THE EFFECTS OF LOWER URBAN SPEED LIMITS ON MOBILITY, ACCESSIBILITY, ENERGY AND THE ENVIRONMENT: TRADE-OFFS WITH INCREASED SAFETY?

Final Report to the Federal Office of Road Safety

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August 1997

FEDERAL OFFICE OF ROAD SAFETY

DOCUMENT RETRIEVAL INFORMATION

Report No.	Date	Pages	ISBN	ISSN
	August 1997	142		

Title and Subtitle

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Abstract

This report describes the application of a computer-based traffic network model to a set of synthetic test networks, to examine the effects of lower speed limits on traffic progression, travel times, speeds and delays, fuel consumption and emissions under different traffic control strategies and levels of traffic congestion. It suggests that although travel times increase with lower speed limits, the increases are proportionately less than those indicated by the differences between speed limits. Further, there are indications that intelligent traffic signal coordination systems may further ameliorate such increases.

Keywords
Speed limit, computer model, traffic flow, travel time, travel speed, fuel consumption, pollutant emissions, delays, traffic control, traffic congestion, traffic management, road safety

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Executive summary

Recent studies (e.g. McLean *et al* (1994)) have indicated that lower urban speed limits offer significant road safety benefits. An issue which has arisen in assessing the importance of these benefits is the likely impacts of lower speed limits on other aspects of road travel, such as mobility and travel time, fuel consumption and emissions. This research project is using a combination of mathematical and computer models, and on-road experiments, to provide preliminary indications of the possible effects of reduced speed limits on these factors. It examines the effects of lower speed limits and speed zoning, as applied to a range of urban road and street types, on journey times, mobility and accessibility, and fuel consumption and emissions, in urban and suburban areas.

The research project involved two separate parts:

1. a literature survey to define the present state of knowledge and hypotheses about the impacts of alternative urban speed limits, and
2. a modelling study, using the TrafikPlan computer model of urban traffic networks to examine the effects of different speed limits on traffic performance in some simple network stereotypes. This part of the project thus explores the theoretical impacts of lower speed limits.

This report includes the following sections:

(a) an overview of the relevant theories of traffic flow and vehicle progression in a road network, including traffic signals analysis, the effects of coordination of signals along a road, gap acceptance, and the effects of congestion on travel times. The incorporation of the various theories and component models in the TrafikPlan package is also described;

(b) the development of simple network configurations for theoretical analysis of the effects of different speed limits and the selection of case study networks for 'real world' cases. The definition of suitable ranges of traffic conditions to be examined for each network is also attempted, and

(c) the results of the model runs for the test networks under different speed limits, with comparisons made of relative performance between the speed limits under different traffic control strategies (e.g. peak direction signal coordination) and different levels of traffic congestion.

On the basis of the modelling studies described in the report and the results obtained from the model runs, the following conclusions were be drawn:

1. computer-based modelling of road traffic networks is a powerful tool for investigating the likely impacts of different traffic management and control regimes, such as alterative speed limits, that may be difficult to test in the real world. The effects of different levels on traffic congestion may also be examined, which is again and advantage as the ability to observe a range of congestion conditions in a given real network may be limited The limitation on modelling studies is the need to ensure that they conform to circumstances that will be encountered in real networks, and

2. modelling studies also allow the means to relate measured traffic performance in one network under a given set of operating conditions to those in another network under different conditions.

An analysis of overall network performance from the modelling studies suggested that:

3. journey speeds in the test networks were considerably less than the set speed limits;
4. differences in overall travel speeds and journey times were much less than the differences in the speed limits themselves
5. signal coordination offered significant advantages for delays and traffic progression, except in some cases at the upper congestion level
6. coordinated signal operation combined with a 50 km/h speed limit could, under some conditions, yield traffic operation conditions at least equal to those for the 60 km/h speed limit with isolated traffic signal control

Considerations of traffic performance at the link level in the test networks further indicated that:

7. delays measured relative to the free flow travel time as determined using the specific speed limit were least for the 40 km/h speed limit;
8. other link-based measures of traffic efficiency - travel time, changes in free flow travel times, fuel usage and pollutant emissions - tended to suggest that operations under a 60 km/h speed limit with coordinated signal control gave the best results. Operations under a 50 km/h speed limit with signal coordination also gave good results, often comparable with those for 60 km/h and isolated signal control, and
9. the modelled results for fuel usage and emissions under lower speed limits need to be considered with the rider that the empirical version of the 'running speed' model used to estimate fuel consumption and carbon monoxide emission rates is based on data collected under a 60 km/h speed limit regime, and thus may not completely reflect the situation when free flow speeds are actually at levels set by the lower speed limits. This is an area for further research.

