pair of sensors such as the inductive loop sensors located at regular intervals on each lane of a motorway, travel times can be estimated from the spot speeds measured along the motorway in a freeway management system. The formula to calculate the travel time between two adjacent detector stations (the link travel time) can be the distance between the two stations divided by the average of the spot speeds measured at the upstream and downstream stations. The segment or route travel times on a motorway are simply the aggregation of link travel times. They can be displayed along the motorway as real-time driver information. Such a scheme has been in operation in the Drive Time System in Melbourne (Hearn et al. 1996) with a recommended spacing of 500 m between two adjacent detector stations and a raw data collection time slice of 20 seconds. The scheme has been progressively introduced in freeway management systems in Brisbane, Sydney, Perth and Adelaide.

A motorway is a non-interrupted flow facility and vehicle speeds can remain fairly steady over a distance of 500 m except in a stop-start situation, e.g. in a peak flow period or during incidents. The estimation of travel times has been found to be accurate. It must be recognised that the displayed travel times suffer from two time lags. The first time lag of a few minutes is due to a system delay in data collection, smoothing and reporting as expected in a control system. The second time lag is due to the reality that the travel times displayed at the beginning of a journey based on current conditions could be different by the time when the vehicle arrives at its destination because of changes in traffic conditions throughout the journey. Road users need to make their own adjustments based on displayed travel times and journey start times, especially around the beginning and end of a peak period.

A similar scheme for travel time estimation on arterial roads is more difficult because the vehicle speeds between a pair of signalised intersections can fluctuate widely between speed limits and zero speeds at the stopline or during a stop. For the estimation of arterial travel times, road agencies have experimented with the processing of detector data from signal control systems (SCATS and STREAMS; see, e.g. Luk et al. 2006). These data include traffic counts, loop occupancy and signal settings. Such a scheme has now been implemented in STREAMS by TMR.

The deployment of travel time estimation through a freeway or signal control system has the benefit of securing a large number of travel-time samples throughout a day. It is an automatic, online technique to monitor the traffic performance of a road network.

Path-trace survey

This method is capable of providing accurate information about traffic movements and travel times in a restricted area. Examples are direct observation, video recordings or even vehicle tagging using GPS and other devices. The travel time and detailed path of a vehicle can be determined. The sampling of vehicles may be a problem. A suitable vantage point is also needed for direct observation or video recording.

C.1.3 Transit and Bus-lane Surveys Using GPS

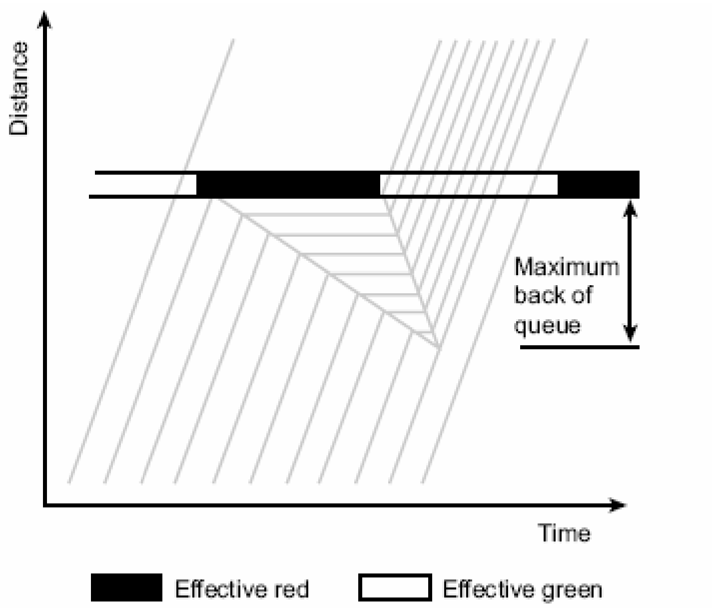
Transit and bus lanes have been important as a congestion management tool in Australian and New Zealand cities. They can provide shorter travel times for buses or high-occupancy vehicles especially during peak hours, and have the potential to attract commuters to carpool or travel by bus. Sydney has implemented more transit and bus lanes than other Australian cities. The transit lanes are known as T2 lanes for two-person carpools and T3 lanes for three-person carpools. The person-throughput on a transit or bus lane can be significantly higher than non-transit lanes (Quail & West 1992). In addition to the travel time information, vehicle-occupancy data are also collected to allow other performance measures such as person-throughput and illegal usage to be collected.

While in old days road agencies undertook regular travel-time and occupancy surveys of all transit and bus lanes using GPS equipped vehicles. The GPS equipped smart bus and electric bus ticketing systems have provided much more accurate and timely data of public transport travel time, speed and occupancy (e.g. Han et al 2016)

C.2 Queuing Survey

To illustrate the various measurements associated with a queue of stopped vehicles, consider a signalised intersection with vehicles arriving and departing uniformly. Figure C 1 presents a time-space diagram for such a case. Vehicles arriving during the red phase are halted and are not able to leave during the red phase. The vehicles stopped at the intersection can depart when the green phase starts. The vehicles stopped at the intersection when the lights turn green represent the **maximum stationary queue**. During the green time, vehicles leave the intersection at a faster rate than they arrive. Hence the queue decreases in total size, but since it takes time for the leading vehicles to start moving, the later vehicles remain stationary for some time after the traffic lights turn green. The point in time when the last stationary vehicle in the queue moves, determines the **maximum back of queue**. This represents the physical end of the queue as perceived by the driver.

Figure C 1: Time-space diagram for vehicles at a signalised intersection



Another queue that is of interest is termed the **overflow queue**. This is the number of vehicles that are still present in the queue at the end of the green plus yellow period. The above discussion relates to traffic signals and the event that determines the critical time of queue measurement is the change in phase. Queues forming at uncontrolled intersections or at other constrictions in the traffic system have the same queue formation characteristics as those present at a signalised intersection. The main difference lies in the critical time of queue measurement. In these cases, it is likely to be the departure of a vehicle from the head of the queue that determines the critical queue lengths.

Measurement of queue lengths involves an observer recording the number of stationary vehicles at a particular point in time. This can be done by physically counting the vehicles, or by placing marks along the road length to indicate the number of vehicles that would be in a queue of a given physical length. Video cameras can be used to record the queue lengths for subsequent analysis manually, or automatically employing digital imaging technologies.

Queue lengths and delay are related. The triangle in Figure C 1 represents the delay experienced in one cycle length (in vehicle-seconds) due to uniform arrivals at a signalised intersection. For non-uniform arrivals, the delay triangle becomes a delay polygon. By sampling the queue lengths at different time points of a signal cycle, the average delay can be estimated. This method is described in the following section and is known as a point-sample method.

The *SIDRA Intersection User Guide*, *Part 4* describes a survey method for the back of queue at signalised intersections (Akçelik & Associates 2011).

Additional travel time and speed survey methods and alternative data sources are available – refer to Appendix K for further details.

C.3 Traffic Delay Surveys

C.3.1 Definition of Delay

There are many definitions of delay. HCM 2000 defines five basic delay terms as:

- Control delay – the delay brought about by the presence of a traffic control device is the principal service measure in HCM 2000 for evaluating LOS at signalised and unsignalised intersections. Control delay includes delay associated with vehicles slowing in advance of an intersection, the time spent stopped on an intersection approach, the time spent as vehicles move up in the queue, and the time needed for vehicles to accelerate to their desired speed.
- Geometric delay – this is the delay caused by geometric features requiring vehicles to reduce their speed in negotiating a system element (e.g. delay experienced where an arterial street makes a sharp turn, causing vehicles to slow, or the delay caused by the indirect route that through vehicles must take through a roundabout).
- Incident delay – the additional travel time experienced as a result of an incident, compared with the no incident condition.
- Traffic delay – delay resulting from the interaction of vehicles, causing drivers to reduce their speed below the FFS.
- Total delay – the sum of control, geometric, incident, and traffic delay.

If on a straight section of road and there is no traffic, there will be no delay. Add a curve to the road but still with no other vehicles on the road then a geometric delay is introduced. Add a stop sign or install traffic signals, but still with no other vehicles on the road then a control delay is introduced. Increase the traffic and a traffic delay is introduced. The additional delay above the control delay is the traffic delay and this delay is also dependent on the type of control. Finally, if there is an incident then the additional delay is an incident delay.

Stopped delay is the delay experienced by vehicles that have actually stopped. It is a part of both the control delay and the traffic delay.

In general, delay (D) on a road section is given by Equation A10:

$$D = T - T_b \tag{A10}$$

where

T = the journey time

T_b = a base travel time represented usually by the FFS

Other choices of the base speed include the posted speed limit or average speed (Reilly & Gardner 1976).

The travel-time survey methods in Table C 1 are directly applicable for the measurement of delay on road sections and are not repeated here. The use of *the* **point-sample** method for measuring intersection delay requires further discussion.

C.3.2 Measurement of Intersection Delay

The measurement of intersection delay using the point-sample method requires the identification of two locations, which could be the stop line and a point further back than the tail end of any expected queue. The sampling can be at small, regular intervals for **stopped delay** or at specific time points of a signal cycle for **overall delay**.

Stopped delay

Stopped time delay can be collected by a manual method. The method is based on the calculation of stopped delay, D_s, from Equation A11:

$$D_S = \sum_{i=1}^{N} [ET_i - ST_i] \tag{A11}$$

where

N = total number of vehicles stopped

ST_i = time when vehicle i stopped

ET_i = time when vehicle i started

This method separately records stopped vehicles in small intervals, as well as all previously stopped and departed vehicles. A matrix is drawn up as shown in Table C 2 and the vehicles remaining in each time slice calculated. The stopped delay D_S is thus determined by multiplying the sum of the remaining vehicles in each period by the time intervals. That is Equation A12:

$$D_S = d\sum_{j=1}^{n} RM_j \tag{A12}$$

where

d = the time interval

RM_j = the number of vehicles remaining in the queue at the end of the *jth* interval

n = the number of time intervals

This equation is strictly correct only if all stoppages and departures occur only at the start of each time interval, whereas in fact they will be spread over each interval. Therefore delay will tend to be overestimated in an interval in which stoppages exceed departures, and underestimated in the opposite case. Over a sufficient period, the errors tend to cancel each other.

For the data in Table C 2 this yields:

$$D_S = 10 \times (6 + 10 + 12 + 8 + 2) = 380 \text{ seconds}$$

Table C 2: Matrix of vehicle movements at a sample intersection

Vehicles	Number of vehicles in 10-second time intervals					
	0–10	11–20	21–30	31–40	41–50	51–60
Stopped	6	4	3	2	2	0
Leaving	0	0	1	6	8	2
Remaining in period (RM_j)	6	10	12	8	2	0

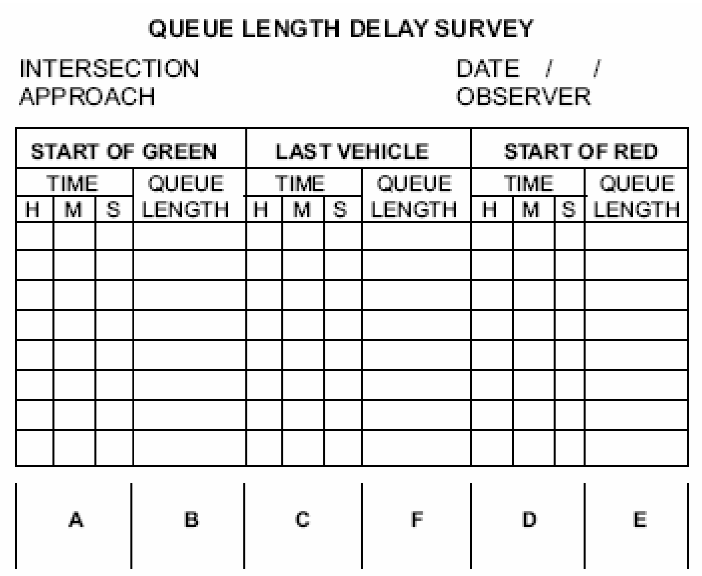
Typically, intervals of 10 to 15 seconds are used. At traffic signals, the time interval should not be a direct proportion of the cycle time.

A variation of this method can also estimate the time travelling across the approach area. All the vehicles in the approach are measured at each time slice, whether stopped or moving (Taylor et al. 2000).

Overall delay using QDELAY

The QDELAY technique requires the recording of queue lengths at different points in time during the signal cycle (Richardson & Graham 1982). Figure C 2 shows the form used to record the data in the field. At the start of the green phase, the time is recorded in column A, and the number of vehicles stopped in the queue is recorded in column B. At this point it is necessary to make a mental note of the last vehicle in the queue at the start of the green time. If the last vehicle crosses the stop-line before the next red phase starts, the time at which it crosses the stop line is recorded in column C. After that vehicle crosses the stop line, the number of vehicles crossing the stop line before the start of the red phase is recorded in column F. The time at which the signal changes back to red is then recorded in column D. Column E is left blank in this situation.

Figure C 2: Survey form for QDELAY


QUEUE LENGTH DELAY SURVEY

INTERSECTION DATE / /
APPROACH OBSERVER

START OF GREEN				LAST VEHICLE				START OF RED			
TIME			QUEUE	TIME			QUEUE	TIME			QUEUE
H	M	S	LENGTH	H	M	S	LENGTH	H	M	S	LENGTH

A B C F D E

If, however, the end-of-queue vehicle does not cross the stop line before the signal changes back to red, the time at which the light changes to red is recorded in column D. The number of vehicles in the queue at this point of time (including the last vehicle) is recorded in column E. Columns C and F are in this case left blank. This process is repeated for every cycle of the survey period.

These data can then be analysed to provide information on cycle settings, total flow, averages and standard deviations of the approach delay and stopped delay per vehicle, average numbers and standard deviations of effective stops and complete stops, and average and standard deviation of maximum stationary queue length per cycle. It cannot, however, distinguish between delays to vehicles making particular manoeuvres if the lanes are carrying mixed turns and through vehicles.

C.4 Data Analysis and Presentation of Results

The previous discussion has included descriptions of the methods of analysis used in travel-time and delay studies. Results of these studies may be presented in tabular, graphical, and diagrammatic form to show variations along the route being studied, together with the location, cause and length of delays, if available. Where large quantities of travel time or delay data are collected, electronic data processing methods for recording and subsequent analysis of data should be used.

Appendix D Origin-Destination Surveys

D.1 Types of Origin-Destination Surveys

Some commonly used techniques for gathering origin-destination data are:

- numberplate survey
- roadside interview technique
- postcard survey
- headlight survey
- windscreen sticker technique
- registration address technique.

The use of GPS technology for speed measurement has been mentioned in Appendix B.2.5. The same technology is applicable for identifying the travel path of a vehicle and hence the origin and destination of that trip. The technology at present is still expensive and its applications are mainly for commercial operations such as taxi dispatching and heavy vehicle or fleet monitoring. Its usage will increase as the cost of using a full GPS system decreases in the future (Stopher & Speisser 2011).

Electronic toll collection technology may also be adapted for origin-destination surveys. The increasing prevalence of electronic toll tags in the vehicle fleet provides opportunities for identifying the entry and exit of vehicles at points which could be used on survey cordons.

D.1.1 Numberplate Technique

This is the most common origin-destination survey technique in use. It involves stationing observers at selected points, in and around an area, to record the number plates of vehicles and the time at which each vehicle passes the observer. The route taken by a vehicle can then be found by matching the observations. This technique is best suited to traffic patterns characterised by a large number of origins and destinations, linked by a complex road network. This is the situation commonly found in an urban area. One disadvantage of this technique is the large amount of effort required in data coding and analysis. The use of laptop and palm PCs in traffic data collection and analysis has helped to mitigate this problem. The technique is discussed in further detail in Appendix D.2.

A recent development is the much improved image processing technology for the automatic recognition of number plates (Keilthy 2008). Once a vehicle is identified at various monitoring points, its origin and destination in a road network together with its travel time between any two monitoring points are available. ANPR is also used for law enforcement. It is used, for example, on CityLink in Melbourne and various toll roads in other cities. Cameras at a toll point in CityLink take a video image of the number plate of any vehicle without a valid electronic tag. These vehicles include account holders without a tag on that vehicle, day-pass holders and other vehicles that could receive a fine (Lay & Daley 2002). The technology is still expensive for general data collection and privacy laws can prevent agencies such as CityLink from making such data available to the public.

D.1.2 Roadside Interview Technique

This type of survey presents the best opportunity to gather comprehensive data on traffic movements, but is not often carried out due to the lack of authority; generally only police have the authority to require drivers to stop (other than at roadworks or a traffic incident). Vehicles are directed into a roadside station, and drivers are asked to respond to a preset questionnaire about the details of their trip.

The technique allows the collection of information not available from simple observation. For example, the exact origin and the ultimate destination of a trip can be found, together with information on trip purpose, frequency and details of the route taken. The interview should be short, simple and unambiguous to both the interviewer and interviewee. It should only seek information relevant to the purpose of the study.

If the interviews are carried out on traffic entering and leaving a cordon line around a study area, the flow making a through movement is actually the mean of the flows determined at the point of entry and the point of exit. The roadside interview technique is generally not suitable for use in urban areas where high traffic flows and restricted right-of-way makes it difficult to stop the sampled vehicles without serious disruption to other traffic.

It is necessary also to check the legality of asking drivers to stop even with the cooperation of the police for reasons other than the non-compliance with road rules.

D.1.3 Postcard Survey

This technique requests drivers to take and complete a self-administered questionnaire, and then post it back to the survey organisation. Reply rates of 30–50% are usually achieved with reply-paid post. A return of at least 20% is considered essential to a recommended maximum of five questions. The postcard survey technique does not provide as much information as a roadside interview, but is useful in situations where it is unsafe to delay drivers more than a minute and provides more information than direct observation.

D.1.4 Headlight Survey

The headlight survey technique works by asking drivers at a particular site to switch their headlights on, and keep them on for a set period or until asked to extinguish them. Observers at downstream sites record the distribution of vehicles with headlights lit. The technique can be applied in rural areas where there are limited trip origins. It traces the destinations of vehicles entering an area along a given road. The technique can only be used in daylight and requires high levels of public cooperation. Advantages of the technique include little traffic disruption and quick and easy analysis of results. The technique can only be applied in relatively simple networks. Due to the lack of authority in asking drivers to switch on their headlights, this technique is being used less frequently.

D.1.5 Windscreen Sticker Survey

This method allows for the differentiation between separate origins by using different coloured stickers. Traffic may have to be stopped briefly to issue stickers to drivers, or stickers can be placed on parked vehicles or handed out to patrons before they leave a particular site. The stickers are then collected at the survey cordon stations. The survey may also be a postcard questionnaire survey, where the ‘sticker’ is a small questionnaire card. The technique may also be used for studies of pedestrian origin-destination movements.

The method is easy to apply and data analysis is relatively simple. However, it requires advance publicity to obtain co-operation from the public, and appropriate signs and police assistance are necessary at the approaches to the survey points.

D.1.6 Registration Address Survey

This technique exploits information on car owners’ home addresses. Vehicle registration numbers are recorded, and then the registered address of each vehicle is checked from official records. A number of problems exist with this technique. It assumes that the registered address of each vehicle is the actual trip origin and/or destination. Vehicles registered at commercial addresses will be recorded as such regardless of whether they are usually garaged in residential areas.

D.1.7 Radio Frequency Identification (RFID) Tags

RFID is a generic term to identify systems (devices) which transmit their identity (serial number) wirelessly. RFID tags are used to track goods in supply chains, for parts management in just-in-time production processes and in some automatic toll collection systems. RFID tags could potentially be used to measure origin-destination freight/freight vehicle movements and freight vehicle travel times across a network, but this would require installation of RFID readers across the network.

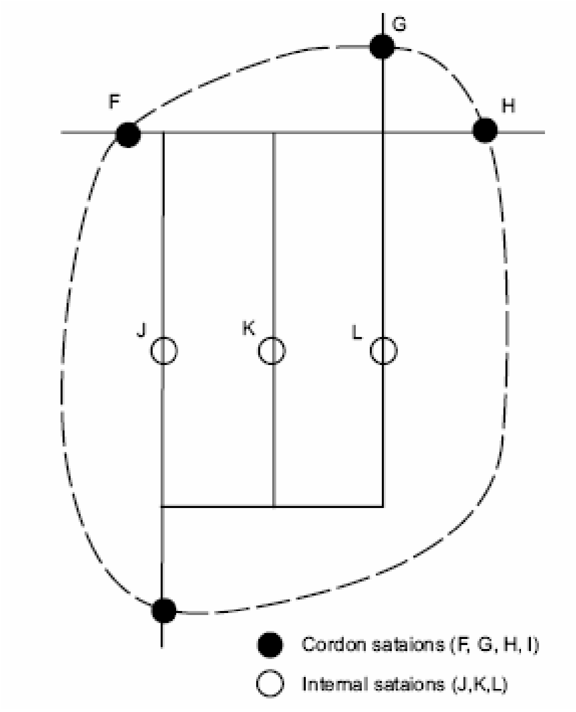
D.2 Conducting a Numberplate Survey

The survey process should begin by defining the purposes of the survey within the context of the traffic study to which it belongs, then setting the objectives to be met, and selecting a survey technique for the population to be sampled. The following considerations are needed once the numberplate technique is selected for an origin-destination survey.

D.2.1 Defining a Cordon

The cordon is the boundary around the study area for which trip movements are required. It may consist of physical boundaries (e.g. rivers, hills, parkland); major transport facilities (e.g. railways, freeways); jurisdictional boundaries (e.g. municipal boundaries); or arbitrary boundaries. The study purposes should point to the extent of the survey area. The survey objectives and constraints should be used to set the exact details of the cordon. Survey stations are then located on the cordon and inside the area to permit the observation of movements into and out of the area as well as route information. Figure D 1 differentiates internal stations from cordon stations.

Figure D 1: Stations on the cordon and inside the study area



Stations need to be located with care, both to minimise the number of stations while maintaining the required level of detail, and to remove possible ambiguities in the data (Figure D 2). For those stations located at junctions, observations should be made of each separate turning movement, as indicated in Figure D 3. Normally it is no more difficult or costly to record the flows by movement, and valuable additional data are then available.