Further conclusions to be drawn from the study include:

10. strategies to increase public support for lower speed limits could, as well as indicating the road safety advantages of lower limits (in terms of stopping distances and severity of pedestrian accidents, etc) include discussion of overall travel time differences being much less than those implied by differences in speed limits and of the improves progression or smoothness of flow implied for the lower limits, for which system delays (i.e. delays relative to free flow travel times) are significantly reduced for the lower speed limits, and
11. modelling of traffic networks should be extended to larger networks, including real world networks of arterial roads and local streets (possibly with LATM schemes) and other traffic calming measures, for which actual origin-destination patterns and intensities are known. Traffic

models should be applied to these areas to test the effects of different speed limits, traffic signal control strategies and traffic calming strategies.

On the basis of these conclusions, a number of recommendations were made:

1. Computer-based models be considered as important tools for the evaluation of traffic network performance and the investigation of traffic management and control strategies, including lower speed limits and traffic signal coordination.
2. efforts to improve traffic signal coordination strategies, for instance to make them more 'intelligent' and responsive to if not anticipative of changes in traffic demand, should be intensified, as improved signal coordination may remove any losses in traffic efficiency (e.g. in door-to-door travel times, fuel consumption and pollutant emissions) that may accompany the introduction of reduced speed limits.
3. Further research and development is needed to examine the likely fuel consumption and emissions performance of traffic streams operating under lower speed limits.
4. Strategies for increasing public acceptance of and support for lower speed limits could use a combination of the safety benefits, an explanation of the actual differences in door-to-door travel times under different speed limits, and the likelihood of smoother, less-stressful driving possible due to reduced delays (measured as a proportion of stopped time for a journey).

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1. INTRODUCTION

Recent studies (e.g. McLean *et al* (1994)) have indicated that lower urban speed limits offer significant road safety benefits. An issue which has arisen in assessing the importance of these benefits is the likely impacts of lower speed limits on other aspects of road travel, such as mobility and travel time, fuel consumption and emissions. This research project is using a combination of mathematical and computer models, and on-road experiments, to provide preliminary indications of the possible effects of reduced speed limits on these factors. It examines the effects of lower speed limits and speed zoning, as applied to a range of urban road and street types, on journey times, mobility and accessibility, and fuel consumption and emissions, in urban and suburban areas.

The research project as initially defined involves three distinct parts:

1. a literature survey to define the present state of knowledge and hypotheses about the impacts of alternative urban speed limits
2. a modelling study, using the TrafikPlan computer model of urban traffic networks to examine the effects of different speed limits on traffic performance in some simple network stereotypes. This part of the project thus explores the theoretical impacts of lower speed limits, and
3. an experimental program designed to offer the means for a necessarily partial verification of the theoretical results produced in 2 above.

This report deals with the design of the modelling study, and the selection of the test networks and levels of traffic activity. It also includes a description of some initial on-road tests, involving the journey time and fuel consumption effects of some traffic control devices used in local area traffic management.

The report includes the following sections:

(a) an overview of the relevant theories of traffic flow and vehicle progression in a road network, including traffic signals analysis, the effects of coordination of signals along a road, gap acceptance, and the effects of congestion on travel times. The incorporation of the various theories and component models in the TrafikPlan package is also described;

(b) the development of simple network configurations for theoretical analysis of the effects of different speed limits and the selection of case study networks for 'real world' cases. The definition of suitable ranges of traffic conditions to be examined for each network is also attempted;

(c) the results of the model runs for the synthetic networks under different speed limits, with comparisons made of relative performance between the speed limits under different traffic conditions, and

(d) recommendations for future work.

2. THEORY

The following chapters (Chapters 3 to 10) of this report outline the theories of traffic flow and vehicle progression through a road network that are implemented in the TrafikPlan computer model of urban traffic networks. TrafikPlan has the capability to represent traffic behaviour on different road types and through different traffic management and control systems, and is the overall model to be employed in the study of the effects of lower speed limits on travel times and traffic network performance. The TrafikPlan model and its predecessor MULATM are described in detail in Taylor (1989) and Taylor (1992a).

An overview of the various concepts and model components of TrafikPlan is necessary, before considering the details.