Figure D 2: Setting cordon station locations

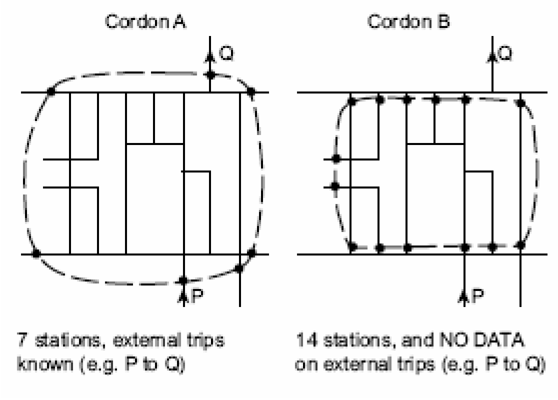
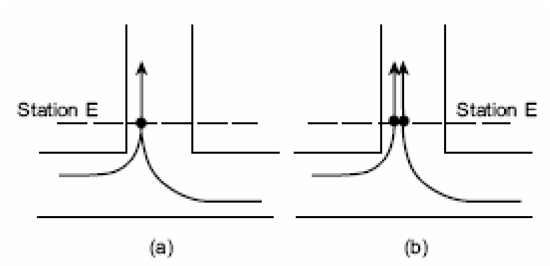


Figure D 3: Recording of separate turning movements at a station



D.2.2 Setting the Survey Time

The time of day for the survey should be chosen to cover the period when 'divertible' traffic will be observed (e.g. peak-period flows in urban areas, or busy shopping times at a shopping centre). The duration of the survey is also of concern. The observation must allow for 'start-up' and 'shut-down' periods that are discussed again in Appendix D.3.2.

D.2.3 Setting the Sample Rate

Sampling of numberplates is usually performed on the basis of a specified digit, such as the last digit of the numberplate. Sampling on the basis of the last digit ending in, say, a 4 will result in an **a priori** sampling rate of one in ten, while sampling all vehicles with plates ending in 1 or 2 would give a sampling rate of one in five. More accurate results will be achieved if the digits 1 or 4 are selected, as opposed to 6, 8 or 9 which are easily confused. Practical experience suggests that sampling on a single value of the last digit yields an approximate 10% sample, once traffic observations at the station exceed 2000 vehicles.

Care needs to be taken in the application of this rule. Some jurisdictions now have specialty numberplates that may not contain any numeric characters at all, and some general release numberplates actually end up with an alphabetic character, e.g. ABC123D.

A useful alternative to characters on a numberplate is the colour of vehicles, which can be assessed quickly. The sample rate is estimated by placing an automatic traffic counter at one or more survey locations. The proportion of different vehicle colours in the traffic stream seems to be relatively stable. For example, the white and cream colours were selected for the travel time survey described in Appendix C.1.2.

D.2.4 Selecting the Characters to be Recorded

It is common to record four characters rather than all the characters on a numberplate. The characters can be three digits and one letter, two digits and two letters, etc. Recording more characters requires more effort but results in less duplication of numberplates. A compromise exists between the effort required and the level of duplication. Recording less than the actual number of characters also addresses any privacy concerns which may arise.

D.2.5 Field Data Collection

The following data are usually recorded for each observation:

- number plate (to a specified number of characters)
- location (the observation station)
- direction of travel
- time of observation
- the type of vehicle.

Recording may be done with paper and a pencil, a cassette tape recorder or laptop/palm PC. The PC has a number of advantages such as in-built clocks to log arrival times automatically, and immediate computer-readable data, but shortcomings such as battery life, impact and water resistance, and initial cost should also be taken into account. Other techniques have their own favourable features. When all else fails, the humble survey form provides an intelligible and re-constructible storage medium. Cassette recordings are useful for heavy traffic flows, but are tedious to transcribe after the survey and may have their own special types of errors. Voice recognition technologies are now more readily available and can be used for automatically transcribing numberplate data.

D.3 Sources of Error in Origin-Destination Surveys

Errors in an origin-destination surveys can arise from two separate sources:

- in field-data collection, including misrecording of data observations and start-up and shut-down errors
- spurious matchings due to combination effects in the analysis of partial numberplates.

D.3.1 Errors in Data Recording

The following errors are commonly made in the collection of numberplate data, and the order is an indication of the relative frequency of occurrence:

- the number is missed completely
- one or more characters are misrecorded, or two characters are transposed
- the direction of movement is recorded incorrectly
- the type of vehicle is misrecorded.

The consequence of any of these errors will usually exclude the possibility of a successful match with another observation. Attention to methods which can reduce recording errors is, therefore, essential in good survey practice. A primary concern is the selection of observers to be used, and the stations they are allocated. Only experienced observers should be sent to sites where heavy traffic volumes are expected.

D.3.2 Start-up and Shut-down Errors

As every trip in the study area will involve a finite travel time, the first vehicles leaving the cordon at the start of the survey will be interpreted as local trips, even if they are in fact through trips. As they entered the study area before the start of the survey, there are no records of their entries. A similar problem occurs at the end of the survey. Two adjustment techniques are as follows:

- Discard all observations leaving the cordon a time period after the start of the survey and those entering the cordon a similar time period before the end of the survey. This time period is approximately the time taken for a vehicle to traverse the study area at a prevailing speed and is necessarily a function of the size and type of study area.
- Use manual matching of numberplates at the start of the survey to determine the time at which the first ‘wave’ of vehicles, entering the cordon at the start, passed each station. Observations before this wave can be discarded. A similar technique may be applied at the end of the survey.

D.3.3 Confused Characters

Depending on the recording technique, there will be pairs of characters that can be easily confused. For example, in written recordings S and 5 and 2 and Z and in audio recordings B and D and M and N can be confused along with others. The potential for this type of error is increased where one observer is calling numberplates to another observer who is writing the information on field sheets. Handwritten data can also be misread. During the analysis of data, an increase of matches could be achieved by replacing one of each of the problem characters with its partner.

D.3.4 Spurious Matchings

Errors due to spurious matchings may occur in the analysis of partial numberplates. For instance, three different numberplates ABC123, BAC123 and CAB123 could be observed. If analysis were performed on the basis of the last four characters, the partial plates would then be C123, C123 and B123. The first two observations would now match. This is termed a spurious match. If only the last three characters were used, the observations could all be interpreted as the same vehicle. With the tightening of privacy laws, it is more common for only partial numberplate data to be recorded, so some account needs to be taken of possible errors.

The compounding errors in origin-destination numberplate surveys are likely to lead to an underestimation of the number of through trips across the study area. Experience suggests that errors in observing and recording numberplate data are likely to outweigh errors due to spurious matching, certainly if at least four characters of the numberplates are observed. Errors can be further reduced if the selected characters contain more alphabetic than numeric characters, i.e. where numberplates are predominantly of the form ABD123, collection of the first four characters will yield less spurious matches than if the last four characters are recorded.

D.4 Data Analysis and Presentation of Results

D.4.1 Analysis of Numberplate Data

There are many computer programs available for analysing numberplate data. Each program will have its own internal logic, and it is possible that two different programs will generate different results. A two-pass procedure is useful for the analysis of numberplate data, with the ability to account for partial misrecordings of the data. The first pass looks for exact matchings between recording stations, creating an initial origin-destination matrix. Unmatched plates are then searched for ‘almost matched pairs’ (e.g. one character different, or two characters transposed). The estimated matches can then be added to the exact matches to create a total origin-destination matrix.

McPherson (1995) describes one such program that takes into consideration the following factors:

- vehicle speed
- vehicle route
- vehicle type
- time of travel
- size of study area
- average distance between observation stations.

The Roads and Maritime Services also has a software package that produces origin-destination trip matrices from numberplate data. The software produces ‘nominated’ paths between any two pairs of points, and origin-destination paths between one entry and any internal point or from where last sighted.

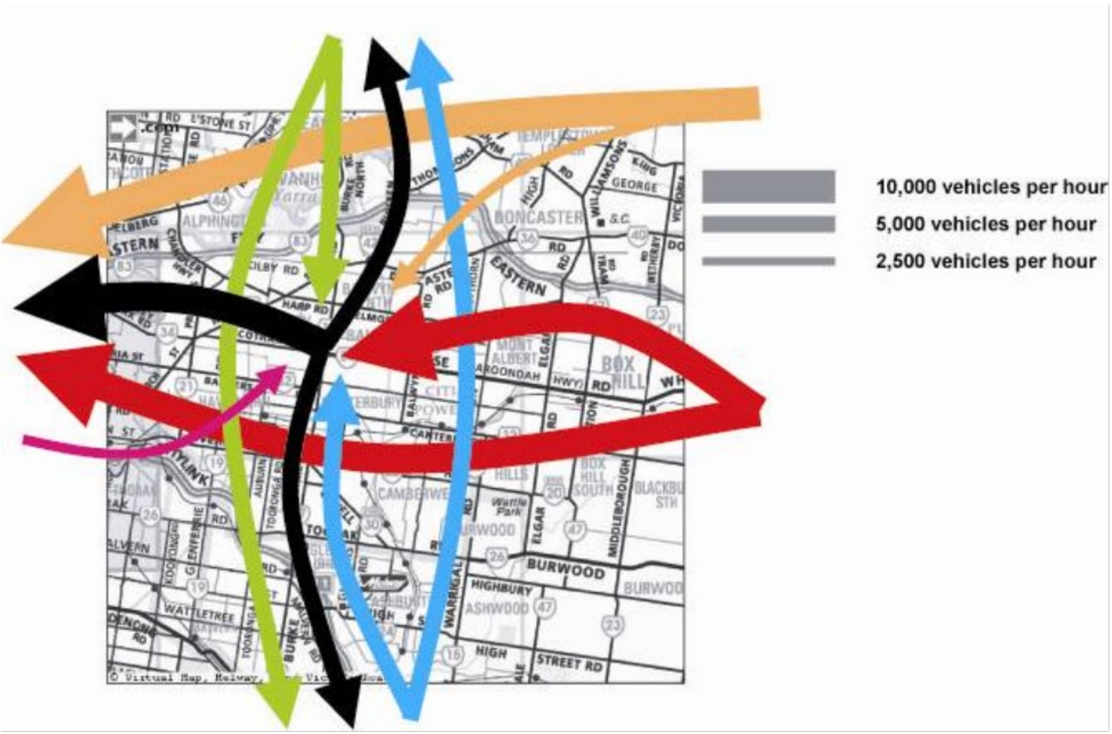
D.4.2 Presentation of Results

The main result from an origin-destination survey is usually a matrix showing the trips from each origin to every destination in the study. A convenient representation of the origin-destination matrix is the desire-line diagram. This diagram shows the travel desire between origin and destination zones represented by straight lines connecting the trip origins and destinations. The relative width of the lines indicates the relative amount of travel between the zones.

The detail with a desire line diagram can be lost if there are many origins and destinations. It is therefore useful if a computer program has the option to produce specific desire-line diagrams for trips from a particular origin, to a particular destination or between an origin-destination pair, as well as a full desire-line diagram. The generation of desire-line diagrams can benefit from a spatial database technology such as the geographical information system.

An example of a desire line diagram is shown in Figure D 4. For clarity, different colours are used to illustrate desire lines from different origins.

Figure D 4: A desire-line diagram showing trips in the vicinity of Kew Junction, Melbourne



Source: VicRoads (2008).

D.5 Summary

Origin-destination surveys provide useful information on travel patterns that can be used for traffic planning. They are important surveys for traffic engineers, but there are some significant difficulties in using them. These difficulties can only be overcome by the use of the best survey practices. This type of survey is both expensive and difficult to manage, and thus deserves careful planning, administration and data analysis, if its results are to be truly useful and valid.

New technologies such as GPS, GIS, automatic numberplate recognition (ANPR) and Bluetooth are now being used for fleet management, law enforcement and possibly electronic tolling. Origin-destination data can be a by-product of these operations. Privacy issues will need to be addressed before such data are readily available. Further information regarding new and alternative data sources can be found in Appendix K and Appendix L.

Appendix E Pedestrian and Bicycle Surveys

Initiatives in several jurisdictions (Austroads 2010b, Premier's Physical Activity Taskforce 2007, Ministry of Transport 2005) are increasing the profile of cycling and walking as well as striving to improve the provision of safe facilities. The planning and design of pedestrian and bicycle facilities is covered in *Parts 4, 5* and *6* of the *Guide to Traffic Management* (Austroads 2016a, Austroads 2017a, Austroads 2017b) and in *Part 4* of the *Guide to Road Design* (Austroads 2017d). This appendix outlines methods for collecting data on pedestrians and bicycles.

E.1 Pedestrian Survey Methods

E.1.1 Pedestrian Counts

The number of pedestrians passing a specific location per unit of time is the flow rate or volume. Information on volumes is required in almost all surveys (Appendix A). Volume data can be used to establish the importance of a route, the variations in flow, the distribution of traffic, the trends in road use and in the determination of other parameters. Traditionally, volume counts have been conducted manually. However, a number of emerging technologies can now be used to collect pedestrian data.

Pedestrian activity is not always simple travel from one point to another. It may also include the activity known as sojourning, involving the social aspects of walking such as wandering, standing, conversing, looking and resting. Traditional methods of measuring pedestrian flow activity are not always appropriate. Depending on the objectives for measuring pedestrian activity (for example, to assess the capacity and attractiveness of an urban space, as compared with the capacity of a planned walkway) it might be more appropriate to measure time spent in the space and monitor routes taken within the space, in addition to counting persons crossing cordon lines at several entries/exits.

Manual pedestrian counts

Manual counts consist of an observer recording the flow of pedestrians past a certain point for the required time period. The most common method of collecting volume data is by manual counts of the flow of pedestrians at a particular point in the traffic system. In the simplest form, the observer manually records the number of pedestrians for the time period. The demands of data collection can be reduced by using mechanical, electrical or computerised tally counters.

Manual pedestrian counts rely on good planning and skilled observers to ensure accurate and useful results. The number of observers will depend on the general level of traffic activity and the data recording task. For example, if classification of pedestrians (by demographic and/or direction of travel) is necessary, more observers will be required. Observation sites need to be chosen so that they provide a good view of the area but also provide protection from the weather and inquisitive people.

Pedestrian classification counts

The use of a pedestrian facility is partly related to the types of pedestrians using it. To determine the proportions of different users, classification counts are necessary. A classification count is an extended volume count in which different types of pedestrians are identified. A clear definition of pedestrian types is essential. For example, there is little point in recording the volume of pedestrians by gender if the objective of the study is to determine the volume of school children using a crossing. The adults could be put into a single group.

The greater precision required in manual classification counts means that more people are required to collect, decode and analyse the data. A manual classification count appears simple. Passing pedestrians are recorded for predetermined time periods using different parts of a survey form or by hand-held counters. In practice, this procedure gives rise to considerable scope for error. Pedestrians can be missed, double counted, incorrectly identified or entered in the wrong place. Even if counts are properly conducted and well supervised there is still possibility for error.

Distribution of arrivals and service-time measurement

The average pedestrian arrival rate can be determined from a known volume. The analyst may, however, be interested in the distribution of pedestrian arrivals at a particular point. This can be obtained by recording arrival times and forming a histogram from the data. Times can be obtained using stopwatches and coding forms or via a computer data logger. Video could also be used.

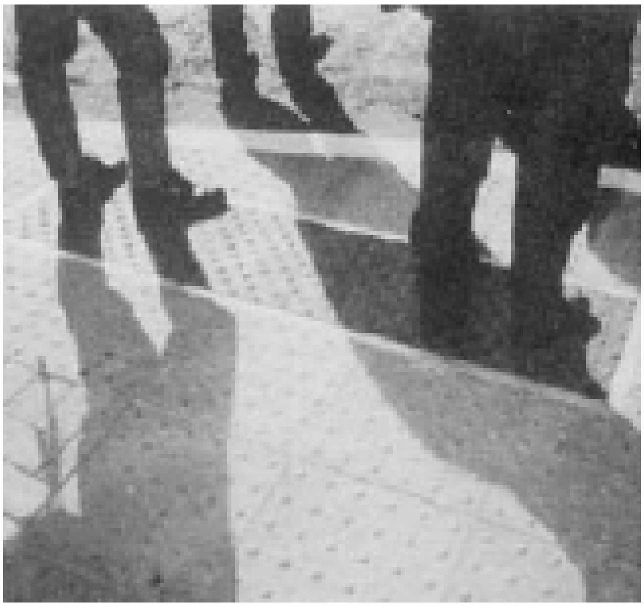
The rate at which pedestrians are serviced is also important. Queuing studies can be undertaken using the method described above for arrival distributions. It is important to note that the service time is strictly the time taken to service a customer. It is not the total time of the study divided by the number of customers, but rather the total time that the server is busy divided by the number of customers.

Automated pedestrian counts

Various pedestrian detection systems have been developed and they include the following:

- Tactile sensors – two types exist. The first consists of a tile with printed circuit-board inductors mounted in a flexible pad underneath. The pressure of a pedestrian standing on the tile distorts the shape of the inductors slightly, which is detected by a conventional loop detector. The second consists of a tile with a piezo-cable moulded into the nonslip covering. The tile can detect pedestrians, even when they are standing still because a pedestrian's muscle groups that are constantly active can be detected. Tactile sensors (Figure E 1) are suitable for pedestrian detection at traffic signals (e.g. puffin crossings).
- Above-ground sensors utilising microwave, infra-red, ultrasonic and laser detection – these systems can be used to count pedestrians passing a fixed point. A major difficulty is in distinguishing between closely positioned pedestrians.

Figure E 1: Tactile sensors



Video detection

The cost of video equipment has decreased substantially, making it an affordable option for traffic studies. Video can be used to collect many types of pedestrian parameters such as flow, volume, classification, distribution, and route and origin-destination data. Video has a number of advantages including:

- observations that do not disrupt the pattern of activities
- the ability to record many events simultaneously or in rapid succession
- a reduction in field staff requirements
- permanent and complete set of records
- both time and space dimensions can be recorded.

A prime advantage of video records of pedestrian activity is that the records can be analysed many times from different perspectives and for different purposes. This implies that careful consideration must be given to the positioning of cameras to ensure that relevant activities and locations are covered. This emphasises the basic need to have clear objectives for monitoring the pedestrian activity.

In the past, the major disadvantage of using video has been the long and tedious task of transcribing data following recording. Advances have now been made in the areas of digital recognition and image processing to allow automatic recording of data during traffic studies.

For example, Rouke and Bell (1994) describe software that can obtain information on pedestrian movements automatically using image processing. Of the two separate algorithms developed, one was designed to give a measure of the pedestrian density within a crowd scene and the second to count and determine the walking direction of individual pedestrians. The reported results suggest that the automatic measurements of crowd density and individual pedestrians are feasible, after some detailed calibration.

In addition, Akçelik and Turner (2001) developed a survey method for pedestrian crossing times at signals.

GPS tracking

The increasing availability, and decreasing cost, of tracking devices offers advantages in monitoring pedestrian activity. Using GPS technology to observe walking patterns in city centres, for example, offers new abilities for collecting data across a broad spectrum. It is possible to gather individual and collective data on whole trips (including interaction with public transport facilities), routes through the network and within defined areas, access points, active and inactive time, visited locations, and intensities of use of space, as well as average speeds and flow information. Collecting data using GPS tracking can provide greater insight into pedestrian behaviour and pedestrian movement, which can be used to help define interventions to improve the walkability of public spaces.

Bluetooth wireless technology can also be used to track pedestrian movements. Many pedestrians are also now Bluetooth enabled, carrying personal devices such as mobile phones and headsets. The use of alternative technologies is further discussed in Section 2.6 and Appendix K.

E.1.2 Pedestrian Audits

Pedestrian audits provide valuable information to engineers, urban planners and local councils on how public spaces or development plans rate from a pedestrian viewpoint (Daff & Cramphorn 1994). They are used to improve road safety, convenience and mobility, and personal safety of pedestrians. Pedestrian audits are ideally carried out at all stages of a major road scheme, in particular the feasibility and draft design stages to identify potential safety problems before the development is completed (Aylward & Valentine 1995).

A road safety audit is carried out by an auditor that walks the study area in a number of different conditions (e.g. day, night, wet), searching for potential safety deficiencies and reviewing existing facilities. The auditor needs to address issues relating to the most vulnerable pedestrians including the elderly, young, disabled and intoxicated. A safety audit checklist should include the consideration of visibility, vehicle speeds, presence of steps, paving types, levels of passive surveillance and length of walk signals. Additional information and identification of problems in the study area could be determined by conducting a questionnaire survey of pedestrians in the area (Appendix E.2.2).

E.2 Bicycle Survey Methods

E.2.1 Manual Bicycle Counts

Manual bicycle counts to determine bicycle flows, classifications of riders, distributions of arrivals and service times are similar to the processes described for manual pedestrian counts in Appendix E.1.

E.2.2 Questionnaire Surveys

Questionnaire surveys can provide useful information on route choice, origin-destination information, characteristics of cyclists, crash history and the adequacy of bicycling facilities. Questionnaires can be mail-back, self-administered or interviewer-administered. The mail-back questionnaire is useful when the respondent has little time to answer the questions. The response to such surveys, however, can be quite low (30%) and unless information on the characteristics of the non-respondents is known, the results could be misleading. Simple, readily understandable questions will provide the highest response rate. Self-administered questionnaires are completed by cyclists (or pedestrians) at the location where they are handed out. For this technique to be successful the respondent must be ‘captive’ and not pushed for time. As with the mail-back survey, the questions must be simple and easily understood. The on-site interview involves an interviewer asking the cyclist (or pedestrian) a series of questions, and recording the answers. Again, the respondent must not be pressed for time and the questions asked should be kept to a minimum. The advantages of this technique are an increased response rate and the ability to further explain difficult questions.

Another form of questionnaire is the household or workplace survey. These types of surveys can be used to collect considerable information on trip purpose, route and origin-destination and socio-economic characteristics.

An important point to keep in mind when preparing a questionnaire is the use of appropriate definitions. One problem area is the definition of a trip. A trip can be defined as a one-way movement of a person or vehicle between two points for a specific purpose, sometimes called a stop.

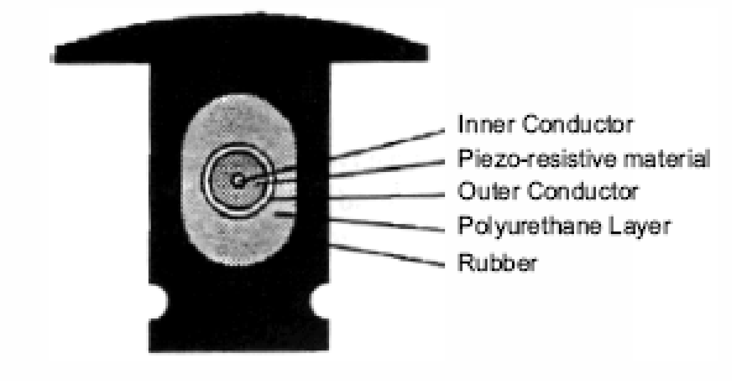
E.2.3 Bicycle Detection

Inductive loop detectors are commonly used to detect vehicles but can also be used to detect bicycles. The loops, which are buried just below the surface of the road or cycle way, record metallic objects passing over, due to a change in the inductance. Bicycles have a lower metal content than vehicles. Bicycle inductive loop detectors therefore need to be more sensitive to produce acceptable results. Various designs for bicycle inductive loop detectors have been proposed to try to optimise bicycle detection (Leschinski 1994).

Commercially available bicycle detectors are also available and were installed on the Sydney Harbour Bridge walkways using asymmetric loop detectors to detect cyclists by direction and ignore the passage of shopping trolleys.

Piezo-detectors can also be used to detect bicycles (Figure E 2). Piezo materials change electrical characteristics when subjected to mechanical deformation caused by pressure as discussed in Appendix A.1 and Appendix G.3. The deformation can cause a change in resistance (piezo-resistive) or the generation of a charge (piezoelectric). The piezo-resistive sensor can detect a bicycle at low to zero speeds, whilst the piezoelectric sensor is not effective at very low speeds.

Figure E 2: A typical piezo-resistive cable for bicycle detection



As with the detection of pedestrians, microwave, infra-red, ultrasonic and laser detection methods can also be used to detect bicycles. Again, these types of sensors may not provide the required accuracy due to difficulties in distinguishing between closely spaced bicycles. Figure E 3 shows the downloading of bicycle counts to a PC, using a tape-switch as a bicycle axle sensor.

Figure E 3: Downloading data from a bicycle counter

E.2.4 Video

Video recordings can be analysed to determine bicycle-flow rates, speeds and headways. The time stamp of the video including the frame number allows an accurate time recording. A technique described by Khan and Raksuntorn (2001) automatically determines bicycle-flow data and could greatly simplify the study of bicycle flow characteristics. The technique estimates bicycle location data by transforming screen coordinates of video frames to ground or roadway coordinates. The process is called rectification and enables automatic recognition of location and hence speed and acceleration data.

E.3 Travel-time and Delay Studies

Travel-time and delay information can be collected via a number of methods including questionnaires, video with time-stamp facilities and tagging.

In a tagging survey, pedestrians or cyclists entering a study area are given a card showing the time of arrival, classification and entry point. This information is updated as they pass other tagging points. The cards are collected and stamped with appropriate time and location information at the exit points. The method can yield a large amount of data on trip patterns and travel times. For detailed route information, the cards need to be marked at a number of locations. The delay involved in this marking could well influence the travel time measures. Cards may also be lost or discarded.

Virkler (1998) described a technique for determining pedestrian travel time. The technique is an adaptation of the floating car method (Appendix C). Observers are instructed to walk at the prevailing speed of nearby pedestrian traffic and, where practical, to pass roughly the same number of pedestrians as those that pass them. The observers record arrival times at each intersection queuing point and the times they step off and back on the kerb while crossing. Walking time, queuing time and average travel speed can be collected using this method.

Delay information is also of interest to traffic professionals. The delay to pedestrians or cyclists can either be a delay determined at a point in the traffic system or the delay over a route. The **route delay** can be determined by subtracting the unimpeded travel time along a route from the observed travel time measured in the study. The unimpeded travel time is the average travel time for a sample of unimpeded pedestrians or cyclists. **Point delay** can be determined by observing the number of pedestrians or cyclists stopping and measuring the length of time they are stopped. In situations where there is a large queue and the delay at different points in time is required, it may be necessary to employ a large number of observers or use techniques that do not involve detailed observation of individual movements. Appendix C provides further details on travel time and delay surveys, some of which can apply in pedestrian and bicycle studies.

E.4 Behaviour and Conflict Studies

The illegal use of traffic signals and other facilities and the incidence of queue jumping could influence the effectiveness of a facility. Information on the proportion of pedestrians and cyclists not observing regulations and making illegal manoeuvres can be collected using manual observation of the system. Video recording is also of help. The unusual nature of the movements recorded, however, precludes most automatic methods of data collection.

E.5 Data Analysis and Results

Austroads (2010b) provides a framework for the reporting of comparable state and territory cycling data, so that relevant comparisons can be made. The development of the guidelines will allow the measurement of progress towards a goal of increased cycling participation. The guidelines cover the reporting of bicycle ownership, infrastructure, usage, demographics and safety.

The guidelines recommended that data be obtained from existing data sources such as those from the Australian Bureau of Statistics (ABS) population census and household travel surveys of capital cities. Surveys should also be undertaken in stages to collect the appropriate data, which include the following in the first stage:

- bicycle ownership per capita
- bicycle network coverage (urban)
- cycling mode share
- cycling trip purposes
- proportion of population cycling
- cyclists age and gender
- cyclist injury rates – hospital reported
- cyclist crash rates – police reported.

E.6 Development of Survey Technologies

The development of new technologies has already automated a number of tasks in pedestrian and bicycle studies, as was described above. Advancements to existing technologies and the development of new techniques should see a further simplification of study techniques by decreasing the manual component even more. These include but not limit to:

- Global positioning system (GPS) – GPS receivers and loggers can currently be installed in vehicles to record route, speed and travel time information (Section 5). Hand-held units could provide similar information about pedestrians and cyclists (Stopher & Speisser 2011).
- Geographic information system (GIS) – in all the questionnaire and interview techniques discussed above there is an opportunity to ask respondents to mark their routes on a map of the locality. These routes can be entered into a GIS and the frequency of trips along a particular route can then be easily determined (Richardson et al. 1995).
- Video – further development of video capture and data processing technology should enable accurate automatic recording of pedestrian and bicycle flow, speed, congestion, route and origin-destination data.
- Smart card – non-contact smart-card technology already exists for payment of public transport fares (Luk & Yang 2001). This technology could be adapted to provide data on pedestrian and bicycle flows, in a study similar to the tagging survey described in Appendix E.3.

In recent years TMR has been actively collecting and using bicycle survey data from GPS tracking, traffic counts, intercept surveys and census journey-to-work data to conduct research and provide decision-making support on bicycle traffic and travel behaviour changes including route change and mode change (Langdon 2016).

Appendix F Noise, Fuel and Emission Surveys

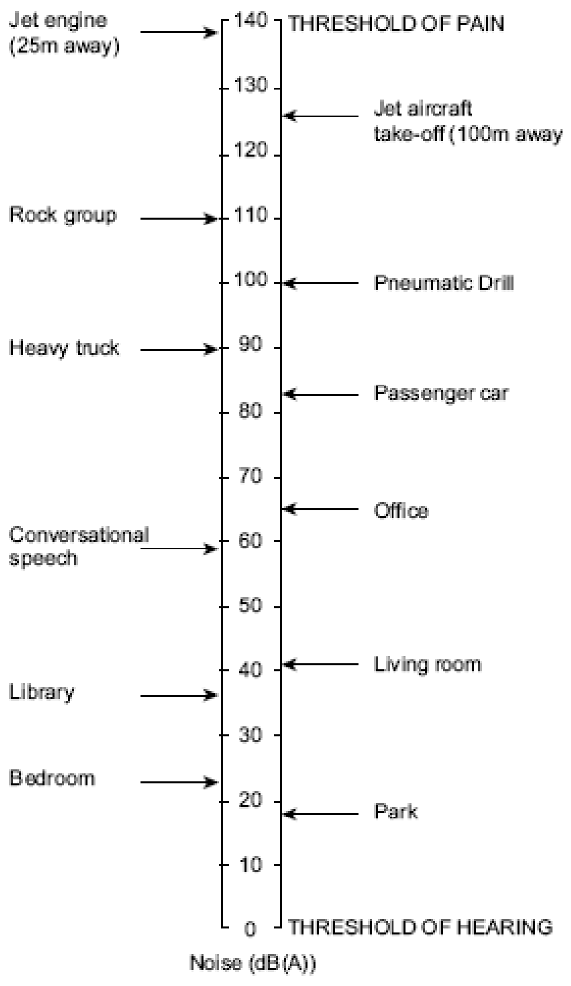
F.1 Traffic Noise

F.1.1 The Nature of Noise

Noise may be defined as unwanted sound. The biggest contributor to community noise levels in terms of number of people affected is road traffic. Besides degrading amenity, noise affects sleep and concentration. Exposure over the longer term to higher levels associated with heavily trafficked roads is increasingly recognised as affecting health. Very high levels of traffic noise can also be injurious, but such high levels are well above the normal levels of road traffic noise.

Sound is measured in decibels (dB). Decibel is a unit for measuring acoustic or electromagnetic energy (P_1) relative to a reference level (P_2), with the number of decibels calculated as 10 x $\log_{10}(P_1/P_2)$. A reduction of 10 dB, i.e. a reduction of P_1 (or P_1/P_2) to one-tenth of its former value, is usually and subjectively considered equivalent to halving the sound levels. Human ears respond to sound levels differently according to the frequency or pitch, variability and loudness of a noise. An A-weighting scale is built into sound measuring equipment to give a better representation of how ears respond to sound. The scale is known as the dB(A) scale. Figure F 1 shows typical environments corresponding to noise levels measured on the dB(A) scale of measurement.

Figure F 1: Typical environments for given noise levels on the db(A) scale



F.1.2 Noise Indices

Noise levels vary continuously at any given site and are usually expressed in terms of maximum noise levels, averages and statistical measures such as percentiles. The L_{10}(18h) noise level has been used extensively throughout Australia to measure noise pollution. It represents the sound level exceeded 10% of the time, measured over the 18-hour period 06:00 to 24:00 hours. L_{10}(18h) is also sometimes referred to as the average maximum noise level (Austroads 2002). The permitted value is commonly taken as 68 dB(A).

Increasingly, the L_{10}(18h) is being replaced by the equivalent continuous noise level L_{Aeq}. This is an energy average over a period of time, with typical periods being 1 h, 9 h (night) and 15 h (day). EEC member countries, USA, Canada and several other countries use the L_{Aeq} unit to measure noise.

Austroads (2002) reviewed a number of models used for predicting traffic noise levels. They included Nielsen and Nordic Council of Ministers (1996), Department of Transport (1988) and the US Federal Highway Administration (Menge et al. 2017). The most widely used model in Australia is the Calculation of Road Traffic Noise, or CORTN, developed by the Department of the Environment (1975) and updated by the Department of Transport (1988). The procedure was adapted for Australian conditions by Samuels and Saunders (1982).

F.1.3 Surveys of Noise Levels

Traffic and environmental regulatory bodies commonly perform measurements of noise emissions from individual vehicles. It is the area-wide noise levels, however, that are of more importance to traffic engineers and in particular the actual levels and annoyance reaction to traffic noise. When conducting noise measurements, in addition to the relevant traffic noise index, Austroads (2002) recommends the recording of the following physical properties at each site:

- traffic conditions, including traffic volume
- speed and composition
- road gradient and pavement type
- topographical detail
- weather conditions, including wind speed and direction, and rainfall.

In view of the random nature of noise and the intervention of many environmental factors, noise surveys must be conducted with great care and in accordance with the relevant standards. These include Australian Standard AS IEC 61672.1-2004 *Electroacoustics: Sound Level Meters*, AS 1055.1-1997 *Acoustics: Description and Measurement of Environmental Noise,* and AS 2702-1984 *Acoustics: Methods for the Measurement of Road Traffic Noise.*

In noise impact studies, sites should be chosen to cover the entire area expected to be affected by changes in traffic patterns, if technically and financially feasible. Where this is not possible, it may become necessary to use a few measurement sites to provide data for calibrating a general noise prediction model. The calibrated model should then be used to provide sound levels at representative sites.

In residential areas, measurements are usually taken 1 m from the house or building facade fronting the roadway. Observations should be taken when the road surface is dry and maximum wind speeds should not exceed 7.2 km/h at a height of 1.2 m.

A windshield should be used on the measurement microphone and no measurements should be undertaken when wind speeds exceed 36 km/h. Other issues that should be noted include:

- Measurement positions should, in general, not be so far back from the road that the length of visible road subtends an angle less than 150° symmetrically at the measurement position. They should not be closer than 5 m to the edge of the nearside running lane for traffic.
- Measurement positions should be located if possible at least 15 m away from any reflecting surface, other than the ground. This is known as a free-field environment.

- The ground cover in the intervening area between an observation position and the road affects noise measurement results. Sites should be chosen to maintain uniformity of ground cover.
- Microphones should be pointed towards the noise source, and a random incidence corrector should be fitted if sound-reflecting surfaces are nearby.

Road surface textures and grades also have some influence in noise measurements. Also, observation sites should not be close to intersections and side-roads – unless they are being studied specifically.

F.1.4 Surveys of Community Reactions

The degree of annoyance with noise pollution is usually ascertained from attitudinal surveys of residents. Another method used to indicate the effects of traffic noise in residential areas is the comparison of real-estate values between neighbouring subdivisions and streets subject to different levels of noise. If all other factors and features of residential property in different neighbouring streets are the same, then the difference in noise pollution may be reflected in the difference in property values. The comparison of real-estate values is one way to determine the externality cost due to noise.

F.2 Road Transport Fuels and Emissions

F.2.1 Fuel and Emission Types

Nearly all vehicles currently in use are powered by petroleum-based liquid fuels. The principal liquid fuel for road transport use is petrol, with some use also made of diesel fuel and liquid petroleum gas (LPG). Each of these fuels has its own properties.

Since 1986, new cars registered in Australia have been required to use unleaded petrol (ULP). The number of vehicles using ULP has risen to almost 70% of all registered vehicles in Australia, according to the Motor Vehicle Census of the Australian Bureau of Statistics (Catalogue No. 9309.0 latest issue in July 2016). ULP is designed to allow the use of catalytic converter systems in vehicle exhausts, to minimise emissions of the gaseous pollutants carbon monoxide (CO), various hydrocarbons (HC) and oxides of nitrogen (NO_X). Lead poisons the rare metal catalysts (e.g. platinum or palladium) used in the conversion of exhaust gases to harmless emissions, hence the need for lead-free fuel.

Super-grade (98 octane) leaded petrol results in the emission of particulate lead (Pb) in addition to CO, HC and NO_X. It is being phased out in Australia. A number of lead replacement fuels, which are kinder to the environment, are now available on the market for cars manufactured before 1986. Lead emissions are subsequently expected to drop further.

Diesel fuel is widely used for large vehicles and, to a limited extent, by passenger cars. Diesel engines have the advantage that they provide greater fuel efficiency for the same volume of fuel. Diesel fuel also offers significant reductions in CO emissions but oxides of sulphur (SO_X) are released, creating potential for acid rain. Low-sulphur diesel is now available and legislation is in place in some jurisdictions to limit sulphur in diesel emissions to 500 parts per million (ppm). Ultra- low sulphur diesel is being developed to reduce sulphur emissions further to 50 ppm.

LPG offers a cheap alternative to petrol, which can be readily used by most petrol-engined vehicles. LPG is regarded as cheap because it is an unavoidable by-product of the refining process and is subject to a different taxation regime. Gaseous emissions from LPG fuel are less than those from petrol (for CO, HC, and NO_X). There is also an absence of particulate lead. Fuel consumption is higher in LPG engines than in petrol engines.

Guidelines are in place to further reduce fuel emissions in *Legislative Instruments (Fuel Standards Instruments) Sunset-altering Declaration 2016*. Alternative fuels to petroleum-based ones include electricity, ethanol and hydrogen. These are slowly being developed and introduced to the market. Pilot programs with buses powered by ethanol has been conducted by private sectors. Car manufacturers in a number of countries have released hybrid vehicles running on electricity and petrol, with claims of emission reductions of up to 80%.

F.2.2 Air Pollution

Air pollution in urban areas consists of the primary emissions CO, HC, NO_X, SO_X, fine particulate lead (Pb), dust and soot, as well as secondary or derived pollution such as photochemical smog. Safe and annoying levels of various pollutants have been set down. Table F 1 lists the standard levels of pollutants that are measured by the Environment Protection Authority (EPA) of Victoria, based on either the National Environment Protection Measure (Air NEPM) or the State Environment Protection Policy (SEPP) standard levels.

Table F 1: Pollutants, standard levels and calculation averaging times

Pollutant	Standard level	Source	Averaging time	How calculated(1)
Ozone	100 ppb	Air NEPM	1 hour	A
Nitrogen dioxide	120 ppb	Air NEPM	1 hour	A
Sulfur dioxide	200 ppb	Air NEPM	1 hour	A
Carbon monoxide	9 ppm	Air NEPM	8 hour	B
Fine particulates (PM10)	50 $\mu g/m^3$	Air NEPM	24 hour	C
Visibility (airborne particle index)	2.35	SEPP	1 hour	A

1 *A: maximum of the preceding 24 one-hour averages; B: maximum of the proceeding 16 rolling eight-hour averages; C: current 24-hour average result.*

Note: ppb = parts per billion.

At high concentrations, carbon monoxide is a dangerous asphyxiant for mammals because it is absorbed readily into the bloodstream and competes with oxygen in the circulating blood. Lead is a cumulative poison. Hydrocarbons and oxides of nitrogen in the presence of sunlight under stable meteorological conditions may chemically react over a period of hours and result in the formation of photochemical smog. Ozone (O_3) is one of the many components of smog.

Computer models are now commonly used to forecast area-wide environmental impacts due to traffic management measures such as lower urban speed limits (Taylor 2000) or congestion charges (Mitchell et al. 2003). These models consider the interaction of route diversions, air quality and energy usage.

F.2.3 Surveys of Fuel Consumption and Emissions

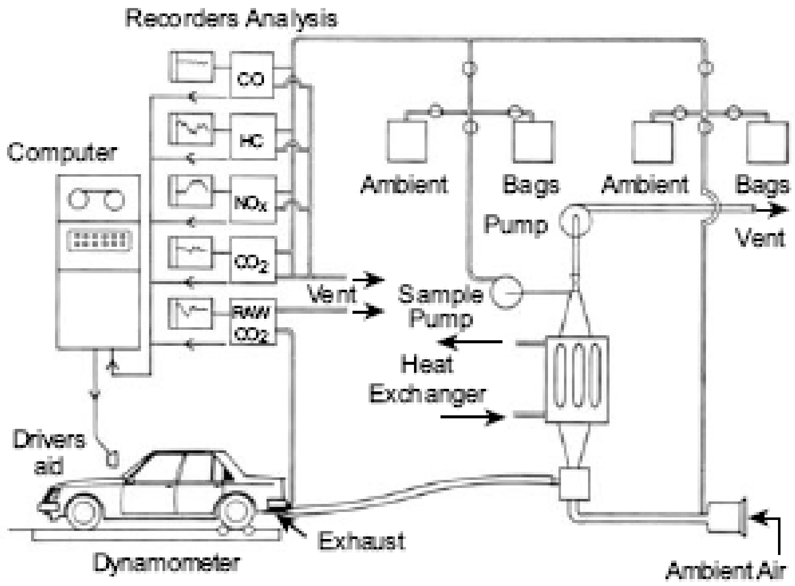
Numerous factors exist that affect fuel consumption. These include vehicle size, shape and mass, engine power and capacity, types of tyres, as well as factors independent of the car such as acceleration and deceleration, stopping frequency, traffic conditions, and road surface and curvature. Surveys of fuel consumption and emissions can be divided into three basic categories:

- individual vehicle-based surveys
- area-wide (system-wide) surveys
- attitudinal surveys of households or individuals.

It is possible to observe individual vehicle performance, fuel consumption, and emissions in a laboratory using a dynamometer. A dynamometer is a test bed on which a vehicle is fixed, with the driving wheels in contact with a system of flywheels that simulate the inertia of the vehicle. The wheels are also in contact with a power absorption unit which can be adjusted to reproduce different road conditions as speed is varied. The vehicle's acceleration, deceleration and speed performance can then be recorded directly. Measuring instruments are attached to the vehicle's engine/carburettor and exhaust to measure instantaneous fuel consumption and exhaust emissions respectively. A computer is attached to directly record the data.

Figure F 2 is a schematic representation of a dynamometer set up to record fuel consumption and exhaust emissions. A driver's aid indicates the desired speed-time curve for the test run, and provides feedback to the driver on his success in following the curve. Samples of exhaust gas are collected by the constant volume sampling (CVS) method. The CVS unit has a constant-flow pump system, with a capacity much greater than the maximum flow rate of the vehicle exhaust. The difference is made up by ambient air fed via the filters, mixing chamber and heat exchanger (Figure F 2), so that the concentration of exhaust gases in the dilute mixture is proportional to the exhaust flow rate. Continuous samples of the dilute mixture are extracted by a smaller constant-flow-rate device (the sampler pump). Samples are stored for analysis of CO, HC, NO_X and carbon dioxide (CO_2). Samples of filtered ambient air are also taken, so that corrections may be made for contamination from other sources.

Figure F 2: A vehicle dynamometer using constant volume sampling



Fuel consumption data are taken by one of two methods: fuel-flow meters or the carbon balance method. Fuel-flow meters are best suited in instrumented vehicles, where they can be installed for long periods of time. Dynamometer testing usually involves short duration tests on large numbers of vehicles, so that fuel meters are inconvenient. Dynamometer testing is described in further detail in *Australian Standard Emission Control for Light Vehicles*, Australian Design Rule 79. A major limitation of the dynamometer test is whether or not the results of the laboratory tests represent observed field data. The alternative experimental method is then to use an instrumented vehicle, driven in a traffic stream, to record these on-road data. Usually only fuel consumption can be recorded, as data logging of exhaust emissions requires some device to be placed over the exhaust system.

The extrapolation of individual fuel consumption and emission data to more aggregated levels in a traffic system is difficult. The characteristics of the individual test vehicle need to be related to all of the driver/vehicle combinations in a traffic stream to permit this aggregation. Experimental design methodology developed by the CSIRO (Johnston et al. 1982) may make the extrapolation possible. Chase cars can also provide some data. The chase car pursues a vehicle selected at random along a survey route, logging speed-time performance. The driving cycle can then be used on the dynamometer to yield laboratory estimates of fuel consumption and emissions.

Fuel consumption data over long periods can be collected using conventional traffic survey techniques such as interviews, questionnaires and diaries. These methods can provide data on fuel purchase and travel behaviour, travel distances and trip destinations. The quarterly Australian Bureau of Statistics (ABS) (2017) Survey of Motor Vehicles Usage (Catalogue no. 9208.0) employs the questionnaire method to estimate fuel consumption and other data.