2.1 Hierarchy of roads

The description of urban road networks and traffic behaviour and expectations on different parts of a network is conveniently addressed through the concept of the functional classification of roads (e.g. Brindle, 1989). This hierarchy of road classes is widely used to examine the primary functions of a given road section, which might be broadly described in terms of traffic carrying (mobility) or land use accessibility functions. Main (arterial) roads are intended to provide for traffic throughput, or mobility. Minor roads and streets provide for access to properties and land uses. The main roads may then tend to be high-capacity, and perhaps high-speed, facilities, with separation of vehicular traffic flows from land use activities. The minor streets have low traffic-carrying capacity and, desirable and probably, low-speed environments conducive to a range on human activities. These distinctions in function are sometimes difficult to make for some roads, and street design, management and control measures may be needed to reinforce the importance of a given primary function (e.g. see Westerman (1990, 1993)).

TrafikPlan requires that the roads in a modelled network are classified according to the following functional hierarchy (with some physical and geometric characteristics also included:

1. local street;
2. collector road;
3. arterial road (single carriageway, single carriageway with tram lines, dual carriageway, dual carriageway with service roads);
4. expressway/freeway access and egress links (on-ramps and off-ramps), and
5. expressway/freeway.

Different speed limits, traffic management and control devices, speed control devices and intersection controls can then be set for the road links in a TrafikPlan network.

2.2 The TrafikPlan model

The modelling capabilities of TrafikPlan include the estimation of travel times on network links, and delays for all turning movements at intersections, using theoretical models of traffic capacity, delays and queuing for different intersection control types, models of free speeds on different road types, and models of vehicle travel times for isolated vehicles, free flowing traffic, forced traffic flow, and interrupted flows at intersections. the model has the ability to determine the minimum costs paths for given journeys and the most likely paths for origin-destination movements, and to assign an origin-destination matrix of vehicle trips through the network. This trip assignment capability enables the model to be used in studies of the route diversion effects of alternative traffic management plans and the traffic impacts of new land used developments in an area.

On the basis of modelled or observed traffic conditions and congestion levels in an area, TrafikPlan can estimate traffic and environmental impacts such as average travel times and speeds, fuel consumption, and pollutant emissions.

Descriptions of the theories and models employed in TrafikPlan follow in subsequent chapters of this report.

For the present project, the primary variable of interest is the posted speed limit on the road links in a network, and the effects of changing speed limits on overall network performance such as trip travel times.

2.3 Concepts of traffic progression

The theories and models of traffic flow employed in TrafikPlan lead to a conceptual model of vehicle and traffic progression along a route through a road network. This conceptual model treats the vehicle trip as a series of movements, along road links, between queuing points (e.g. intersections), at which delays may be experienced. The movements along links are made at a cruising speed, which is set by the speed limit, road type and geometry, and the prevailing traffic conditions. Achievement of the cruising speed requires that the vehicle is able to accelerate to that speed on entry to a link, and is also constrained by the speed that the vehicle must adopt to leave the link (e.g. deceleration to rest at a stop sign). The delays depend on the queuing regime (dependent on traffic control type and traffic management measures) in place at the queuing point, and on the level and spatial orientation of traffic activity at the queuing point.

2.4 Overview of the report

Chapters 3-10 introduce the concepts, theories and models that are applied in TrafikPlan to model vehicle progression through the network.

Chapter 3 describes the components of travel time that combine to determine the overall travel time along a link, or on an entire journey. It provides a simple model of the movement of an isolated vehicle, subject to the attainable speeds on various components of a link. The chapter also defines the delay and queuing parameters that need consideration when assessing the influence of intersections on travel times.

Chapter 4 considers the nature of traffic congestion and the means to assess the level of congestion. This includes an introduction to some of the ‘aggregate’ models that may be used to predict overall link travel times.

Chapter 5 introduces the main classes of intersections in urban road networks, and the broad types of mathematical models used to estimate intersection capacity, delays and queuing. These models, as implemented in TrafikPlan, are then described in detail in Chapters 6 and 7. The description includes the definition of some traffic performance parameters, such as average delay and number of stops. Chapter 8 describes some other important performance parameters, for fuel consumption and pollutant emissions, and indicates how these can be estimated using the delay and queuing models.

Chapter 9 enlarges the focus of the investigation from the traffic performance of individual intersections to are-wide traffic control, and the relationships between neighbouring intersections, including signal coordination. A model for signal coordination on arterial roads is then presented in Chapter 10.