Air pollution can also be measured on an area-wide basis. For example, the Environmental Protection Authority of Victoria (EPA) has various permanent recording stations set up across metropolitan Melbourne which provide hourly air quality data to a central computer for monitoring and analysis. The area-wide systems record absolute levels of air pollution from all sources, not just road traffic, providing continuous base data. To gather more detailed local data, mobile testing stations can be used. The CSIRO Division of Atmospheric Research has developed a portable laser-based device called lidar (for light detection and ranging) to measure vehicle emissions and other measurements. The lidar emits brief and intense beams of coherent light. The air pollutants, dust and water droplets in the atmosphere absorb or backscatter the light, or rotate the plane of the polarised beam. The intensity and polarisation of the backscattered light yields details of the composition of pollutants in the beam's narrow path.

F.3 Data Analysis and Presentation of Results

In the case of traffic noise surveys, NAASRA (1980) and Australian Standard AS 2702-1984 provide guidelines for the presentation of results. These include a description of the following details in the reporting of results:

- location sketch
- detailed site geometry
- microphone location
- weather conditions
- time of day
- traffic flows and composition
- speed limits
- instrumentation used
- traffic noise descriptors (e.g. L_{Aeq}) and sampling methods used to determine them.

There are no specific rules for data analysis and the presentation of results from fuel and emissions surveys because of the specialised nature of the different survey types. Each survey needs to be treated on its merits, using the principles for analysis and reporting cited in Section 1.

The EPA in Victoria measures pollutant levels on an hourly basis, and issues air quality bulletins. The air quality bulletins are expressed in terms of an air quality index:

$$\text{Index} = \frac{\text{Pollutant Concentration}}{\text{Pollutant Standard Level}} \times 100$$

The standard level of pollutant used is either the Air NEPM or SEPP standard, as already shown in Table F 1. The level of pollutants is assigned a category, based on the calculated index value for the relevant period. The categories are shown in Table F 2.

Table F 2: Environment Protection Authority of Victoria air quality categories

Category	Index range
Very good air quality	0–33
Good air quality	34–66
Fair air quality	67–99
Poor air quality	100–149
Very poor air quality	150 or greater

In summary, the systems planning process for traffic survey methods places a requirement on traffic engineers to investigate the impacts of traffic on other systems. Traffic surveys need to account for the environment and energy consumption, as these are areas in which traffic has significant impacts. The present methodology is still not completely satisfactory. It is apparent that noise, fuel and emission surveys need to be undertaken through a multi-disciplinary approach, to which the traffic engineer has much to contribute.

Appendix G **Vehicle Mass and Dimensions Surveys**

This section describes the methodologies employed by member jurisdictions for the measurement of vehicle mass, and surveys undertaken by the Australian Bureau of Statistics that are relevant to heavy vehicles.

G.1 Vehicle Dimensions

Notwithstanding the high cost and possible bias that may be involved, there will continue to be a need for data on vehicle dimensions for which it will be necessary to stop and measure a sample of the vehicles at a suitable site. The data collected usually involves vehicle dimensions such as those on which legal limits are set, e.g. overall length, width, height and other internal dimensions such as axle spacing and front and rear overhang, which may be important for determining vehicle mass regulations, vehicle off-tracking and turning circle. The latter is important for geometric design of intersections and parking and loading areas.

Automatic vehicle classifiers are now readily available to classify vehicles by length or axle configurations. Loop sensors on freeway and arterial roads, or virtual loops using video imaging in urban environments, have also enabled the real-time collection of classified counts. ITS can potentially provide useful continuing and permanent classified counts, although the quality of these data needs to be checked.

G.2 Vehicle Mass

Apart from the important aspect of monitoring and enforcing gross vehicle mass and axle load regulations to control excessive damage to roads and bridges, vehicle mass data are particularly important in bridge design and in pavement design and management. This is illustrated in the shaded area of Figure A 10 in Appendix A.5.1 indicating the requirement of axle spacing, axle load and load frequency (repetitions) for the structural design of bridges and pavements. In addition, vehicle mass data play an important part in economic analysis, transport planning and freight route planning.

Vehicle mass is often expressed in terms of the number of equivalent standard axle (ESA) loads. A standard axle load is treated in Australia as the load of 8.2 tonnes on a single axle using dual tyres. The number of equivalent standard axles (N_{ESA}) for a particular load and axle configuration is defined as the number of passes of the standard axle load which could cause the same damage on a pavement as a single pass of the load and axle configuration in question (Lay 2009).

To calculate the N_{ESA}, first assume that deflection is used as the criterion of damage. It is common to employ a fourth power law to relate the deflection and hence damage to the fourth power of the applied load. Let P_a be the load on the axle configuration in question, and P_{ESA} be the standard equivalent axle load (= 8.2 tonne). Then one pass of a load of P_a on the axle configuration in question produces the same deflection as N_{ESA} passes of the standard axle load. The equation is therefore:

$$N_{ESA} \; x \; (P_{ESA})^4 = 1 \; x \; (P_a)^4$$

or

$$N_{ESA} = (P_a \; / \; P_{ESA})^4$$

For example, the pavement deflection due to an 8.2 tonne load on a single axle, single tyre is 1.5 times the same load on a standard axle (dual tyre, single axle). The number of equivalent standard axles is therefore $(1.5)^4$ or 5.

G.3 Vehicle Mass Data Collection Systems

Many different types of low and high-speed weigh-in-motion (WIM) systems are in operation in Australia. Austroads (2010a) provides a review of technologies in use. Table G 1 summarises the attributes of representative sensor technologies. These attributes include:

- installation mode
- maximum number of lanes
- wheels weighed
- ambient temperature
- tolerances
- mass sensor life
- accuracy.

Table G 1: Weigh-in-motion systems used and available in Australia

Feature	Strain gauge	Bending plate	Piezoelectric cable			Capacitance pad
WIM system name	ARRB Culway	PAT DAW 100	ARRB Express-weigh (Kistler Lineas)	Mikros RAKTEL 8000 (Roadtrax BL)	TDC Hi-Trac 100 (Roadtrax BL)	Mikros Systems capacitance HSWIM system
Installation mode	Semi-permanent within pavement culvert installed	Semi-permanent flush mounted	Permanent flush mounted	Permanent flush mounted	Permanent flush mounted	Semi-permanent flush mounted
Max no. of lanes	1 – 4	4	2	8	8	8
Wheels weighed	All	All	All	All	All	All
Measurement of axle load up to (tonnes)	50	20	–	20	20	20
Ambient temperature (°C)	–10 to +70	–40 to +75	–50 to +80	–20 to +65	–20 to +65	–20 to +65
Sensor life span (years)	10	15	–	6	6	20
Accuracy(1)	GVM (±10% at 95%)	GVM (±10% at 95%)	GVM (±10 at 95%)	GVM (±10 at 95%)	GVM (±10 at 95%)	GVM ± 10% Not better than ± 300 kg
ASTM WIM type	Type I	Type I	Type II	Type I	Type I	Type I

1 *GVM (±10% at 95%) means that the accuracy of measuring GVM is ±10% for 95% of vehicles weighed. For Culway, a higher accuracy of ±7% is possible.*

Source: Austroads (2010a).

Stationary or low-speed WIM systems using load cells have long been used to monitor vehicle or axle loads at weighbridges for compliance with legislation on mass limits. The research in the last two decades has been on high-speed WIM systems.

Of the high-speed systems, temporary surface-mounted systems offer the advantage of portability but at a lower level of accuracy.

G.3.1 Flush-mounted Systems

The flush-mounted sensors used in Australia include the following:

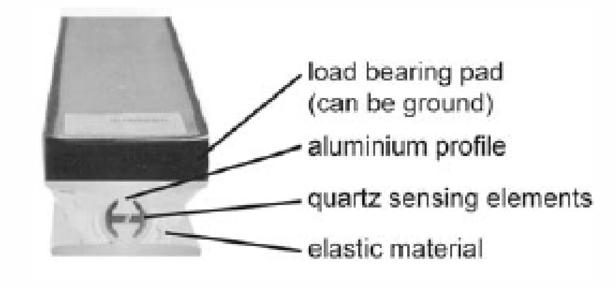
- bending plate, as used in the DAW-100 system from the PAT Group
- piezoelectric cable, e.g. the Kistler quartz sensor and used in the Express-weigh system from ARRB Group
- capacitance pad, e.g. Mikros capacitance mat system.

The bending plate and capacitance pad can be installed on site and thus offer some portability. The piezoelectric cable has to be permanently mounted in-ground.

The piezoelectric cable has been used as an axle detection sensor (Appendix A.1; Luk & Brown 1987), especially in the CULWAY system described below. It produces a measurable electrical response when pressure is applied to it. The magnitude of the electrical response is related to the pressure applied so that this device can be used also as a weighing device.

Figure G 1 illustrates the cross-section of a quartz piezoelectric WIM sensor. The quartz-sensing material has shown good reliability in yielding electric charges proportional to the applied force. It is practically not affected by temperature changes and is suitable for very slow-moving traffic.

Figure G 1: Quartz piezoelectric sensor



G.3.2 Strain Gauge/Transducer Systems and CULWAY

The primary example of the strain gauge/transducer system currently used in Australia is CULWAY. Earlier types included systems such as FASTWEIGH (Moses 1979) and AXWAY (Peters 1984) for bridges. CULWAY has been installed in many culvert sites in most states and territories (Figure G 2). It is a high-speed weigh-in-motion system and provides data on axle loads as well as vehicle classifications and speeds.

Figure G 2: A CULWAY data logger

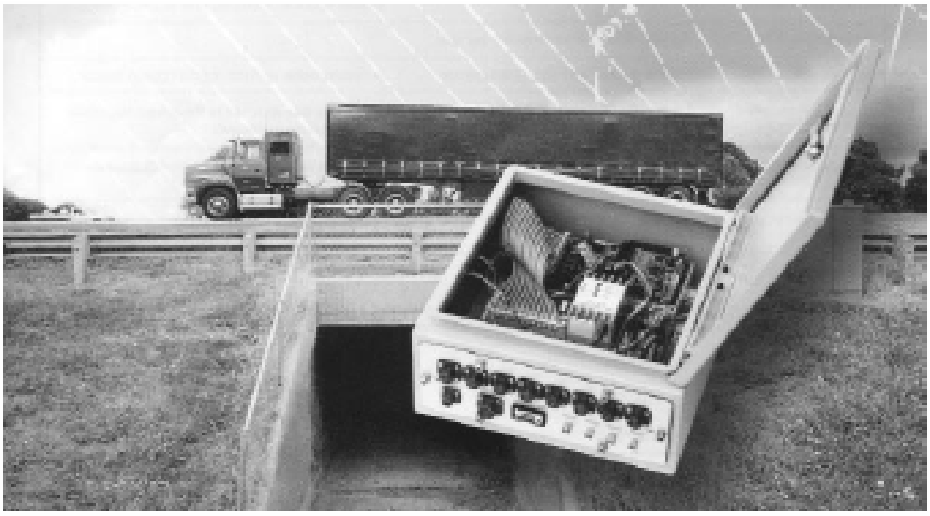
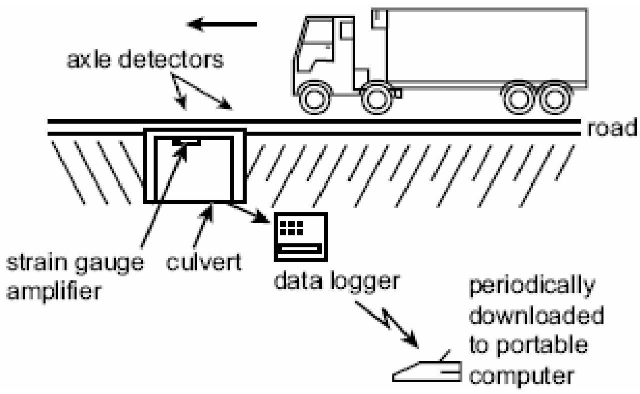


Figure G 3 shows a schematic layout of a CULWAY installation with the following main components:

- Strain sensors – these are attached to the roof section of an appropriately sized box culvert.
- Two axle detectors – piezoelectric cables are embedded in the road surface. These are placed at a set distance apart, one over the centre of the culvert in which the strain sensors are placed and the other at a known distance (e.g. 10 m is used in Western Australia) upstream. The axle detectors provide data for vehicle classification and speed and are also used to trigger the recording of strain data as each axle passes over the culvert.
- Data logger – this is either battery or mains powered. The logger stores vehicle count, strain measurements and other data on vehicles heavier than a minimum gross mass threshold. This threshold is generally 2.5 tonnes gross vehicle mass, although a 4.5 tonne threshold is sometimes used to reduce the amount of stored data and maintain accuracy in the critical higher mass range.
- PC – it is used to set up the data acquisition system and retrieve the stored data. It may also be used to carry out some processing and reporting functions.

Figure G 3: Elements of a CULWAY axle-weight measuring system



The CULWAY system generally operates unattended with data being periodically retrieved when the recording capacity of the unit is reaching its limit. This is generally about 10 000 vehicles or about two weeks' duration, depending on battery life and other factors. The data may also be retrieved by appropriate telemetry using data modems, avoiding the need for regular site visits. Some CULWAY systems now operate with solar power and are particularly useful at remote locations.

G.4 Tonnage Estimation

It is also possible to estimate the tonnage of a heavy vehicle from its axle configurations using historical WIM data, say, from CULWAY. The relationship would be site-dependent but is useful when WIM systems are not available. Some results reported in Pearson and Foley (2001) suggested the following relationships from data collected at 16 CULWAY sites in Victoria from 1995 to 1998 (Table A 1).

Table G 2: Mean axle group and gross vehicle mass

	Mean mass (tonne)				Trend (% growth per annum)
	1995	1996	1997	1998	
Class 9 articulated vehicle [0 00 000]					
Axle group mass 2	8.9	9.4	9.5	9.2	1.3%
Axle group mass 3	10.1	10.7	10.7	10.5	1.3%
Gross vehicle mass	23.5	25.3	25.3	24.9	1.8%
Class 10 B-double [0 00 000 000]					
Axle group mass 2	8.6	9.5	9.5	9.3	2.4%
Axle group mass 3	9.7	10.8	10.9	10.7	3.3%
Axle group mass 4	9.2	10.4	10.4	10.2	3.3%
Gross vehicle mass	32.0	35.9	36.0	35.6	3.4%

Source: Pearson and Foley (2001)

G.5 Compliance of Weigh-in-motion Technologies

CULWAY has been installed in many culvert sites in most states and territories. Due to the dominance of CULWAY for WIM applications, both the quality and reporting of data is quite uniform amongst different users. Table G 3 shows the accuracy requirements of WIM applications.

Table G 3: WIM accuracy requirements for various applications

Applications	WIM accuracy (gross vehicle mass for 95% of vehicles)
Economic analysis	< 20%
Transport studies	
Classification of vehicles	
Road and bridge management	< 15%
Overload warning	
Pre-selection for static weighing	
Road safety	
Enforcement	< 5%
Compliance	

WIM systems installed in Australia achieve an accuracy of better than 10%, although some systems specify the accuracy for only 66% of vehicles rather than 95% of vehicles. The single-lane or multi-lane CULWAY system provides an accuracy of 10% for 95% of vehicles. Note that there is currently no standard Australian specification or method to evaluate WIM systems.

The maximum potential of WIM technologies can only be realised with a mass sensor that promises a good accuracy and a low cost both for the sensor itself and its installation. CULWAY systems are cost-effective but can only be installed at sites where culverts are available.

High-speed WIM systems have been integrated with other systems for a range of applications. Two examples are as follows (Austroads 2010a):

- High-speed electronic mass unit (HSEMU) – this system automatically classifies, measures dimensions (length, height and width) and weighs each vehicle as it passes a HSEMU site at highway speed. In NSW, the unit is used as a screening system. Every heavy vehicle is diverted with a roadside advisory sign to a left-hand HSEMU lane and checked for compliance with pre-defined weight and dimension parameters. Vehicles that conform to the HSEMU check are diverted back to the traffic stream without stopping. Only those that fail the compliance tests are diverted to a static weighbridge for further checking.
- Automatic number plate recognition (ANPR) – this system consists of a number of video cameras to recognise the image of a heavy vehicle and identify the number plate using image processing techniques. It is used to detect heavy vehicles that violate one or more regulations such as speeding, being unregistered or a stolen vehicle, causing pollution if integrated with a pollutant sensor, or being overloaded if integrated with WIM systems.

G.6 Australian Bureau of Statistics Surveys

The monitoring of vehicle usage, especially that of heavy vehicles, is of national significance. The Australian Bureau of Statistics (ABS) has provided various types of data related to heavy vehicles. Note that vehicle classes used to describe vehicle types differ from the Austroads classifications described in Appendix A. Three relevant data sets are as follows:

- *Survey of Motor Vehicle Usage* (ABS Catalogue No. 9208.0; ABS 2017) – The publication provides the annual statistics for private and commercial vehicles registered for road use with a motor vehicle registration authority, by state or territory of registration or area of operation. It provides the number of vehicles, total and average kilometres travelled, driver characteristics, vehicle usage, fuel consumption and load carried. The publication also contains statistics on buses, including kilometres travelled by bus type and the main type of service. Data on capital city travel have also been available in recent years. The publication was first released in 1976.
- *Freight Movements Survey* (ABS Catalogue No. 9223.0; ABS 2015) – The publication provides statistics on tonnes and tonne-km of freight by Statistical Divisions, commodity type and transport mode. The commodity type is also classified by whether it was general freight, food, sand, stone and gravel, cereal grains, metalliferous ores and metal scrap and petroleum and petroleum products. The data were initially published on a quarterly basis from the June quarter in 1994 but the survey was suspended for a while. A different survey methodology for the road freight component was introduced in 2000. The issue in October 2015 covered all freight movements in the period 1 November 2013 to 31 October 2014.
- *Motor Vehicle Census* (ABS Catalogue No. 9309.0; ABS 2016) – Results are available yearly and was first issued in 1955. The 2011 Motor Vehicle Census (MVC) was conducted on 31 January 2011. It provides data on the number of vehicles on register for each state and territory. Vehicle classification is by type of vehicle (passenger, light commercial, rigid trucks, articulated trucks, non-freight carrying trucks, buses and motorcycles), year of manufacture, make, gross vehicle mass, fuel type and average age of vehicle.

G.7 Data Analysis and Results

In addition to the software associated with the data acquisition and retrieval units, a range of PC-based software has been developed for processing the large amount of raw data derived from CULWAY sites. Analysis programs have been developed at ARRB Group, and reporting programs are available from member jurisdictions.

The analysis software creates files containing summaries over various time periods from which graphical outputs including histograms of mass and vehicle speed can be generated. Figure G 4 and Figure G 5 show examples of WIM reports from the Department of Transport and Main Roads, Queensland.

Figure G 4: Example of a WIM report at Yandina, Sunshine Coast

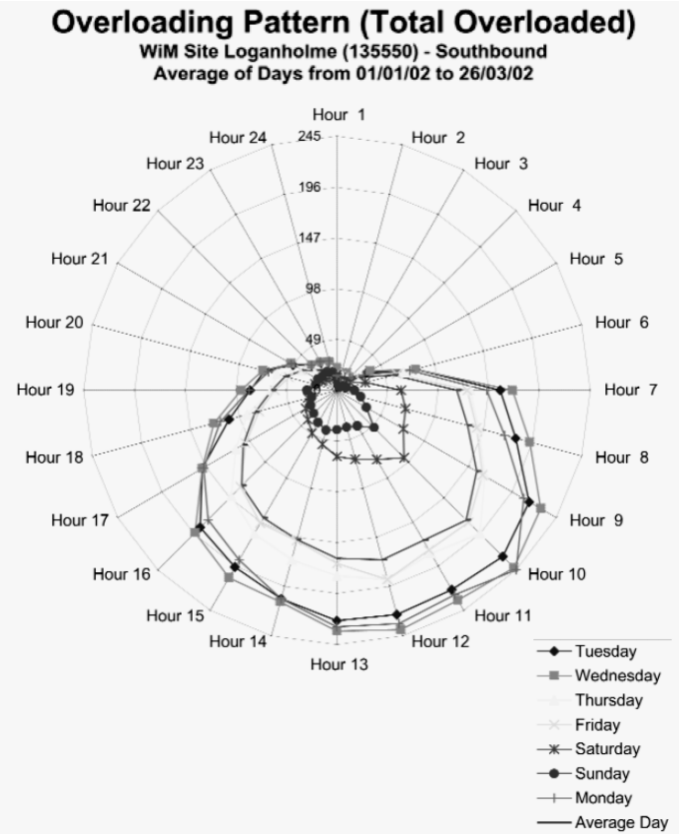
District	2 NORTH COAST HINTERLAND DISTRICT (MR)
Road Section	All Road Sections
Site	20739 WiM Site Yandina Bypass
Site Stream	All Site Streams
Traffic Class	All Vehicles
Date Range	Tuesday 01-OCT-2002 - Thursday 31-OCT-2002

Austroads Class	Description
	Bicycle
1	Short 2 Axle Vehicle
2	Short Vehicle Towing
3	2 Axle Truck or Bus
4	3 Axle Truck or Bus
5	4 Axle Truck
6	3 Axle Articulated
7	4 Axle Articulated
8	5 Axle Articulated
9	6 Axle Articulated
10	B Double
11	Double Road Train
12	Triple Road Train

	Equivalent Standard Axle					Weighed Axle Group Count			Percent of Axle Groups			Axle Group Mass (tonne)		
		Per Day												
Axle Group Class	Per Group	Unloaded	Loaded	Overloaded	Total	Report Total	Per Day	% of Total	Unloaded	Loaded	Over loaded	Mean	Standard Deviation	95%<
Single Steer	1.0	8.3	24.5	560.0	592.8	75013	605	11.6	93.8	2.6	3.7	2.1	n/a	2.6
Twin Steer	1.0	0.3	0.1	13.8	14.2	1785	14	0.3	96.7	0.5	2.8	3.4	n/a	1.4
Single Non Steer	0.2	1.6	15.0	40.5	57.1	37244	300	5.8	88.2	9.6	2.2	2.3	n/a	3.2
Tandem Non Steer	0.4	9.8	15.7	156.3	181.8	64065	517	9.9	86.6	10.4	3.0	5.5	n/a	7.0
Triaxle Non Steer	0.2	0.7	8.8	57.7	67.2	35672	288	5.5	46.9	50.4	2.8	7.1	n/a	8.7
Quad Non Steer	0.1	0.0	0.0	0.0	0.1	145	1	0.0	45.5	53.1	1.4	8.5	n/a	1.2
Totals	0.5	20.6	64.1	828.3	913.1	213924	1725	33.1	82.8	14.1	3.1	4.0		5.0

Source: Department of Transport and Main Roads, Queensland.