Chapter 11 of the interim final report provides a summary of the project plan, for the use of the TrafikPlan model to study the effects of different speed limits in a range of synthetic networks. These test networks are introduced in Chapter 12, and travel demand patterns and congestion levels for the networks are devised.

The modelling analysis and results are reported in Chapter 13.

Chapter 14 provides conclusions and recommendations, and indicates the possible directions for future work.

3. THE NATURE OF TRAVEL TIME ON URBAN ROADS

Travel time for a journey by motor vehicle through an urban network, i.e. the progression of that vehicle, may be viewed as a process of movement between queuing points. Movement takes place along road links, and the maximum permissible instantaneous speed is determined by the speed limit or speed zone set for the particular road section. At the queuing points, which correspond to intersections, pedestrian crossings, traffic and speed control devices, public transport stops, railway crossings, 'bottlenecks' (e.g. where the road geometry changes) and other points of traffic friction, the vehicle has to slow down to negotiate the traffic obstacle, or even has to stop (such as at a traffic signal, or to join the back of a queue). The effects of traffic friction may be felt at a variety of locations along a given road section, whereas the other effects will be experienced at set locations. The description of the journey made by the vehicle may thus be seen as consisting of two broad components. The first is 'cruising', in which the moving vehicle travels at speed which fluctuate around an overall 'running speed' (v_r). As a first approximation the running speed may be taken to be roughly equal to the speed limit, or at least to be indicative of it. The second broad component of the journey is 'idling', when the vehicle is waiting, perhaps in a queue, for an opportunity to resume its motion. There are also components of the journey time spent in

(1) decelerating to join the end of a queue, to slow down to a speed at which a traffic control device can be negotiated, or to allow other traffic manoeuvres to be made in front of the vehicle (e.g. allowing traffic out of a side street, or a bus to move off from a bus stop), and

(2) accelerating back to the cruise speed range after leaving a queue or negotiating a bottleneck or traffic control device.

3.1 Travel time components

If T is the total time taken to complete a journey of length D, then this travel time comprises the total time spent moving, decelerating, accelerating and idling on a given journey. A speed-time profile can be constructed, to show the speeds of the vehicle over the journey time. Figure 3.1 provides one example of a speed-time profile for an urban journey by a passenger car. The figure shows the second-by-second speed of the vehicle on a given journey, and clearly indicates that the vehicle spends a succession of short intervals of time in cruising and idling, with accelerations and decelerations in between. The important factor to note is that time spent idling is a considerable component of the total journey. Only a small fraction of typical journeys in urban road systems is spent cruising. For the case shown in Figure 3.1 the car is the TSC's instrumented vehicle, a Ford Falcon station wagon equipped with the FCTTDAS fuel and travel time data acquisition system developed by the Australian Road Research Board. FCTTDAS records the speed, distance travelled and fuel consumed by the vehicle in one second intervals. These data may then be used to plot the speed-time profile for a given journey, and to analyse the journey and the components of vehicle progression in it. For instance, one important parameter reflecting the level of congestion on a road is the proportion of time that the vehicle is stopped (Taylor, 1992b). If the total time for the journey is T and the time spent at rest (i.e. stationary) is T_i, then the proportion of time stopped is T/T_i.

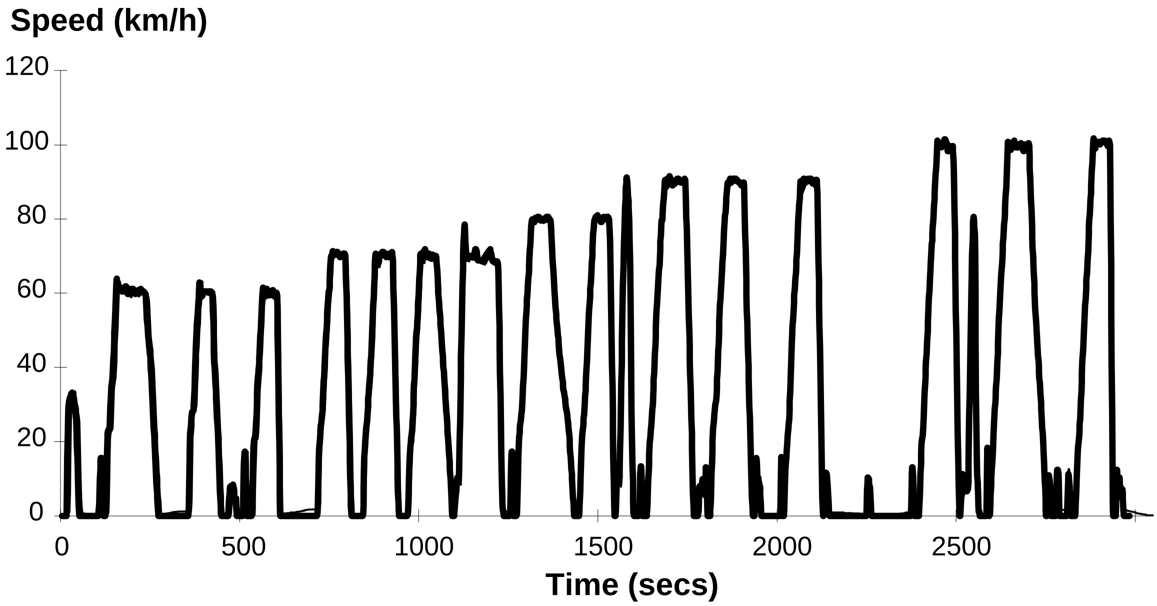


Figure 3.1: Speed-time profile for a journey through a metropolitan area

Now if T_i is the stopped time on the journey, then the total time spent moving is $T - T_i$. Although there is a need to construct detailed models of travel time, as will be seen in later sections, a first useful approximation is to assume that the acceleration and deceleration times are so small that they can be ignored, and Figure 3.1 provides some justification for this approximation. This approximation gives the first indication of the possible relationships between total travel time (T) and running speeds and idling times. Assuming the total time spent in motion is T_r, then we can write

$$T = T_r + T_i \tag{3.1}$$

Now, using the assumption that when in motion the vehicle will travel at the running speed v_r given by

$$v_r = \frac{X}{T_r} \tag{3.2}$$

where X is the distance travelled, we can see that the overall journey speed v_s, given by

$$v_s = \frac{X}{T} \tag{3.3}$$

will be less than v_r, i.e. $v_s < v_r$, if T_i is finite. Thus the idling time becomes a major factor in determining the overall travel time for a vehicle journey. Models have been developed to describe the running time and idling time components of a journey, and these models can be incorporated into a network model of traffic progression. Subsequent sections of this report describe specific models for different elements of the traffic system. An initial exploration is in order at this stage, to consider the conceptual relationship between cruise speed and travel time along a road section, taking into account the physical capabilities of the vehicle.

Consider Figure 3.2, which indicates a typical speed-distance profile for the progression of an individual vehicle along a road link. The vehicle enters the link at a speed v_0, (e.g. after turning a corner to enter the link, or travelling through a signalised intersection, leaving a queue, or negotiating a traffic control device). The driver aims to reach a cruise speed v_c for travel along the link (of length X), but then has to decelerate to leave the section at a speed v_X, determined by the intersection control, road geometry or traffic management device at its end. The speed v_X might well be zero, if (for instance) the exit control is a stop sign, or the vehicle has to join a standing queue in order to leave the link.

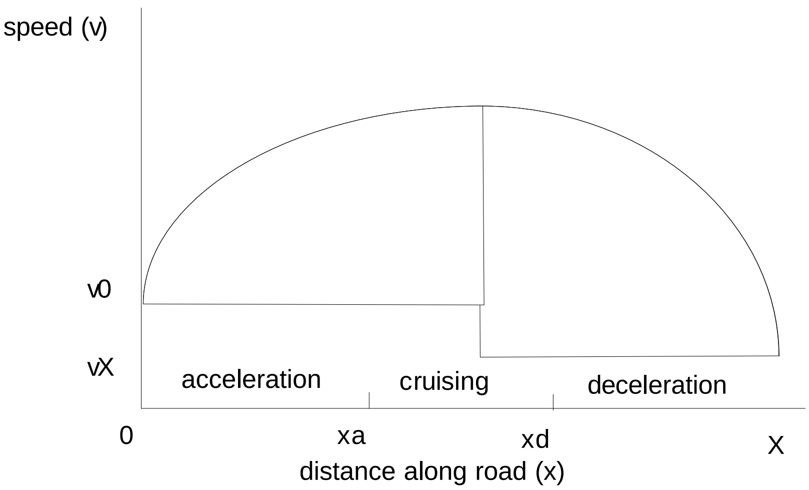


Figure 3.2: Speed profile for an unimpeded vehicle on a road link

Given that a function v(x) of vehicle speed at point x along the link can be defined from Figure 3.2, then the travel time along the link is given by