Figure G 5: An overloading report from a WIM site at Loganholme, Brisbane Parking Surveys



Source: Department of Transport and Main Roads, Queensland.

Appendix H **Parking Surveys**

H.1 Data Types

Many different procedures are available for collecting the variety of data types required to understand the parking process (Young 2003). Possible information required for parking analysis is:

- the number and location of existing parking spaces, both kerbside and off-street
- existing parking practices, including usage and available spaces, parking duration and illegal parking
- the need to impose or vary parking time limits or to install parking meters
- the adequacy of existing enforcement.

Information is required in respect of the *supply of* and the *demand for* parking facilities.

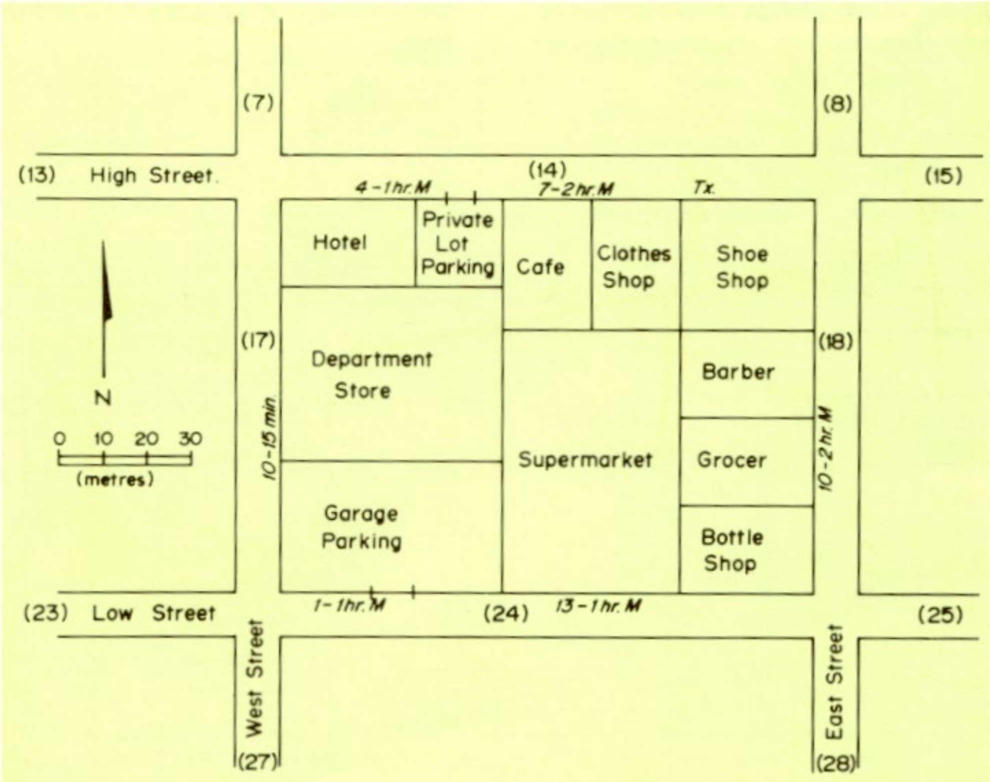
H.1.1 Parking Supply Data

A fundamental part of any parking survey is an inventory of the parking facilities and the possibilities for new development in the area of concern. Such an inventory should detail the type of parking and its location. Supply studies seek inventory information on:

- parking spaces
- type of parking
- method of operation of off-street facilities
- parking restrictions
- parking fees.

In compiling a record of street facilities, the data can first be entered on prepared sketch plans in the field. These sketch plans may be based on existing local maps or each street can be sketched onto a graph paper to the correct scale, with respect to length. The use of a suitable key enables the exact location of parking and parking restrictions to be marked on the map. The location of off-street parking could be marked on the map but the layout of these facilities is usually detailed separately. Normally the road network is coded in relation to road lengths with each block given a number key. This coding provides a basis for recording and analysing the data. An example of a typical inventory map is shown in Figure H 1.

Figure H 1: Typical inventory map



The inventory should be carried out by a two-person crew, equipped with a base map, kerbside and off-street inventory forms, and a 20 m tape. Portable computers installed with geographic information systems (GIS) are also readily available. Map data can be edited in the field to update parking inventory data.

The inventory may also involve the inspection of signs and markings, particularly those relating to the location of and access to parking facilities, or to parking restrictions. This inspection should include a record of deviations from standard signs and of poorly located, confusing, conflicting or misleading signs and kerb markings. Information on signs placed without authorisation should be included in the inventory.

H.1.2 Parking Demand Data

It is important to distinguish between revealed demand at the time of observation and latent demand for parking. Revealed demand is the observed use of the facility. Latent demand is a measure of the total desired use of the facility. Latent demand will only be exposed when supply exceeds demand. Most parking demand data collected in the field are revealed demand data, whereas future planning data require the latent demand.

The type of parking demand data needed includes:

- spatial distribution of parking demand and parking demand generators
- total number of parking events in study area over a study period
- parking duration and turnover
- trip purpose and destination
- trip origin.

The data collection procedures may involve interview surveys and observational surveys.

H.2 Interview Surveys

If the parking demand is to cover a large geographic area, and it is expected that changes in parking supply could cause substantial changes in the total number of parkers or their spatial distribution, data collection by interviews may be necessary. Four techniques commonly used are person interviews, reply paid questionnaires, home interview surveys and site-specific interview surveys.

H.2.1 Person Interview

This approach involves assigning an interviewer to a predetermined number of parking spaces. The interviewer records each parking incident and attempts to interview people parking in this area.

Questions asked in the interview relate to the following items:

- trip purpose
- final destination of trip
- origin of trip
- places visited
- duration of parking
- alternative parking locations considered
- frequency of parking in the study area.

Other details that can be collected by observation are:

- the vehicle registration number
- vehicle classification (car, taxi, truck, etc.)
- nature of parking (legal, kerbside, off-street, garage, etc.)
- time of arrival or departure.

The information obtained can be recorded onto an appropriate survey form (e.g. Figure H 2) and then transferred to a computer for further analysis. Alternatively, the answers can be coded directly into a hand-held computer. The latter approach reduces the number of times the data must be manipulated, reducing the chance for errors in coding.

The personal interview can be used to obtain data on people's attitude to various parking policies (e.g. changes in parking fees, parking restrictions). Care should be taken to keep the length of the interview to tolerable limits.

Interviews of on-street parkers can be carried out on arrival or departure. The departure interview has a number of advantages. Firstly, places visited can be reported more accurately since the parker has already visited them. Secondly, accurate information on the duration of stay can be available. Thirdly, the parker is less likely to be in a hurry and therefore more likely to complete the interview. The major disadvantage is that the interviewer has less time to catch parkers before they leave.

Non-response to the interview could introduce bias into the estimation of population statistics. The bias may result from the people who will not answer the questions since they are in a hurry, being unique in character and not typical of the people who complete the questions. Some attempt should always be made to determine the character (age, sex, occupation, etc.) of the people not completing the questionnaire. This may enable corrections to be made to the results of the responding sample. Sampling of parking vehicles can be carried out but care must be used not to introduce bias into the analysis.

In the case of off-street facilities, interviews can be carried out when the vehicle is entering or leaving the facility. During peak periods, sufficient interviewers are needed to avoid the unnecessary build-up of vehicles. Interviewing people as they leave has the advantage of avoiding vehicles having to queue onto the adjoining roads. If interviews are carried out upon entry, provision for queuing vehicles should be made to prevent disruption to traffic flows.

Figure H 2: Typical personal interview survey form

Facility Reference: __________________ Interviewer's Name: _________________

Vehicle Type	*Time In*	*Time Out*	*Trip Origin*	*Trip Destination*	*Trip Purpose (Circle one)*	*Parking*	
						Type	*No.*
					1. Shop 2. Work 3. Friend 4._____		
					1. Shop 2. Work 3. Friend 4._____		
					1. Shop 2. Work 3. Friend 4._____		
					1. Shop 2. Work 3. Friend 4._____		
					1. Shop 2. Work 3. Friend 4._____		

Date: __________________ Supervisor: ____________________

H.2.2 Reply-paid Questionnaire

When detailed information is not required, reply-paid questionnaires may be used. A typical example of the questions asked in a card-type questionnaire is shown in Figure H 3. These questionnaires can be inserted under the windscreen wipers of parked vehicles. Personnel costs for this method are smaller than for a personal interview since one person can cover a larger number of parked vehicles. Information on the parking location and arrival time of the vehicle can be obtained by marking or pre-coding the questionnaire. If pre-coding is used, the person distributing must note the time of distribution and location. This can be recorded in a logbook or hand-held computer.

The response rate of self-completion questionnaire surveys can be quite low, at about 30%. The response rate can be increased with the offer of a prize-draw or similar enticements, and subsequent reminder mails.

Figure H 3: Questions in a typical reply-paid questionnaire

Parking Survey

The rapid growth of population in Knox has reached a point where parking facilities are inadequate. Your answers to the following questions will aid in determining the need for parking facilities.

1. Where did your trip start before parking here?

   ____
   No. Street

   ____
   Suburb

2. Where did you go after parking?

   ____
   No. Street

   ____
   Suburb

3. What time did you park today?

   _______am _______pm

What time did you move your car from this parking space?

   _______am _______pm

4. What did you do while your car was parked here?
   (e.g. work, shop, visit friend etc.)

   ____

H.2.3 Home Interview Survey

The surveys described above do not measure the latent demand for parking, but rather the revealed demand. Many people wishing to visit the area may be turned away by the lack of parking facilities. Indications of latent demand could however be obtained by a home interview survey. The large cost associated with such an approach usually results in questions on parking being grouped with other questions on a large transport questionnaire. Sampling of the population may be a problem if the study is concentrating on improvements in a particular part of the urban area.

H.2.4 Site-specific Interview Survey

The home interview survey addresses the entire population of an urban area and is the only approach that can determine the latent demand in multi-use parking lots. However, some parking lots are only used by people working at specific locations. In such a case the total population of possible users can be defined. This population can then be used as a basis for determining the latent demand for the particular site. This approach will not provide an indication of the latent demand of visitors for parking.

H.3 Observational Surveys

Many parking studies are not concerned with information on the parker’s overall trip. In this case a simplified type of survey can be warranted. These can be either cordon counts or patrol-type surveys. Technologies are available for carrying out observational surveys by video with automatic reading facilities or by electronic tagging.

H.3.1 Use of Parking Revenue Collection Data

Computerised revenue collection systems are now common in the operation of parking facilities, and large amounts of data on entry, exit and parking duration times can easily be collected as a by-product. Bonsall (1991) describes a typical system for an off-street carpark as one that monitors the entry and exit of vehicles detected via inductive loops and/or barrier movements, and all financial transactions. Arrivals of special types of users such as season ticket holders can be logged separately. Data produced by a central computer could include (per hour intervals):

- entry flows
- exit flows
- accumulation
- fees paid – disaggregated by class of user
- numbers of tickets sold
- distributions of lengths of stay.

Data on the usage of unrestricted-entry off-street car parks such as pay-and-display facilities and on-street parking spaces have traditionally been harder to collect apart from levels of revenues. The introduction of electronic and computerised meters with various payment methods (e.g. stored value cards, credit cards, mobile phones) has meant that information such as durations of stay and distributions of usage can be recorded. The reading of meter data has also been revolutionised through use of portable enquiry terminals via infrared or optical scanners, and the networking of meters. Data from parking meters relates to transactions rather than actual usage, so correction factors may need to be used.

H.3.2 Cordon Counts

In this type of survey, the study area is surrounded by a closed cordon. Counting stations are established on all crossroads entering and leaving the cordon. At each station, a separate count is made of vehicles entering and leaving the area hourly or in shorter periods. The algebraic summation of entering and leaving traffic gives the accumulation of vehicles in the area. This accumulation represents the sum of vehicles parked and on the move in the study area. After identification and removal of the moving vehicles, a measure of the required parking facilities is obtained. The accumulation revealed can also act as a control on the usage of a car park.

Counting can be carried out either manually or by automatic counters. Manual methods are more expensive but may be required in special surveys, or to check and/or make corrections to the automatic counts. More detailed information can be obtained by recording the numberplates of the vehicles entering and exiting the cordon. The numberplate survey technique has already been discussed in previous sections.

The following information can be obtained from cordon counts:

- total number of parkers
- arrival and departure rates
- composition of population
- parking accumulation
- duration of parking.

H.3.3 Patrol Surveys

The patrol survey approach involves an observer patrolling along a predetermined route at fixed intervals. The location of parked vehicles and/or their registration numbers are recorded. Each trip around the section enables the accumulation for the parking facility to be estimated. Further, the number of times a vehicle is observed in the same parking place multiplied by the observation interval gives an indication of the parking duration.

The information that can be obtained using this approach is:

- total number of parkers
- arrival rate
- departure rate
- parking accumulation
- parking duration
- spatial distribution of parkers within the parking lot.

The study area must be divided into sections sufficiently small for each to be toured once every hour, half-hour or other small interval of time.

Patrolling by car enables longer sections to be considered in a given interval but both a driver and an observer are required. The observer would record the location and numberplates of parked vehicles, preferably using a portable computer to reduce analysis time and transcribing errors. It is important that the recording of the numberplates is done as inconspicuously as possible. Drivers of the cars being observed may change their normal habits if they are aware they are being observed.

Patrol surveys can be used to locate the spatial distribution of particular users. This requires the users to be identified by either a parking sticker or predetermined numberplates.

An obvious disadvantage of the patrol method is that many short-term parkers may be missed. The number missed depends on the interval of observation and the distribution of parking duration. Corrections can be made by carrying out a small survey of a particular location to find the general shape of the distribution. Adjustments can be made using this sample survey. Richardson (1974) also described a statistical method to determine these corrections.

H.4 Presentation and Analysis of Parking Data

The presentation and analysis of parking data has been facilitated by PC software such as spreadsheets, statistical/graphical analysis packages, high quality graphics functions and databases such as GIS. Typical results would include summaries of:

- variations in parking demand throughout the day and comparisons with available parking space on the kerb-side and in off-street areas
- variations in traffic flow and vehicle accumulation in the study area throughout the day
- parking duration and comparison with existing and proposed time limits
- time series, spatial and comparative analyses.

H.5 Summary on Parking Surveys

The introduction of electronic and computerised revenue collection equipment for parking systems has greatly increased the ease with which parking data can now be collected. Advanced technology has been developed for remotely monitoring space occupancies in a car park. Photoelectric and infra-red beams, or inductive loops, could be installed to monitor the occupancy of each parking space. Electronic chips energised by radio signals identify individual vehicles. When economical, these technologies will aid in the automatic collection of parking data, but privacy issues will again need to be addressed.

The ease of undertaking observational surveys has also increased with the introduction of video, voice recognition technologies, new parking and meters facilities such as park-and-ride, shared parking etc. and related ITS systems such as electronic parking guidance system, automatic vehicle identification, mobile telephony, ANPR, emerging enforcement technologies. The parking demand and data and supply data can be by-product of many of these new systems. Further discussion on new parking technologies can be found in Austroads (2017c).

Appendix I Traffic Generation Surveys

I.1 Data Collection

The techniques for collecting traffic generation data are of two broad types:

- techniques for observing traffic movements at a site
- interview or questionnaire surveys of people involved in a given activity in a given study area.

These techniques involve the application of other traffic survey methods, such as traffic counting (Appendix A), origin-destination surveys (Appendix D) and parking surveys (Appendix H), or interviewing (Richardson et al. 1995). Observed data on traffic flows, numbers and travel patterns of people, etc. may then be analysed to produce rates or absolute levels of traffic generation for a given land use activity.

Most surveys of traffic generation are conducted as observational surveys, particularly when the land use activity can be ascribed to a well-defined area or a site, such as a shopping centre or fast- food restaurant. When the land use activity is more dispersed, such as the traffic generation from a residential area, then the interview or questionnaire survey should provide more useful data.

I.2 Observational Surveys

Observational surveys of traffic generation involve the recording of traffic movements across a cordon line drawn around the site. The surveys involve the collection of traffic count data, either manually or automatically. These data indicate the accumulation of traffic at a site over time. Further data, such as the duration of stay in the survey area, may be collected using the methods for parking surveys. Observational methods have been used to collect data for the RTA (2001) and Institute of Transportation Engineers (2008) data sets. An example of their use for analysing traffic generated by shopping centres in an Australian city is described in Foley (1981).

A summary of the procedures for collecting traffic generation data is as follows (Institute of Transportation Engineers 2010b):

- Select traffic generators with similar development characteristics.
- Conduct automatic counts for at least a three-day period. The counts should be directional in 15-minute time periods, and the count locations should avoid through traffic and double counting.
- If peak traffic hours are not known, choose a typical week of the year to provide the weekday and weekend peak hours. If the development exhibits large seasonal variations, 'design days' should be selected, reflecting the 30th highest hour.
- If only peak-period counts are needed, conduct manual counts on one or more typical weekdays. Vehicle classification and occupancy data should be collected where appropriate. In some cases, weekend counts may be necessary to cover developments with peak activity on Saturdays or Sundays, e.g. shopping centres, markets and museums.

Observed behaviour, particularly that specific to one transport mode (such as private vehicles) and a particular site (such as the off-street car park at a shopping centre), provides a measure of revealed demand. Thus a count of vehicles entering or leaving the shopping centre indicates the actual use made of the facility. It does not indicate the overall demand to use the facility as the observational survey will provide no information on the latent demand (Appendix H).

I.3 Interview and Questionnaire Surveys

If there is reason to believe that a development has unique traffic generation characteristics, interview or questionnaire surveys should be used to determine the average weekday person trip ends by transport mode, and the number of trips produced by and attracted to the site. Information concerning the survey generator should be obtained through interviews with the site owner or manager, telephone conversations and mail-out/mail-back questionnaire.

The survey should obtain as many variables as possible to determine those closely correlated to traffic generation. Some essential variables are (Institute of Transportation Engineers 2008 and 2010b):

- current number of employees
- gross, net or leasable floor areas
- number of occupied rooms and dwelling units
- population and acreage of the development
- hours of site operation
- work shifts
- available public transport.

Interview and questionnaire surveys should theoretically provide better information than from observations of vehicle movements because they can provide a better understanding of travel demand. These surveys are, however, more expensive and difficult to design and administer than the observational surveys.

I.4 Summary of Survey Requirements

The general principles for traffic survey design and management are of crucial importance in traffic generation studies. The following requirements should be noted:

- Exact definitions must be provided for the traffic variables to be measured. For example, in a shopping centre survey, is it all people arriving at the centre, all vehicles arriving, or those vehicles that park there?
- Decisions must be made as to whether the measured generation includes all vehicles, or just private vehicles. For example, should service and goods vehicles be included in, say, a shopping centre or residential area survey?
- Area-wide surveys need to account for through traffic movements separately. Through traffic would be defined as that part of the traffic not stopping in the survey area.
- The possibility of internal trip movements, i.e. trips not crossing the external cordon, must be considered in area-wide studies. The larger the survey area, the more likely are such trip movements. Cordon-line observations will miss these movements.
- Precise definition of traffic generation rates is needed, e.g. the measurement of trip ends (the sum of trip productions and attractions) is preferred to a measurement of trips per se. This is particularly important when examining time profiles of traffic activity.
- Collection of long-term data is recommended to provide information on the variations in traffic generation over days of the week and weeks of the year.
- As traffic generation is often related to measures of activity, such as floor area, care is needed to ensure consistent measurements and definitions are used. There are several alternatives for area, e.g. gross area, net floor space, leasable floor space.

I.5 Forecasting Traffic Impacts

Computer models are often used to forecast the traffic generation impacts on a road system due to new developments such as a shopping centre or other land use changes in general.

The computer program SIDRA Intersection (Akçelik & Associates 2011) is a tool often employed in traffic impact studies. It is suitable for the evaluation of alternative intersection designs in terms of capacity, LOS and other performance indicators such as delay, queue length, fuel consumption and pollutant emissions. The software is particularly useful as an evaluation tool to investigate alternative traffic growth scenarios in traffic impact studies. Users supply traffic counts, the geometric details of the intersection and the types of intersection control, e.g. signal, give-way signs, roundabout, and the program analyses capacity and performance of the intersection lane-by-lane.

A model with traffic assignment capability is necessary to predict changes in the flow pattern of a study area. The use and application of microsimulation traffic models in assessing changes in network traffic patterns is reviewed in Austroads (2006). See also the *Guide to Traffic Management Part 8* for a summary of their use in local area traffic management studies (Austroads 2016e).

Appendix J Error in Seasonal Adjustment Factor

Suppose counts are available from three permanent stations which have similar AADTs and exhibit similar patterns.

Calculate ADT for each month for the three stations and then 12 average seasonal adjustment factors to convert each ADT to AADT. Select a random, seven-day count in each month for each station and calculate the ADT. Apply the corresponding seasonal adjustment factor for the particular month to obtain an estimated AADT, and compare each with the actual AADT for the particular station.

Calculate the number of estimates that are within the nominated accuracy. This proportion of estimates will represent the confidence limit corresponding to the accuracy. A typical tabulation is shown in Table J 1.

Table J 1: Typical values for the estimation of errors in seasonal adjustment factor

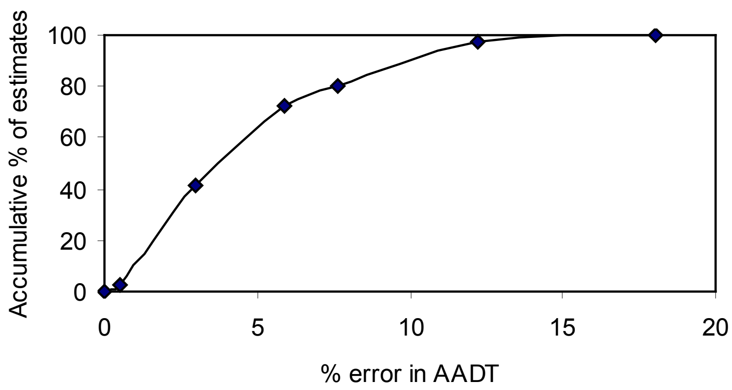
Relative % error in estimated AADT	Number of estimates	Accumulated % of estimates
≤ 1.0	1	2.8
1.1–5.0	14	41.7
5.0–6.8	11	72.2
6.8–8.5	3	80.6
8.5–16.0	6	97.2
> 16.0	1	100.0
Sum	36	–

The data from Table J 1 is plotted in Figure J 1, from which the errors at different confidence limits can be estimated. For example, at the 75% confidence level, the % error due to the seasonal adjustment factor is 7.4%.