$$T(X) = \int_0^X \frac{dx}{v(x)} \qquad (3.4)$$

The simplest form for v(x) occurs when it is assumed that acceleration and deceleration take place at constant rates a and d respectively, and that the vehicle will travel at the maximum cruise speed v_c for as long as possible. Then v(x) is given by

$$v(x) = \sqrt{v_o^2 + 2ax} \qquad 0 \le x \le x_a$$

$$v(x) = v_c \qquad x_a \le x \le x_d \qquad (3.5)$$

$$v(x) = \sqrt{v_c^2 + 2d\left(x - x_d\right)} \qquad x_d \le x \le X$$

where the distance required to accelerate to speed v_c is given by

$$x_a = \frac{v_c^2 - v_0^2}{2a} \qquad (3.6)$$

and the distance required to decelerate to speed v_X is given by

$$x_d = X - \frac{v_c^2 - v_X^2}{2d} \tag{3.7}$$

Then the unimpeded travel time for the vehicle is given by the solution of equation (3.4) using speed profile defined by equation (3.5). This is

$$T(X) = \frac{v_c - v_0}{a} + \frac{X - (x_d + x_a)}{v_c} + \frac{v_c - v_X}{d} \tag{3.8}$$

Now this result holds for the case when the vehicle can reach the desired speed v_c within the length of the link. In the case when the link is too short for the vehicle to reach the desired speed before it has to decelerate to leave the link, the model is for the vehicle to accelerate to reach a speed v_m at $x = x_m$, and then decelerate to leave the link at speed v_X. The position x_m along the link is defined by

$$x_m = \frac{2dX + \left(v_X^2 - v_o^2\right)}{2\left(a + d\right)} \tag{3.9}$$

and the speed $v_m < v_c$ is given by

$$v_m = \sqrt{\frac{dv_0^2 + adX + av_X^2}{a + d}} \tag{3.10}$$

The adaptation and use of this model for the modelling of vehicle progression along a road with speed control devices is described in Taylor (1986). The model is employed in TrafikPlan to determine the 'free flow' travel time for a given road link, allowing for the cruise speed, acceleration and deceleration distances, traffic control devices and speed limits.

3.2 Delays and queuing

Delay is an important component of travel time, reflecting the additional amount of travel time imposed by the level of travel demand. In general, we can say that

$$T = T_0 + d \tag{3.11}$$

where T is total travel time, T_0 is a 'free' travel time and d is delay. In this relation d is an overall 'system' delay, including effects from waiting in queues, slowing down to join the end of a queue, and accelerating back to a cruise speed on leaving the queue. It may also include extra travel time incurred in speed fluctuations in moving traffic, as may occur when one vehicle is following behind another in busy traffic. Free travel time (T_0) is the absolute minimum time required to cover a given section of route, determined in TrafikPlan by using the model described in Section 3.1. It thus includes a consideration of the speed limit for a given road section.

3.2.1 Queuing

Before discussing the measurement of queue lengths, it is necessary to define just what a traffic queue is. 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. This section, therefore, concentrates on queues forming in the proximity of a junction or a constriction in the road.

To illustrate the various measurements associated with a queue of stopped vehicles, consider a signalised intersection with vehicles arriving and departing uniformly. Figure 3.3 presents a trajectory diagram for such a case.

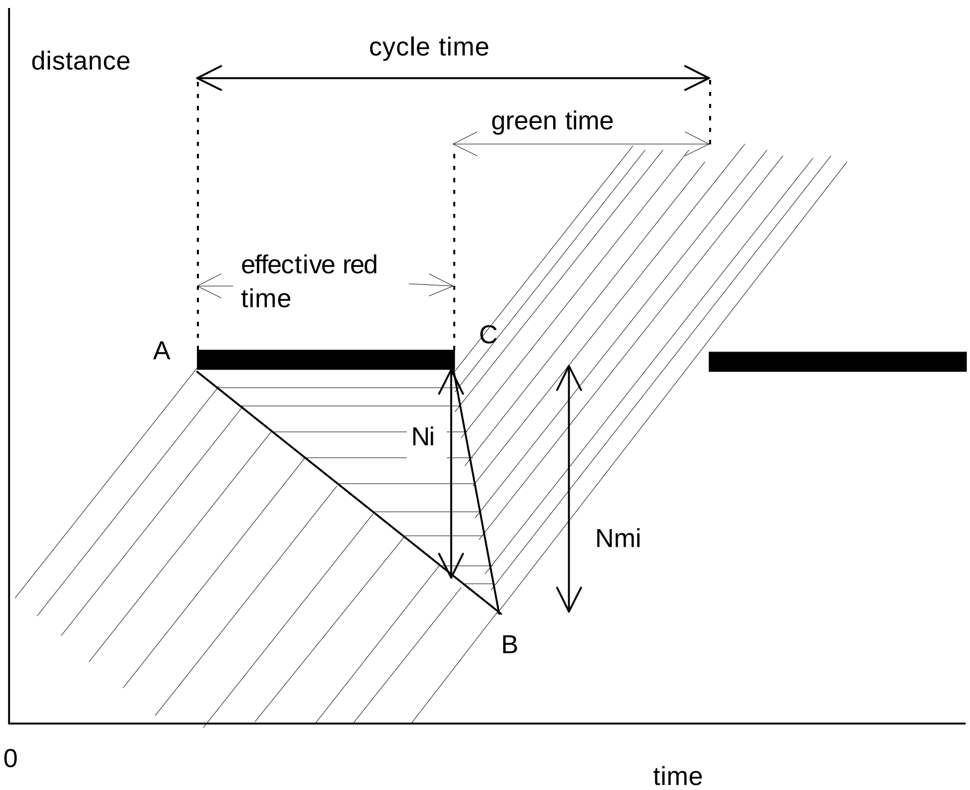


Figure 3.3: Trajectory diagram at a signalised junction

Vehicles arriving during the red phase are halted and are not able to proceed during the red time. The vehicles stopped at the intersection can begin to depart when the green phase starts. The vehicles stopped at the intersection when the lights turn green is the *maximum stationary queue* (N_i). 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* (N_{mi}). This of interest since it represents the end of the queue as perceived by the driver.

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 period.

The above discussion relates to traffic signals. In this case the event that determines the critical time of queue measurement is a change in phase. Queues forming at uncontrolled intersections or at other constrictions in the traffic system have the same 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, as will happen under gap acceptance. In mathematical terms the general introductory queuing problem may be represented by a storage-output equation of the form

$$I(t) = \frac{dN}{dt} + O(t) \qquad (3.12)$$

where I(t) is the arrival flow rate, N(t) is the queue length, and O(t) is the departure flow rate, at time t. Queues form if input flows exceed output flows. Output flows are limited by the discharge capacity of the approach leg to the intersection. At a signalised intersection, for example,

$O(t) = 0$ during the red time

and

$O(t) \le s$ during the green time

where s is the saturation flow. For a fixed time, signalised junction operation with a cycle time c and green time g on one approach, we can say

$O(t) = 0$ for $(n - 1)c \le t < nc - g$

and (3.13)

$O(t) \le s$ for $nc - g \le t < nc$

where n = 1, 2, 3, ... is the cycle number. The inequality in relation (3.13) comes from the rule that

$O(t) = I(t)$ if $N(t) = 0$ and $I(t) < s$.

Queue length at time t is found by integrating equation (3.12) with respect to time, yielding:

$$N(t) = N_0 + \int_o^t I(u)du - \int_0^t O(u)du \qquad (3.14)$$

where N_O is the initial queue length. Delays can also be found. The total delay time D(t) (i.e. delays summed over all vehicles entering the system up to time t) is given by the area between the cumulative arrivals and departures curves. That is,

$$D(t) = \int_0^t N(u)du \tag{3.15}$$

3.2.2 Traffic delay

The second measure of performance of a traffic system considered in this section is traffic delay. There are many definitions of delay. Stopped delay is the delay experienced by vehicles that have actually stopped. This can also be referred to as queuing delay. It is one component of overall delay. Congestion delay can include both the delay due to queuing and that resulting from a vehicle having to slow down because of interactions with other vehicles, and is measured by the difference between journey time and the desired travel time (i.e. using equation (3.11).