In the example given in Appendix A.4.3, the estimated AADT is 7051 vehicles from the short-term station. Applying this error to the estimated AADT gives:

$$\text{Absolute error} = \frac{7.4}{100} \times (7051) = 522 \text{ vehicles}$$

Figure J 1: Determination of errors due to seasonal adjustment factor



The absolute error from the count duration is 818 vehicles (Appendix A.4.3). This error can be combined with the above error from the seasonal adjustment factor as follows:

$$\text{Combined error} = \sqrt{818^2 + 522^2} = 970 \text{ vehicles}$$

$$\text{Combined \% error} = \frac{970}{7051} \times 100 = 13.8\%$$

Appendix K Emerging and Alternative Traffic Data Collection Technologies

As discussed in Section 2.5 and Appendix A to Appendix I, conventional traffic surveys usually use stationary, fixed-point measurement devices distributed at selected locations across the road network (the exceptions being mobile floating car surveys, which have traditionally been used to measure route-level traffic speeds). New and alternative traffic data sources including Bluetooth technologies, GPS-based probes, cellular mobile phone and more, are becoming increasingly available in providing more data sources for traffic studies. The appendix provides a brief discussion on those technologies.

K.1 Bluetooth

Bluetooth applications in transport take advantage of the unique device media access control (MAC) address. The Bluetooth media access control scanner (BMS) scans the media access control identifier (MAC-ID) of the discoverable Bluetooth devices (BT) within its communication range, which is generally around 100 m in radius and depends on factors such as BMS antennae characteristics. Most of the portable electronic devices such as mobile phones, car navigation systems, and headphones are equipped with BT and its usage is increasing. Installing time-synchronised BMSs on the road network has the potential to provide live reporting of the transportation of BT devices over the road network. Assuming the devices are transported by the vehicles, individual vehicle travel patterns can be easily obtained (Bhaskar et al 2013*)*.

Bluetooth data can provide estimates of travel time and speed between a pair of BMS device sites, and with a network of Bluetooth measuring sites spread across the network, it could be used to derive matrices of origin–destination (OD) travel, and information about travel routes (TraffiCast 2016).

Use of Bluetooth and WiFi sensors to construct local (point-to-point) trip and travel time matrices is common in the USA and Europe. In Australia, Bluetooth devices (BMS) have been installed along arterials and freeways in South Australia, Queensland, Victoria and Western Australia to supply supplementary information for traffic operators. BMS can be installed in a traffic controller cabinet or other ITS cabinet at the roadside. Blogg et al. (2010) reported on MAC O-D trials funded by TMR and found that the Bluetooth MAC data compared favourably to both video and automated number plate recognition (ANPR) O-D data. Bhaskar et al (2013) analysed Bluetooth data from both motorway and arterial networks from Brisbane and reported that BMS has the potential to provide an excellent overview of the traffic profiles over the BMS networks. It can provide a snapshot of the daily traffic conditions where congested and uncongested conditions can be clearly identified.

Benefits of using Bluetooth technology and data are:

- It has the capability to capture long-term continuous traffic data (high resolution and frequent intervals) with little to no interaction with subjects.
- Setting up and maintenance cost is low and it is a non-intrusive technology that utilises wireless communication protocols.
- It has demonstrated good accuracy in travel time measures and OD information.

Limitations of current Bluetooth applications include:

- limited sample size on low-volume roads
- still relies on roadside equipment (portable or permanent)
- limited applicability at present to measuring total traffic volumes, as they will not be present or captured in all vehicles and so penetration rates (or sample size) have to be estimated
- historic data may or may not be available depending on the time the technology was adopted.
- potential sample bias due to heavy reliance on fleet vehicles or toward drivers that are more likely to have Bluetooth-enabled devices (TRB 2014).

Comprehensive specifications including the following parameters should be investigated before the use of Bluetooth data:

- data source type, e.g. from what type of vehicles/road users, at what road conditions
- spatial and temporal resolution
- link/segmentation information
- latency and sample size estimation
- data formats and contents
- available statistics from the data supplier
- data access methods/system interface/query criteria
- business models
- privacy and legal feasibility depending on the use case.

K.2 Mobile Phone Tracking

Mobile phones and the associated towers transmit various types of radio signals in order to carry voice and data. The signals (and their intensities) can be used to locate a mobile phone in time and space. The position of a mobile or cell phone is known from the cell and antenna sector allocated to the phone. The phone location within a cell can be identified by analysing its distance from three or more nearby mobile phone towers, i.e. by reiteration similar to global positioning using satellites. Travel times between two locations can be estimated by processing mobile phone location data and the times when the locations are read (time stamps).

A relatively accurate average travel time between locations can be calculated in real time without identifying individual mobile phones. Steenbruggen et al. (2013) commented that accuracy of mobile phone positioning is generally *lower* than that derived from GPS technology.

K.3 Probes

Probe data means travel speed or travel-time data that have been derived from persons or vehicles that carry devices that record or transmit their location such as those with the global positioning system (GPS). These probe vehicles include taxis, commercial vehicles and some private vehicles that are equipped with the GPS system, and communications systems that relay their speed and location to a back office for processing. It may also include personal electronic devices such as smartphones (Espada, Salt & Li 2014). Probe data services are provided by a few private companies in Australia and New Zealand including HERE (www.here.com), TomTom (www.tomtom.com), Intelematics (www.sunatraffic.com.au) and Google (maps.google.com).

Probe data sets can provide travel-time information historically and in real time. Probe data can also be used to generate OD information. This has significant potential application for road operators. Road agencies in Australia and New Zealand such as MRWA, Roads and Maritime Services, VicRoads, TMR and NZTA have investigated the suitability of utilising probe data (and combined with other data sources) for network operations planning, identification of congestion hot spots, assessing LOS and evaluating project performance.

The benefits of using probe data for traffic studies and analysis are:

- It allows for easier acquisition of long-term continuous traffic data (high resolution and frequent intervals) with little to no interaction with subjects.
- It is a low-cost solution without relying on the presence of sensors, is non-intrusive and does not interrupt surveyed subjects.
- It provides high temporal and spatial resolution with good coverage on high-volume roads (both freeways and arterials); the resolution and coverage are getting better with time. It is possible to use probe speed to replace more expensive floating car surveys.

The limitations of current probe data sources include:

- Limited sample size on low-volume roads.
- It generally requires an interface for the users to obtain traffic data from probe suppliers, and the effort to compile raw data could vary between different suppliers.
- It currently does not provide an accurate indication of traffic volumes, and sample sizes could vary with different types of roads and time of day.
- Licence restrictions from commercial suppliers need to be managed properly.

Comprehensive data specifications including the following parameters should be investigated before the use of probe data:

- data source type, e.g. from what type of vehicles, at what road conditions
- spatial and temporal resolution
- link/segmentation information
- latency and sample size estimation
- data formats and contents
- available statistics from the data supplier
- data access methods/system interface/query criteria
- business models
- privacy and legal feasibility depending on the use case.

Smart phones are very versatile sensing platforms given that most contain a GPS (or other GNSS receiver) and many are also equipped with accelerometers, gyroscopic sensors and a compass. Coupled together, these sensors can be used to provide not only the position of the phone and the mode by which it is travelling, but also road network and traffic conditions and OD travel patterns. It is worth noting that development of smart phones has also led to a proliferation of tracking apps. Practitioners should note that crowd-sourced data may show inherent self-selection user biases and may not be representative of all users. However, crowd-sourced GPS tracking data could be a useful source to support the selection of manual or other counting sites.

Mobile carriers also routinely obtain mobile phone positions (at least to cell level) via call data records (CDRs) as part of their operations. Geers and Karndacharuk (2016) reported that TMR is currently investigating the utility of the Vincinity offered by Telstra. Vincinity uses CDRs to obtain accurate location data from the Telstra 24 × 7 app and WiFi-based positions from the Telstra Air WiFi network. Together with anonymised and aggregated customer demographic data, Vicinity is able to estimate customer OD, peak demand and to assess public transport capacity.

K.4 Public Transport Ticketing System

In some cities, public transport information could be derived by using the transaction data from automatic ticketing systems. In Queensland, the electronic ticket (*go* card) is a tap-on/tap-off system, from which the bus travel times and occupancy data can be determined with good accuracy (Han et al. 2016). According to TransLink (2016) 85.9% of all public transport trips are paid for using *go card* enabling high confidence when estimating, for example, overall public transport travel demand, OD and mode shifting.

During the latter half of 2016, TransLink deployed the NextBus system as a means of providing real-time data on public transport vehicles to travellers.

K.5 Drones

Drones potentially provide a rich new source of data on traffic movements. Geers and Karndacharuk (2016) reported that trials are taking place using drones to collect real-time traffic information in some jurisdictions such as the Czech Republic. These devices can be equipped with high-resolution cameras, high-capacity data links and automated navigation systems with flight times of several hours. Their use in Australia is currently restricted by the Civil Aviation Safety Authority (CASA) and Air Services Australia.

K.6 On-board Mass (OBM) Measurement

An alternative to measuring vehicle or axle loads at a fixed location using a weigh-in-motion (WIM) device such as CULWAY or at a weighbridge is the recent research into OBM measurement.

In an OBM monitoring system, vehicle mass is measured dynamically as the vehicle travels. OBM measurement uses two mass sensors: load cells and air pressure sensors that measure the mass above the sensors rather than the mass of the whole vehicle. Load cells measure loads as applied to the steel-sprung suspension and fifth wheel of a HV. (A fifth wheel is the horseshoe-shaped coupling device on the rear of a towing vehicle to which a semi-trailer is connected via its coupling pin.) Pressure transducers fitted to air suspension valves measure pressure applied to the air-bag suspension of a HV.

Testing of OBM systems undertaken by Transport Certification Australia (Karl et al. 2009) found that the commercial OBM systems have sufficient accuracy for all types of regulatory applications. Furthermore, tampering can be addressed via the use of dynamic data and it is therefore possible to specify an evidentiary standard OBM system.

Appendix L Use Cases of New and Multiple Data Sources

This appendix describes four recently completed use cases that have demonstrated the application methodology and results of new and alternative data sources in Austroads road agencies.

L.1 Use of Probe Data for Traffic Studies in Perth

Project title

Use of Probe Data for Traffic Studies in Perth (Espada, Salt & Li 2014).

Objective of the analysis

- comparative evaluation of travel time estimates from probe data sets and from floating car survey
- arterial analysis to apply the Network Operations Planning framework using probe data sets through case studies on Stirling Hwy (Eric St to Loch St) and Reid Hwy (Marmion Ave to Malaga Dr)
- freeway analysis using probe data sets through case studies on Roe Hwy (Great Eastern Highway to Kwinana Freeway) and Kwinana Freeway (Beeliar Dr to Mounts Bay Rd) focussed on the use of probe data for performance assessment.

Data sources

This project used Suna probe data sets, developed by Intelematics Australia Pty Ltd, which includes both historic and real-time travel speed estimates.

Methodology highlights

The evaluation of the probe data was conducted by comparing estimated travel time and speed from probe data sets and floating car survey (FCS) data. The comparative analysis examined the following:

- mean and spread of the percentage difference in average travel time and speed over a 10 km route
- correlation of estimated average travel time and speed over a 10 km route
- qualitative assessment of the estimated travel speed profile along three selected routes.

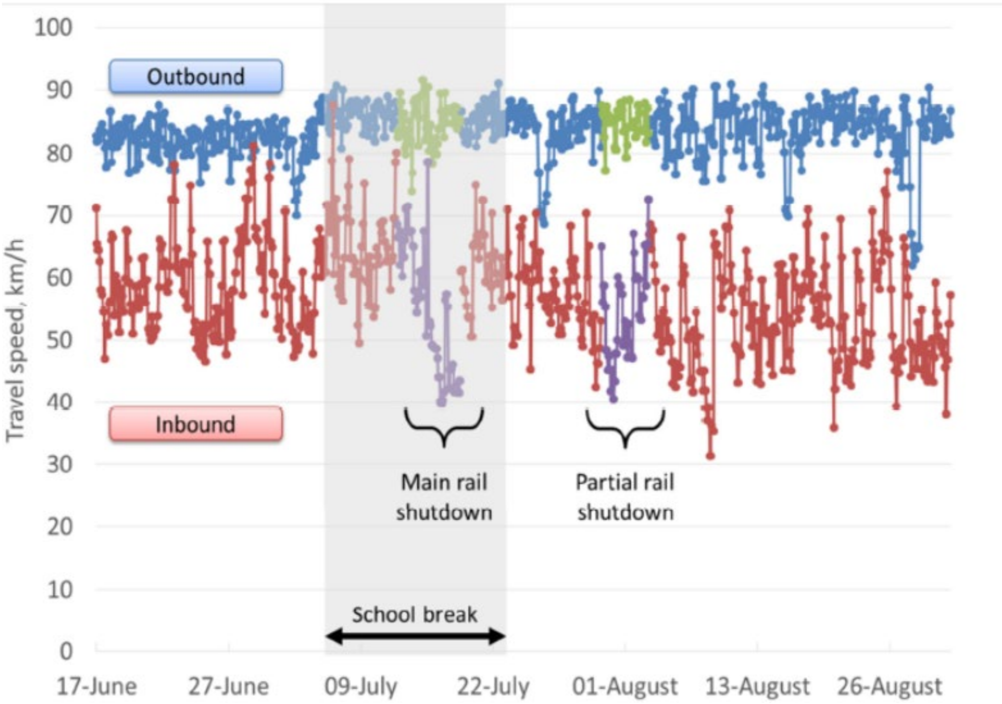
KPIs generated for analysis

Performance gap analysis was conducted taking into consideration the volume and LOS (defined by travel speed) for various road users, i.e. car, transit user, pedestrian and cyclist.

Example of outputs

Examples of system outputs on probe travel time comparison and speed heatmap are given in Figure L 1 and Figure L 2.

Figure L 1: Travel speed from probe data on Kwinana Freeway during railway shutdown estimated (4-minute intervals) during the AM peak



Source: Espada, Salt and Li (2014).

Figure L 2: Weekday speed heat map of Stirling Hwy in the inbound direction from probe data

	Forrest	Eric	Queenslea	Bay View	Leura	Bay	Loch	Vincent
5:00	53	51	55	50	47	57	50	53
	53	50	53	49	49	58	47	54
	56	49	51	47	46	57	43	54
	56	47	49	43	45	57	41	56
6:00	56	47	49	40	45	57	40	57
	55	47	50	36	40	55	40	57
	54	48	50	34	35	53	41	54
	51	48	50	30	37	51	39	53
7:00	49	47	48	27	35	48	38	52
	46	44	45	26	33	48	36	50
	46	42	41	26	32	46	34	51
	45	36	37	25	32	46	32	52
8:00	46	32	32	24	30	45	33	51
	44	31	32	24	28	46	31	50
	44	32	33	25	29	46	30	50
	42	34	36	25	28	47	29	48
9:00	43	38	39	24	30	46	29	48
	41	43	44	24	29	47	29	48
	42	43	46	24	31	48	29	49
	40	43	47	23	27	46	31	50
10:00	40	44	45	22	27	44	30	51
	41	43	45	21	24	43	29	51
	40	43	45	22	24	41	28	50
	41	43	45	22	23	38	30	50
11:00	40	43	45	22	25	38	30	48
	40	44	44	22	25	38	30	46
	37	43	43	22	24	40	30	44
	37	45	43	22	22	42	30	44
12:00	37	44	41	20	22	44	28	43
	39	44	42	21	21	44	26	43
	38	44	44	22	23	43	27	46
	40	45	47	22	26	42	27	46
13:00	42	43	47	22	26	41	27	47
	43	45	47	25	25	40	28	49
	40	46	48	25	25	41	31	49
	42	46	48	26	24	42	30	49
14:00	42	44	47	26	24	42	31	51
	40	45	48	25	25	43	31	50
	39	45	47	22	24	44	31	50
	42	43	45	20	24	43	30	49
15:00	40	41	42	17	23	43	31	49
	42	41	40	17	21	44	32	49
	42	41	38	18	21	43	32	48
	40	40	38	19	20	42	32	48
16:00	39	42	40	21	21	42	31	49
	41	44	43	22	22	42	30	50
	41	44	45	23	22	42	28	49
	45	45	46	24	23	43	28	49
17:00	45	44	47	25	24	44	28	49
	46	44	48	26	25	45	29	50
	47	44	47	27	28	46	28	51
	47	45	48	31	29	47	27	53
18:00	46	44	49	33	33	50	25	54
	50	45	49	36	37	52	26	55
	51	45	47	36	40	52	26	55
	48	45	46	37	39	54	29	55
19:00	47	44	47	36	40	53	32	54
	46	44	47	35	40	52	34	53
	45	45	48	35	40	53	35	53
	44	47	49	36	41	54	36	53
20:00	47	48	51	34	41	54	40	52
	49	49	52	36	42	54	40	52
	49	51	53	40	42	55	43	53
	50	50	53	40	41	55	45	54

Source: Espada, Salt and Li (2014)

L.2 Use of Probe Data for a Congestion Response Plan

Project title

Network Performance Analysis for Perth Congestion Response (Bennett, Espada & Weeratunga 2016).

Objective

To characterise the performance of the metropolitan Perth roads network, to assist in the development of a congestion response strategy.

Data sources

Table L 1: Data sources used for the Perth network performance analysis

Parameter	2015 Data source/analysis	Period
Speed	One year of probe data from TomTom	June 2014 to May 2015 (5 am to 10 pm)
Volume	ROM calibrated using SCATS and VDS	2011 demands calibrated to 2016
Base map	TomTom road network links	n/a (ROM volumes joined to TomTom links)
SCATS congested minutes	SCATS stop line detectors	Recorded during May 2015

Note: ROM is Regional Operation Model.

Methodology highlights

The study applied a reference speed within the formula: delay = travel time (actual) – travel time (reference speed). This reference speed was considered to be the speed at optimal flow, achieved just before flow breakdown, which varies by road types environments and speed limits. The volume to capacity ratio (VCR) above which this initial flow breakdown is likely to occur on arterial roads is 0.85, while for freeways, it is slightly higher, at around 0.9.

KPIs generated from combined data sources

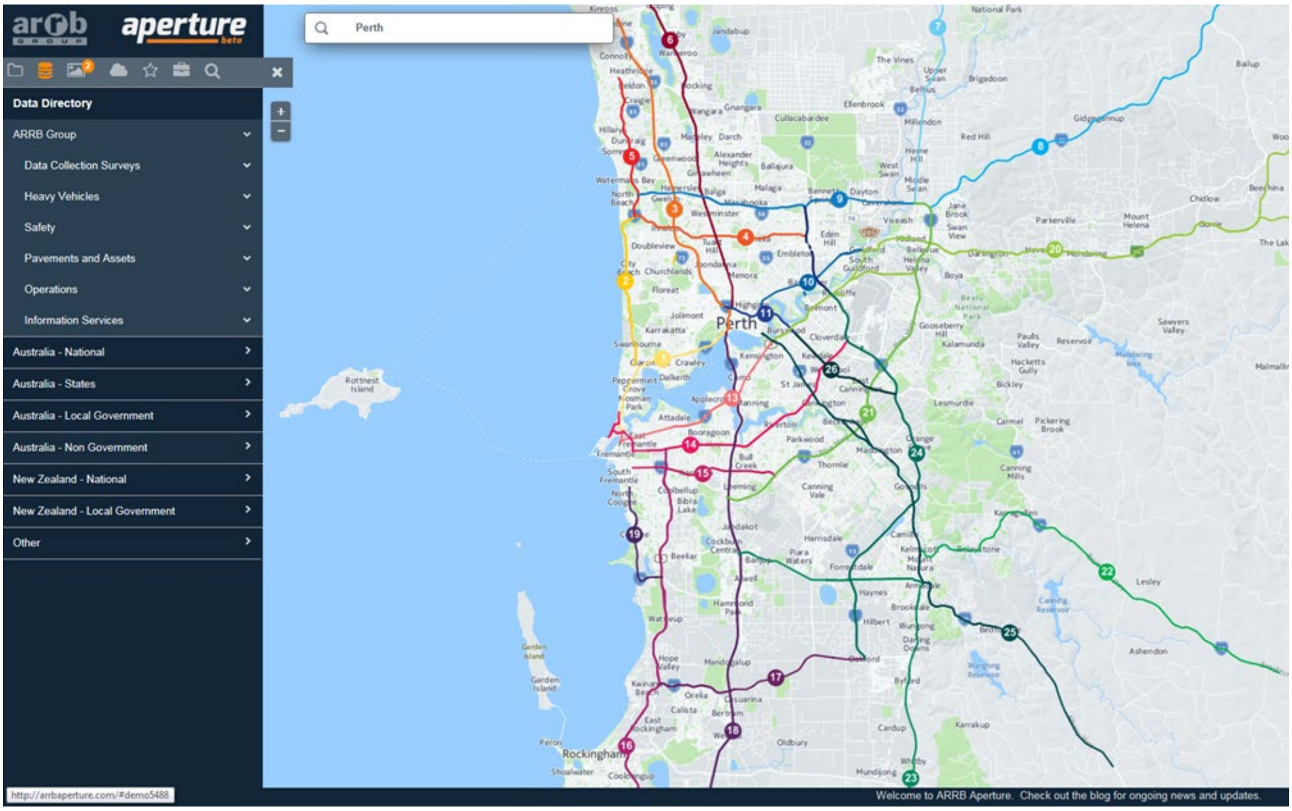
Table L 2: Network performance indicators for Perth

Indicator	Units	Formula
Road length	km	sum of link length
Lane length	lane-km	road length × lanes
Vehicle kilometres travelled (VKT)	veh-km	flow × road length
Vehicle hours travelled (VHT)	veh-h	flow * average travel time
Network speed (Austroads NPI)	km/h	VKT / VHT
Travel time per 10 km (Austroads NPI)	minutes	10 × 60 × VHT / VKT
Total delay	veh-h	flow × (travel time – reference travel time)
Delay per 10 km travel	veh-min	10 * 60 * total delay / total VKT
Variation from posted speed (Austroads NPI)	ratio	(posted speed – average speed) / posted speed
Travel time reliability (Austroads NPI)	ratio	1 + 1.6449 * CV where CV = sd of travel time / average travel time
Planning time index	ratio	90^{th} percentile travel time / travel time at speed limit note: weighted average by flow of links
Relative travel time reliability (5 am = 1)	ratio	90^{th} percentile TT for hour / 90^{th} percentile TT at 5 am note: weighted average by flow of links
Productivity (Austroads NPI)	ratio	(average speed × flow) / (nominal speed × nominal flow); if speed > nominal speed, then 1
Congested lane-km	%	lane-km (speed < reference speed) / lane-km
Cost of congestion	$	total delay × value of time
Total SCATS congested minutes	minutes	sum of congested minutes for all sites
Average SCATS congested minutes	minutes	total congested minutes / number of sites

Examples of outputs

Examples of project outputs on main corridors in GIS, estimated traffic delay and congestion costs are given in Figure L 3, Figure L 4, Figure L 5 and Figure L 6.

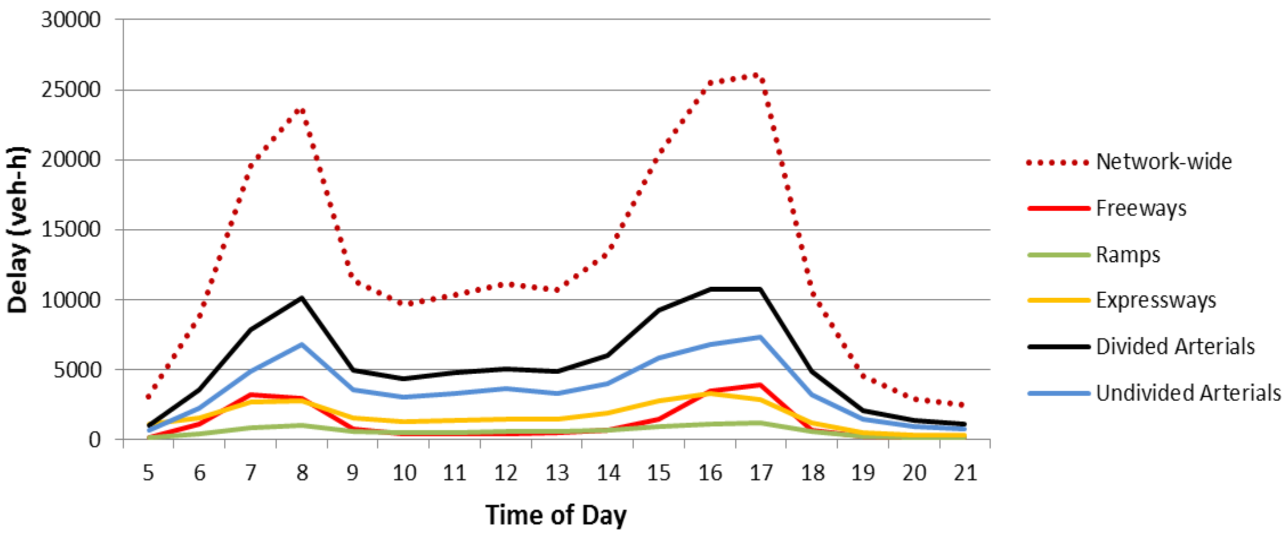
Figure L 3: Screen shot of Perth in Aperture showing the 26 major corridors over a HERE base map



Note: Aperture is an interactive web-based tool developed by ARRB for visualisation of spatial road network data.

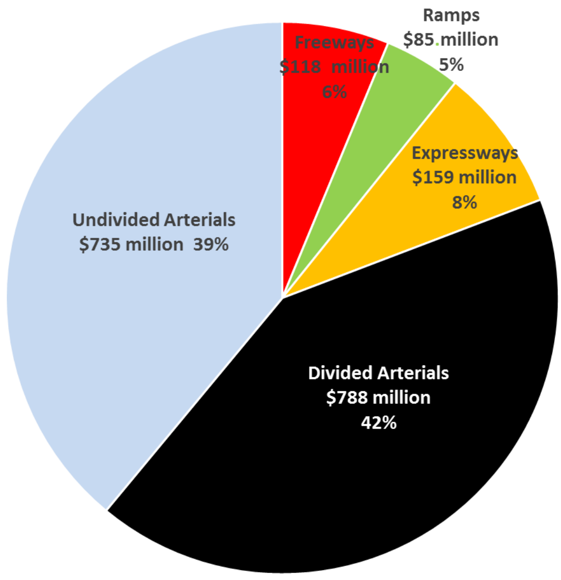
Source: Espada and Bennett (2016).

Figure L 4: Total delay per hour by road type in 2015



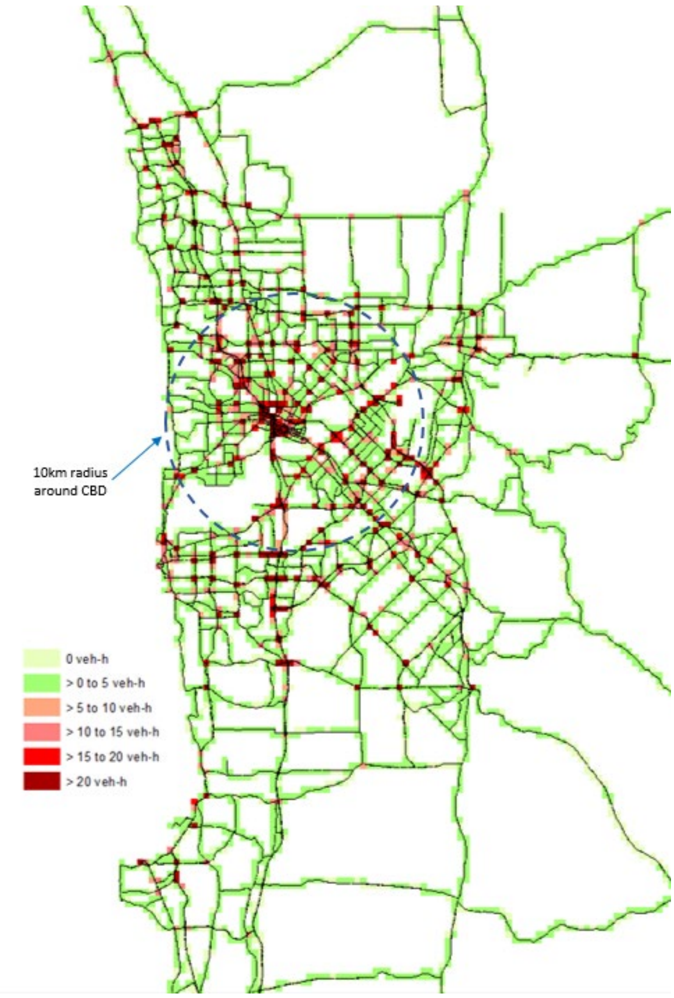
Source: Bennett, Espada and Weeratunga (2016).

Figure L 5: Total annual cost of congestion calculated from speed at optimal flow, achieved just before flow breakdown by road type in 2015



Source: Espada and Bennett (2016).

Figure L 6: Average hourly delay heat plot in 2015



Source: Bennett, Espada and Weeratunga (2016).

Remarks

- This use case analysed Perth road performance at the intersection, route and network level by using a combination of SCATS data and probe data.
- Multiple visualisation tools were also developed and applied for road managers to access data and results at different levels.

L.3 Use of *go* card Data for Bus Congestion Cost Estimation

Project title

Measuring Excessive Congestion Delay and Travel Time Reliability Cost for Multi-modal Travel (Han et al. 2016).

Objective

To estimate congestion costs for buses by using automatic ticketing data.

Data sources

Automatic ticketing system (*go* card) transactions between 1 and 29 March 2015 were used for the study. Bus travel times and occupancy data could be determined with good accuracy.

Methodology highlights

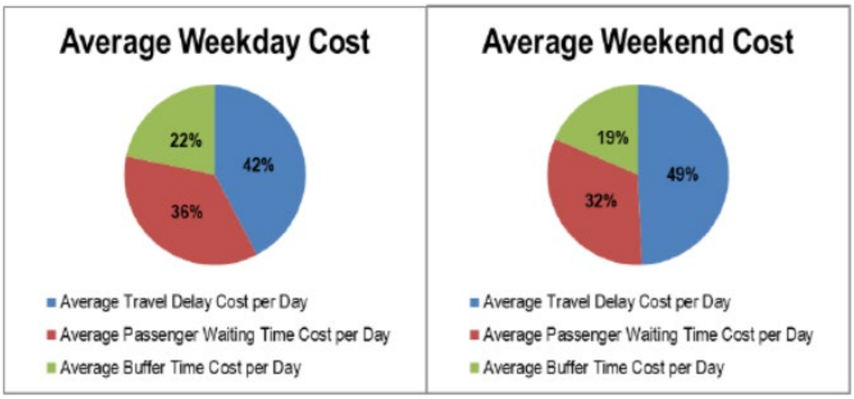
Bus congestion delay was estimated as the sum of the followed three components:

- *In-bus travel time delay*: defined as the prevailing travel time of the bus minus the scheduled bus travel time at each time interval on the same link. The prevailing travel time could be estimated as the time difference between the last *go card* transaction time at an upstream bus stop and the last *go card* transaction time at the next downstream bus stop.
- *Measured buffer time (MBT) for a bus route*: the variability or unreliability of bus travel times was considered in terms of a bus buffer time. When the variation of travel times was analysed at the route level, the measured route buffer time was determined by the difference between the 95th and 50th percentile bus route travel times.
- *Excessive passenger waiting time*: defined as the time difference between a passenger's *go card* touch-on time and the bus scheduled arrival time at a stop (or zero if the bus arrives early). Passenger waiting times at a stop due to late arrivals were also considered as part of the excessive congestion delay.

Examples of outputs

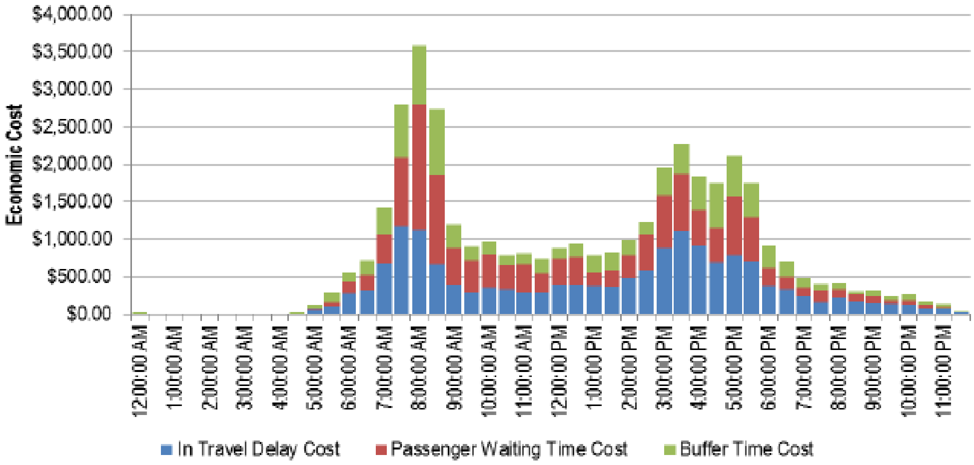
Some examples of project outputs on estimated congestion costs and cost breakdowns are given in the figures below.

Figure L 7: Average weekday and weekend congestion cost, calculated from the sum of excessive congestion delay cost and travel time reliability cost proportions



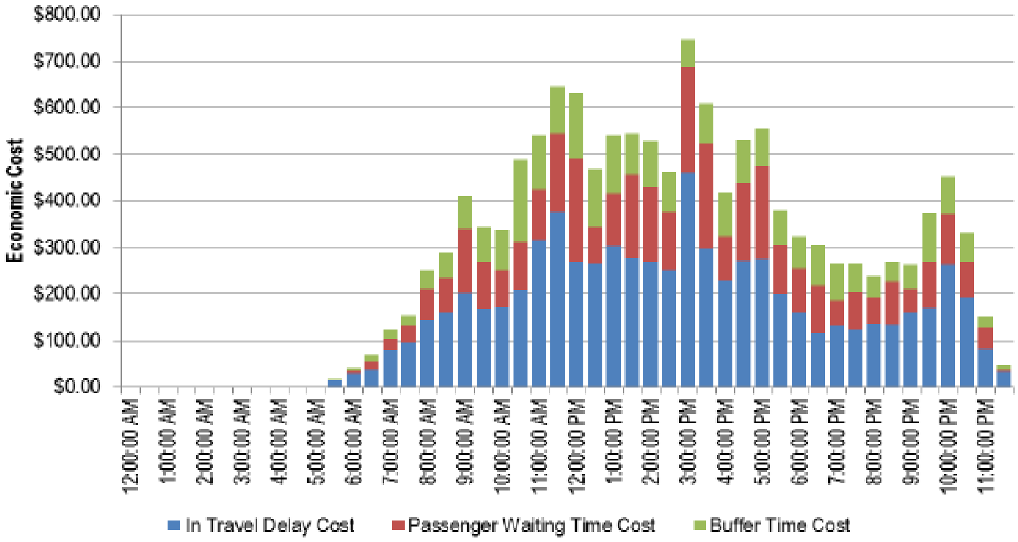
Source: Han et al. (2016).

Figure L 8: Congestion cost, calculated from the sum of excessive congestion delay cost and travel time reliability cost, by time-of-day on 2 March 2015



Source: Han et al. (2016).

Figure L 9: Congestion cost by time-of-day on 1 March 2015



Source: Han et al. (2016).

Remarks

- The use case tested the feasibility of estimating bus congestion cost by using the electronic ticket data. The data analysis yielded reasonable congestion cost values that closely followed expected commuting patterns.
- The methodology used in this case study could also be generalised for other travel modes (e.g. passenger cars and HVs). The feasibility testing of using new data sources (e.g. Bluetooth data and probe data) is on-going.

L.4 Use of Probe Data for the Assessment of Pinch Point Projects in New South Wales

Project title

Post-Implementation Assessment for Pinch Point Projects (Espada & Inglis 2015).

Objective

The Pinch Point Program of Roads and Maritime Services targets traffic congestion points, intersections or short lengths of road at which a traffic bottleneck exists. The purpose of this project was to develop a methodology and tool to assess the impacts of Pinch Point Projects (PPP) through before-and-after analysis.

Data sources

This project used SCATS traffic count data and travel time data from probes to calculate performance indicators for the assessment.

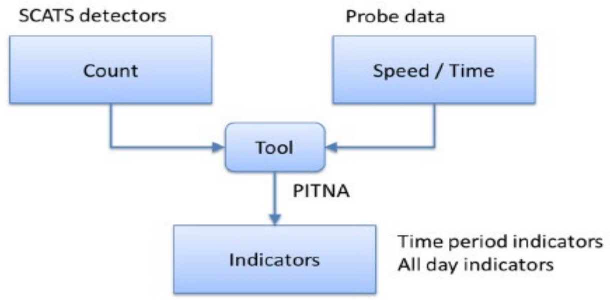
Probe data was sourced from Roads and Maritime Services and could also be sourced from other data suppliers. The probe data sets are travel speed data from fleet vehicles equipped with GPS devices or other technologies that can detect location, such as smartphones. The travel speed database from probes is of high temporal and geographic resolution. The percentage of vehicles on a highway segment with probe devices is relatively low at this stage, but over a long period the number of samples that can be generated would be adequate for analysis in most cases. Nonetheless, the number of probe data points in the network is growing rapidly and the quality of probe data sets will only improve in the future.

Methodology highlights

An Excel tool, Post-implementation Traffic Network Assessment (PITNA) was developed in this project to process probe data and SCATS counts (Figure A 10).

PITNA compiles the traffic count from SCATS and travel speed data from probes and generates the operational characteristics of a road segment before and after the treatment. The analysis is based on the fundamental unit of a road segment which is a stretch of road in one direction. Before-and-after means the before-project indicators are calculated with the before-project traffic flow and before-project operational characteristics. The after-project indicators are calculated with the after-project traffic flow and after-project operational characteristics. For visualisation, SCATS controller locations and detector locations may be included.

Figure L 10: Data sources for PITNA



Source: Espada and Inglis (2015).

KPIs generated from combined data sources

Table L 3: KPIs for the comparison of before-and-after assessment of PPPs

Time period-specific (e.g. 7am-8am)	All day
• Inflow • Outflow • Travel speed • Travel time	• Demand • Vehicle-kilometre-travelled (VKT) • Congested hours, hours wherein the travel speed is below the defined threshold for congested travel speed • Vehicle-hour-travelled (VHT) • Delay, VHT hours travelled under congested conditions (delay is calculated probabilistically) • Planning index, the difference between the average travel time and the 90th percentile travel time multiplied by the number of vehicles. Interpreted as the extra travel time added to the planned travel time for allowance for possible delays such that on-time arrival is achieved 90% of the time, which is a measure of the reliability of travel time on the segment • Nominal planning index – similar to planning index but the extra travel time is the difference between 1015 times the average travel time and the 90th percentile travel time, i.e. it is assumed that travellers normally add 15% of travel time, such that only travel time over and above 15% the average travel time is considered extra • Throughput, the observed maximum outflow when the inflow is sufficiently high (i.e. higher than the outflow) • Monetary benefits, the difference between the monetised values of vehicle-hours with and without the project

Source: Espada and Inglis (2015).

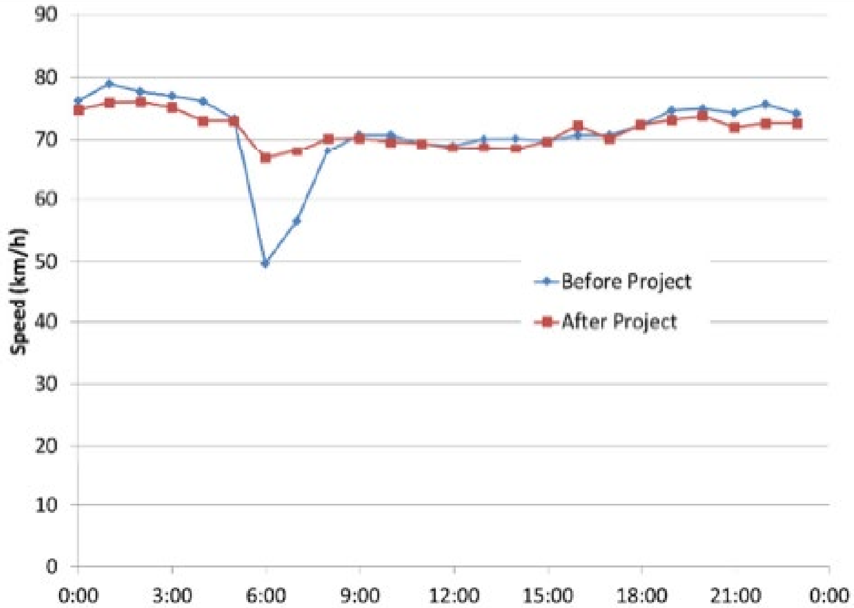
Examples of outputs

Examples of project outputs on the before-and-after study of Pacafic Highway off-ramp widening project are given in Figure L11, L 12 and L13.

Figure L 11: Traffic flow on the Pacific Motorway segment, before and after the off-ramp widening project

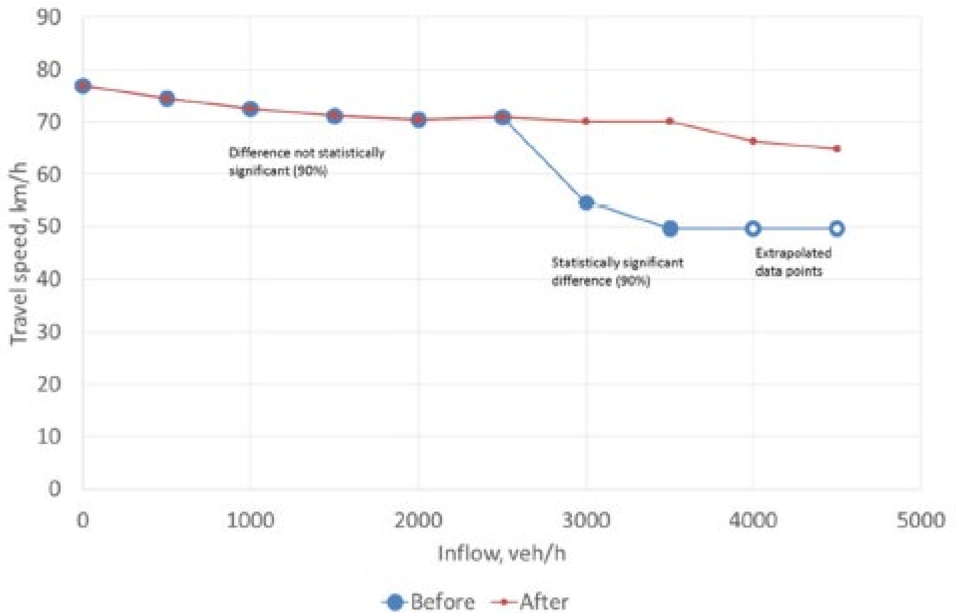
Source: Espada and Inglis (2015).

Figure L 12: Travel speed on the Pacific Motorway segment, before and after the off-ramp widening project



Source: Espada and Inglis (2015).

Figure L 13: Speed and demand relationship of the Pacific Motorway segment, before and after the off-ramp widening project



Source: Espada and Inglis (2015).

Remarks

- The PITNA tool developed in this project utilises a combination of traffic count data from SCATS detectors and travel speed data from probes to calculate network KPIs for project assessment.
- The tool has been successfully utilised by a few before-and-after analysis cases such as the Pacific Motorway off-ramp project and Victoria Road clearway project. The PINTA methodology and tool is simple, easy to understand, robust, inexpensive and able to be used for all relevant existing Roads and Maritime Services datasets. More PPP evaluation case studies are currently being conducted by using the PITNA tool.

Appendix M **Example Packages for Modelling**

Table M 1: Software examples for each modelling technique

Technique	Example software	Description
Macroscopic model	EMME	Transport demand modelling software based on 4-step model (http://www.inro.ca/en/index.php, viewed November 2016)
	CUBE	For the modelling of passenger demand, including the 4-step model and activity-based models (http://www.citilabs.com/software/cube/, viewed November 2016)
	TransCAD	Combines GIS and transportation modelling capabilities in a single integrated platform. It can be used for all modes of transportation, at any geographic scale or level of detail. (http://www.caliper.com/tcovu.htm, viewed December 2016)
	CUBE Voyager	For the modelling of passenger demand, including the 4-step model and activity-based models (http://www.citilabs.com/software/cube/, viewed November 2016)
Mesoscopic model	TRANSYT	For the simulation of signalised road network, with traffic signal optimisation (http://mctrans.ce.ufl.edu/featured/transyt-7f/, viewed November 2016)
	SATURN	Macrosimulation combined with assignment and trip matrix estimation (http://www.saturnsoftware.co.uk/, viewed November 2016)
	SYNCHRO	A traffic signal optimisation tool for arterials and networks, using time-space analysis and platoon dispersion models (Sabra, Wallace and Lin 2000)
	LinSig	http://www.jctconsultancy.com/Software/LinSigV3/linsigv3.php#
Hybrid model	CUBE Avenue	Use and works with traditional four-step transportation planning models or with any model type that uses highway assignment (http://www.citilabs.com/citilabs_products/cube-avenue/, viewed December 2016)
	VISUM	Demand model based on the 4-step model with enhanced traffic assignment which incorporates a node delays and time-dynamic assignment and integrated with VISSIM (microsimulation) (http://www.ptvag.com/, viewed November 2016)
	OmniTRANS	Multimodal and multitemporal system, suitable for modelling the interactions between the various means of transport within an urban context. It supports both aggregated and disaggregated methods for modelling the mobility demand (http://www.dat.nl/en/products/omnitrans/, viewed December 2016)
	INRO Dynameq	Multiscale traffic simulation, it provides an advanced vehicle-based traffic simulation and simulation-based dynamic traffic assignment. It is scalable across wide-area urban networks and provides comprehensive vehicle-level detail throughout the model (https://www.inro.ca/en/products/dynameq/, viewed December 2016
	AIMSUN	For the simulation of a multi-modal transport network, it has the capability model hybrid assignments (http://www.aimsun.com/site/, viewed November 2016)

Technique	Example software	Description
Microsimulation	AIMSUN	For the simulation of a multi-modal transport network (http://www.aimsun.com/site/, viewed November 2016)
	PARAMICS	For the simulation of a multi-modal transport network (http://www.paramics-online.com/, viewed November 2016)
	VISSIM	For the simulation of a multi-modal transport network (http://www.ptvag.com/, viewed November 2016)
	SIDRA TRIP	A single-trip microsimulation model for assessing travel LOS, performance (delay, speed, travel time), operating cost, user cost, fuel consumption, vehicle emissions and noise in real-life road networks (http://www.sidrasolutions.com/, viewed November 2016)
Intersection model	SIDRA Intersection	For the design and analysis of single intersection (signal, roundabout, priority intersections, etc.) (http://www.sidrasolutions.com/, viewed November 2016)
	HCS	A computerised implementation of the Highway Capacity Manual (HCM), and is used to analyse signalised intersections (Sabra, Wallace and Lin 2000) updated to the HCM 2016 (http://mctrans.ce.ufl.edu/mct/index.php/hcs/https://mctrans.ce.ufl.edu/mct/index.php/hcs2010/, viewed September 2017)
	ARCADY	For roundabout analysis and it utilises empirically based models calibrated from British field data (FHWA 2000) (https://trlsoftware.co.uk/) – this UK application is more advanced

Note: Software packages often include a combination of techniques, which sometimes makes classification of software into one of the four types of modelling techniques challenging and subjective.

Source: Adapted from Austroads (2010c), Roads and Maritime Services (2013) and VicRoads (2012a).

Commentary 1

This commentary provides worked examples in capacity analysis.

C1.1 Uninterrupted Single-lane Flow

C1.1.1 Example

In order to carry out major maintenance works on one carriageway of a four-lane divided rural arterial road in rolling terrain, it has been decided to temporarily restrict traffic to a single lane with a pavement width of 3.2 m, with minimum lateral clearances of 1 m on each side of it. Calculate the capacity of a single lane under these conditions, assuming uninterrupted flow, and 10% trucks.

Solution

The base equation is Equation 5 in Section 4.1.1:

$$C = 1800\ f_W\ f_{HV}$$

In this case:

$$f_W = 0.8 \text{ (from Table 4.1)}$$

$$E_{HV} = 4.0 \text{ (from Table 4.2)}$$

$$f_{HV} = 1/[1 + P_{HV}\ (E_{HV} - 1)] = 0.77$$

$$C = 1800 \times 0.8 \times 0.77 = 1110 \text{ veh/h.}$$

C1.1.2 Example

A total of 600 vehicles per hour arrive at a toll booth, the average service time being 4 seconds per vehicle. Calculate the expected mean queue length and mean waiting times. What is the probability of the queue length exceeding 4 vehicles?

Solution

Use Equation 9 to Equation 14 from Section 5.1. In this case:

$$r = 600/3600 = 1/6 \text{ vehicles per second}$$

$$s = 1/4 \text{ vehicles per second}$$

$$\rho = 1/6\ /\ 1/4 = 2/3$$

The mean queue length, including the vehicle being serviced:

$$n_q = \rho/(1-\rho) = 2/3 /1/3 = 2$$

The mean waiting time in the system, including the time being serviced:

$$wm = nq/qa = 2/1/6 = 12 \text{ seconds}$$

The probability of n (> 4) vehicles in the system, including the one being serviced:

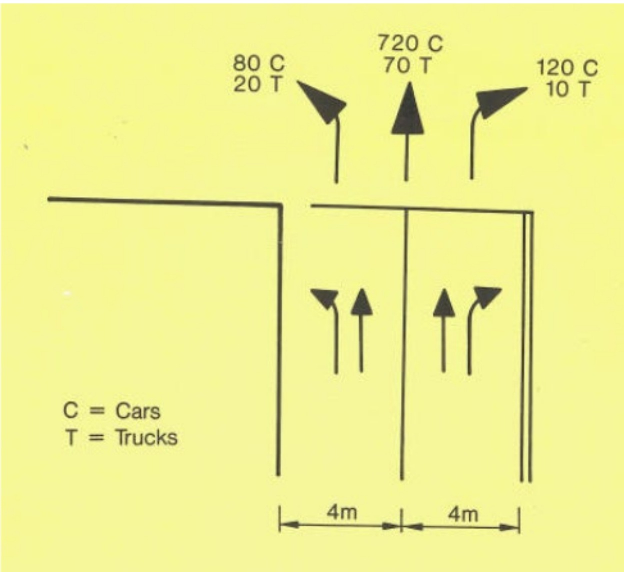
$$P(n>4) = \rho^{n+1} = (2/3)^{4+1} = 0.132 \text{ or } 13.2\%$$

C1.2 Signalised Intersections

A simple example is given here. Refer to Akçelik (1981) for more comprehensive examples.

C1.2.1 Example

Figure C1 1: Example: traffic flows at signalised intersection



The signalised intersection approach in Figure C1 1: is located in a small shopping area where the numbers of pedestrians and the effects of loading and unloading of goods are small. It is designed to desirable geometric standards and the approach grade is flat. Calculate the saturation flow for the approach, and its degree of saturation, assuming the green-time ratio is 0.5.

Solution

Using the descriptions in Section 6.4.2, it can be assumed that:

- environment class = B
- lane type = 2 (for each lane).

From Table 6.4, the basic saturation flow for each lane is 1670 through-car units per hour.

The basic saturation flows should be adjusted to take account of lane widths, grades and traffic composition.

In this case:

As the lane widths are 4 m, then $f_w = 0.83 + 0.05\ w = 1.03$

As the approach grade is flat, then $f_g = 1.00$

The traffic composition factor is calculated as follows:

Adopt an average value of $e_o = 3$, then the data can be tabulated as follows (Table C1 1):

Table C1 1: Calculation of weighted flows for a signalised intersection as shown in Figure C 1 1

	Left		Through		Right		Total
	Car	Truck	Car	Truck	Car	Truck	
Flow (veh/h)	80	20	720	70	120	10	1020
Through-car units/vehicle (Table 6.5)	1.25	2.5	1	2	3	4	
Weighted flow (tcu/h)	100	50	720	140	360	40	1410

Thus:

$f_c = 1410/1020 = 1.38$

Hence, the estimated saturation flow for the approach:

$= f_W\ f_g\ (1 / f_c)S_b$

$= 1.03 \times 1.0 \times (1/1.38)\ (1670 + 1670)$

= 2490 vehicles per hour

The flow ratio for the approach equals the ratio of the arrival flow to the saturation flow. i.e.

$y = 1020 / 2490 = 0.41$

Given that the green time ratio u = 0.50, the movement degree of saturation:

$= y / u = 0.41/0.50 = 0.82.$

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Commentary 2

This commentary provides further detail in the performance of roundabouts which can be explained by evaluating the gap acceptance parameters. The measurement of these parameters is explained below.

C2.1 Measuring the Gap Acceptance Parameters

Troutbeck (1989) found that the gap acceptance parameters were dependent on the circulating flow. At higher circulating flows the gap acceptance parameters were generally shorter. The critical acceptance headways (gaps) were determined using the maximum likelihood method which has been found to have little bias. Most other techniques were significantly affected by the traffic flow in the circulating stream (Brilon et al. 1999).

In the analysis of roundabouts Troutbeck found that all entering drivers gave way to all circulating vehicles. It was on this basis that the critical acceptance headways were evaluated. If it had been found that drivers, entering from the left lane, did not give way to all circulating drivers then the critical acceptance headway would be evaluated on a lesser number of headways (per time period) and the critical acceptance headways would have been larger. The net result would be about the same entering capacity.

Similarly, if it was found that exiting vehicles affected the gap acceptance parameters, then the total number of vehicles (per time period) affecting drivers' ability to enter the roundabout would be increased; these vehicles would be the circulating and the exiting vehicles. With this increased number of vehicles (per time period), the average gap size would be reduced and the recorded critical acceptance gap would be similarly reduced.

C2.2 Development of the Australian Analysis Procedure

Early research by Horman and Turnbull (1974) established the gap acceptance parameters that represented the performance of all roundabouts. Towards the end of the 1980s, it was decided that these parameters should be related to the geometry of the intersection (Troutbeck 1989) and later described in previous Austroads guidelines on roundabouts.

Troutbeck's data was collected at sites with little or no congestion. It was not possible to record the capacity of the roundabout or an approach but only record the traffic operation through the gap acceptance parameters.

Akçelik (1981) was able to observe congested or near congested roundabouts and concluded that the use of the gap acceptance values recorded in previous Austroads guidelines on roundabouts, with the absolute priority capacity theory equations, resulted in an overestimate of capacity for some roundabouts particularly those with higher circulating flows.

Using a detailed simulation of the complete roundabout, Akçelik derived adjustment factors to correct for the overestimation at higher circulating flows. Later, Troutbeck and Kako (1999) confirmed the use of these adjustment factors. They collected field data to validate the concept of limited priority. This process allows for entering drivers to accept a short headway with the following circulating vehicle slowing a little to accommodate the entering vehicles and thereby reducing the following headways and reducing the capacities at the higher circulating flows.

Another innovation from Akçelik was the concept that the follow-up headways and critical acceptance headways would be reduced if the entry flow was much larger than the circulating flow. This effect increased capacity for circulating flows less than 900 pcu/h.

The Akçelik analysis process adopted in SIDRA intersection has two main features. The first is that capacity is increased when the ratio of the entry flow to the circulating flow is high as discussed above and the second is that capacity is reduced when there is a high flow from a dominant approach and vehicles from this approach were queued. Applying these two attributes provides the range of possible capacities for a particular circulating flow.

Akçelik (1998) introduced time-dependent delay equations, and geometric delays. These aspects improved the evaluation process. Akçelik also re-analysed the Austroads data and introduced limits on variables; however, most of the equation forms are based on the Austroads data. In keeping with the philosophy that SIDRA intersection is both a design and research tool, the software has enabled the user to change values from defaults.

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Commentary 3

This commentary lists recommended standard model parameters for MTSMs that were developed by Austroads project NS1229: *Microsimulation Standards* (2006–10). Where cells in Table C3 1 are blank, default values could be appropriate – but users need to apply engineering judgement.

It is acknowledged that these values were developed in 2007 and may have varied over time as the packages have been further developed and it is likely that driving behaviour and some other parameters have changed since then. For driving behaviour parameters, further parameters might be desired in future guidelines such as for different driving conditions (e.g. merge, diverge, weaving), different behaviours (e.g. aggressive near congested areas, relaxed in rural areas), and road types (e.g. highway, motorway, local road).

Table C3 1: Recommended standard model parameters for MTSMs

Category	Parameters	Recommended values or principles			Comments
		AIMSUN NG	Q-PARAMICS	VISSIM	
Traffic	Mean global headway (time)	Not used	1 s	Not used	Q-PARAMICS' default mean global headway is 1 s which is the space time between two adjacent vehicles, i.e. does not include occupancy time. It can be changed on a link-by-link basis
	Minimum headway (space)	2.4 m	2 m (used in Main Roads WA)	2 m (varies by vehicle types and locations)	2.4 m in AIMSUN NG is based on aaSidra's queue space of 7 m and a vehicle length of 4.6 m.
	Speed distribution Car Rigid Bus Semi-trailer B-double	max desired speed 110 km/h 95 km/h 75 km/h 90 km/h 90 km/h	Varies, site dependent	Varies, site dependent	Q-PARAMICS further provides a spread of ±5% based on a default value, e.g. a top speed of 100 km/h will have a range of 95–105 km/h in a simulation.
	Flow profile (OD matrix)	Specified in 15 min slices if data is available			In AIMSUN NG and VISSIM, the flow profile can be inputted as either link flows and turning proportions or OD matrices and routes or paths. Q-PARAMICS only allows an OD matrix as input demand data, specified as profiles at both the origins and destinations
	Yellow time at a signal	Site dependent and a value of 3 s is common			In AIMSUN NG, yellow time is the same for all signals. Q-PARAMICS and VISSIM allow individual yellow time for each intersection.
	All red times	Site dependent and a value of 2–4 s is common			For safety reasons, an all-red time of 2–4 s is used in practice (NSW adopts 4 s all-red time).
	Priority (gap acceptance) rule	Site dependent and a value of 3 s can be used			In VISSIM, the critical gap can vary by vehicle type.

Category	Parameters	Recommended values or principles			Comments
		AIMSUN NG	Q-PARAMICS	VISSIM	
Behaviour	Speed limit factor Car Rigid Bus Semi-trailer B-double	 1.04 1.04 1.00 1.04 1.04	Allow variation of top speeds		The values in AIMSUN NG are based on a speed monitoring study at 6 locations across Adelaide. In VISSIM this is incorporated into speed distribution functions. Q-PARAMICS provides a spread of ±5% based on a default value, e.g. a top speed of 100 km/h will have a range of 95–105 km/h in a simulation.
	Aggressive index	Not used	Normal distribution	Specify maximum deceleration (up to 4 m/s2)	VISSIM uses maximum deceleration and other rates of change/forcing that impact upon vehicles in an adjacent lane.
	Familiarity	Allow sufficient distance in each zone of a three-zone specification	Car - 50% Taxi - 95% LGV - 50% Bus - 100% Rigid - 70% Semi - 85% B-double - 85%		VISSIM uses parameters related to attention span. Different vehicle types in Q-PARAMICS have different familiarity defaults values, which should be changed where appropriate.
	Urban vs rural links	Use special link types with specific parameters to represent, say, rural road characteristics			Q-PARAMICS allows overtaking on the left on urban roads (but not on rural roads).
	Signposting	Allow sufficient distance in each zone of a three-zone specification	750 m for urban freeway, 250 m for urban arterial, sub-arterial, collector and local roads.	1–2 km on freeways	Different packages use different method to warn drivers in a model of an imminent ‘event’ ahead; an event can be an exit via an off-ramp on a freeway. Q-PARAMICS has a plug-in to facilitate lane control anywhere.
Assignment	No feedback	Fixed distance or fixed time mode; fixed-time mode preferred	All-or-nothing assignment based on time, distance or cost, with route perturbation by vehicle types	Fixed routes specified by a user	In AIMSUN NG, fixed distance mode calculates the shortest path at the beginning of the simulation, and the fixed time mode calculates the shortest time paths at the end of a warm-up period if specified. VISSIM also accepts an OD matrix from another planning package, e.g. VISUM. The amount of route perturbation in Q-PARAMICS can be specified in terms of % or in square roots to allow a reasonable level of route diversion for long and short trips.
	With feedback or variable routes (dynamic)	Suggest that shortest paths be recalculated every 5 min and the recalculation makes use of the past 10 min of data			All packages allow dynamic assignment. Need to check stability or convergence of results for each scenario. Suggest getting a model working properly in no-feedback mode before employing dynamic mode. To ensure stability, Q-PARAMICS will decrease the cost on a loaded link that has no assigned trips in a previous iteration (as specified in the feedback delay factor).

Category	Parameters	Recommended values or principles			Comments
		AIMSUN NG	Q-PARAMICS	VISSIM	
Visualisation	Vehicle colours	Colours are a source of data to observers. The Austroads NS1229 Project Group recommends the use of RTA colour standards as shown in Appendix A, which also shows the use of different colours for different trip types in RTA.			In AIMSUN NG, different colours can be applied for the same vehicle type. VISSIM provides for colours to vary according to vehicle type, speed, acceleration, origin or destination.
	Projection (same in a state)		Lamberts 94 system adopted in RTA		
	6 road types (freeway, primary arterial, secondary arterial, collector, local road and centroid connector)	Default road types include arterial, road, freeway, ring road, on/off ramp, urban road, street, signalised street, reserved for public	255 road Categories developed and used. Centroid connectors coded according to links they join	Allows the modeller to create a number of user defined road types	RTA uses a standard road numbering system and standard colour codes for the 255 road categories. Six colours are used, e.g. the red colour is used for 'main roads' of different configurations.
Statistics and post processing	Definitions on stop and queue lengths	Queue-up speed = 5 km/h; queue-leave speed = 10 km/h	A stopped vehicle occupies < 10 m with a speed < 7.2 km/h (MRWA)	Queue-up of 5 km/h and queue-leave of 10 km/h	The default maximum distance occupied in VISSIM is 20 m.
	Suppressed demand	Measure suppressed demand at generators	Measure suppressed demand at generators	Record lost vehicles	
	Level of aggregation – point, segment, route, network (network delay, distance and travel time)	Use default statistics at route, segment and detector levels; post-processing is required to calculate network statistics	Point and network statistics available	Point and network statistics available	RTA calculates all performance metrics/economics based on vehicle files produced in Q-PARAMICS. To ensure consistency in all models developed, RTA also has a 'sealed' value file that must be used on all models.
	Validation and audit		RTA has separate plugin modules for validation and auditing		The Q-PARAMICS validation plug-in compares observed and modelled outputs. The audit plug-in displays variations from default values.

Source: from internal work undertaken by ARRB on behalf of Austroads in the project NS1229 Microsimulation Standards in 2007.

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Commentary 4

Road and Maritime Services has recommended two methods to address the latent demand problem for microsimulation studies under congested conditions:

1. Amend the model to reduce localised congestion and hence reduce or remove the latent demand. Amendments to the model could include one or more of the following:

- reduction in the demand near the congested locations
- longer simulation times, with peak demand spread over a longer period
- expansion of the modelled footprint so that queued vehicles are within the model (and hence contribute to total VHT, VKT, etc.), not outside
- rephrasing of signals near congested release points, to favour vehicles entering the modelled area
- inclusion of small, relatively low-cost ‘essential’ enhancements to the network to address congestion, e.g. new turn bays, longer turn bays, slip lanes, extra lanes in roundabouts, other intersection upgrades and removal of parking.

2. If latent demand still exists after all feasible changes have been made to the model, then the results of the model run need to be manually adjusted to account for the unreleased vehicles. To ensure that all results are normalised and thus usable for comparative purposes (especially economic analysis), all outputs such as VKT and VHT require adjustment to reflect a more realistic and consistent representation of the performance of the network.

   One method of adjustment involves assigning the same average trip length, trip duration, etc. to the unreleased trips as was observed for those vehicles which completed their trips. The steps in this process are:

- number of stops

  Total number of vehicular stops = number of stops as per model output + (average number of stops per vehicle X number of latent vehicles). The assumption here is that the unreleased vehicles would have experienced the same average number of stops per trip as those vehicles which did commence and/or complete their trip.

- travel time

  Average travel time = total travel time as per model output/(number of vehicles that left the network + half the number of vehicles remaining in the network). It is assumed that of the vehicles which are within the network at the end of the simulation period, some have just commenced their trip while others are about to finish their trip. Assuming a uniform distribution between these extremes (i.e. the ‘rule of half’), on average the vehicles within the network have completed half of their desired trip.

  Total travel time = average travel time X (number of vehicles that have left the network + total vehicles in the network + latent vehicles). This adjustment apportions the derived average trip time to the total vehicle demand (i.e. complete, incomplete and unreleased). This is a slightly conservative adjustment, since the additional traffic had been able to enter the network, average travel times across all vehicles would have increased, due to volume/capacity effects.

- trip length

  Average trip length = total distance travelled as shown in the model output/(number of vehicles that left the network + half the number of vehicles remaining in the network). Again, it is assumed that of the vehicles which are within the network at the end of the simulation period, some have just commenced their trip while others are about to finish their trip. Assuming a uniform distribution between these extremes, on average the vehicles still in the network have completed half of their desired trip.

  Total travel distance = average trip length X (number of vehicles that have left the network + total vehicles in the network + latent vehicles). This adjustment apportions the derived average trip length to the total vehicle demand in the model.

Another more abstract way to accommodate latent demand is to add the total latent waiting time due to the latent demand to the total network VHT. This may tend to overstate the effect though – for example, average trip duration within the model may be only 15 minutes, yet the unreleased trips may thus be ‘assigned’ a trip duration equivalent to the modelled period, e.g. 2 hours.

Roads and Maritime Services has also been actively conducting research into more options to reduce the latent demand impact. It is essential that the issue cannot simply be ignored - the use of the incorrect VHT, VKT and stops data would then distort any subsequent project selection and economic analysis.

Further, it must be borne in mind that latent demand is itself a statistical abstraction, an artefact of the modelling process, and as such it would be inappropriate to use the level of latent demand (or its inverse, the level of satisfied demand) as a performance measure when comparing options.

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Austroads' Guide to Traffic Management consists of 13 parts and provides comprehensive coverage of traffic management guidance for practitioners involved in traffic engineering, road design and road safety.

Guide to Traffic Management Part 3: Traffic Studies and Analysis is concerned with the collection and analysis of traffic data for the purpose of traffic management and traffic control within a network. It serves as a means to ensure some degree of consistency in conducting traffic studies and surveys. It provides guidance on the different types of traffic studies and surveys that can be undertaken, their use and application, and methods for traffic data collection and analysis.

Guide to Traffic Management Part 3

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Austroads

Austroads is the association of Australasian road and transport agencies.

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