Delays represent time that is non-productive, and when converted to monetary values, represent a large proportion of the cost to the community of inadequate transport facilities. Reductions in delay are thus part of the economic return that can be expected if a route is improved and therefore may be used in setting priorities for road improvements. At the same time some congestion delay is an inevitable consequence of traffic demand (e.g. Rahmann (1972), Taylor (1992b)). Thus traffic engineers may seek to reduce delays, but can never eradicate them. The following definitions associated with studies of traffic delay should be noted:

(a) *delay section*. The section of road where all or most of the delay takes place. This section is defined by means of an upstream and downstream marker. The upstream marker should be placed where the vehicles have not started to slow down. This point may be difficult to find under some conditions. The downstream marker is placed at the front end of the queue. In intersection studies this is usually taken as the stop line, even though some delay is incurred while accelerating across that line;

(b) *desired travel time*. The minimum time for vehicles to traverse the delay section. This time can be determined by considering the speed limit and the requirements for acceleration and deceleration in the section;

(c) *desired speed*. The length of delay section divided by desired travel time;

(d) *stopped delay*. The time the vehicle is stationary, due to intersection or other related activity. Stopped delay is the same as stopped time;

(e) *joining time*. That portion of the travel time during which the vehicle enters the delay section and comes to a stop;

(f) *motion time*. That portion of the travel time which occurs between two periods of stop time;

(g) *time in queue delay*. Time from when the vehicle first stops to when it exits the queuing section. In intersections the exit time is measured when the vehicle crosses the stop line;

(h) *unnecessary stopped time delay*. At intersections, this is that portion of the stopped delay which occurs when there is no vehicle entering an approach on the cross approaches of the intersection;

(i) *delay ratio*. Delay time divided by the actual travel time, and

(j) *system delay*. Stopped delay is simple to define and measure. In contrast, system or congestion delay is less precise and its components are difficult to measure with precision. It is the delay caused by the constriction or slowing down effect of overloaded intersections, inadequate carriageway widths, parking and parked cars, crowded pavements and other factors. In overall terms for an extended section of road equation (3.11) provides the measure of system delay.

The prime concerns when determining system delay are the determination of the travel time through the delay section and the desired travel time. The speed limit may be taken as a guide to the cruise speed through the delay section, with the computed value of the desired travel time found by considering the entry and exit speeds for the delay section, the desired cruise speed, and the length of the section.

4. TRAFFIC CONGESTION

The management of congestion is an important issue in modern traffic planning. Much of the work of traffic engineers, planners and analysts focuses on how to ameliorate the effects of congestion, or indeed on how to use congestion to regulate traffic movement through an area. The impacts of new developments or traffic arrangements on existing levels of congestion are always important issues in traffic impact assessment. Congestion is an integral part of a transport system, but its specific definition and identification are not immediately obvious. A number of different definitions of traffic congestion and the observed phenomena associated with it were reviewed by Taylor (1992b). Three recurrent ideas were found:

(1) congestion involves the imposition of additional travel costs on all users of a transport facility by each user of that facility;
(2) transport facilities (e.g. road links, intersections, lanes) have finite capacities to handle traffic, and congestion occurs when the demand to use a facility approaches or exceeds the capacity, and
(3) congestion occurs on a regular, cyclic basis, reflecting the levels and scheduling of social and economic activities in a given area.

The following definition of congestion was proposed for use in traffic studies: 'traffic congestion is the phenomenon of increased disruption of traffic movement on an element of the transport system, observed in terms of delays and queuing, that is generated by the interactions amongst the flow units in a traffic stream or in intersecting traffic streams. The phenomenon is most visible when the level of demand for movement approaches or exceeds the capacity of the element and the best indicator of the occurrence of congestion is the presence of queues' (Taylor, 1992, p. 89). It follows that congestion may always be present in any part of a transport system, but that the level of congestion may have to exceed some threshold value to be recognised. The threshold may be context-specific. Peak periods are recognised as prone to congestion, but this is not to say that congestion does not occur at other times.

4.1 Measuring the level of congestion

The investigation of any traffic planning or traffic management strategy requires the determination and possible subsequent monitoring of the level of congestion. Thus there is a need to collect and analyse data on congestion. Several measures can be used, and although the definition of traffic congestion would suggest that delay time and queue length are essential parameters, they are almost certainly not sufficient measures. The set of factors reflecting the level of congestion includes:

(a) *delay*, possibly disaggregated to consider delays to different road users (e.g. private vehicles, public transport, pedestrians, etc) or delays on different roads (major arterial roads, local roads and streets, etc);
(b) the *equitable distribution of delays* between competing traffic streams;
(c) the *reliability of travel times and travel costs*. Delays reflect an overall (or average) level of congestion experienced by travellers. Under congested conditions individual travellers may experience considerable variation in travel times which have significant effects on travel choices;
(d) *queue management*, which is of importance in urban traffic network control, in the attempt to prevent queuing and congestion at one point in a network from moving

upstream to block other intersections. Queue management is necessary in congested road networks to maintain the overall capacity of the network;

(e) *incident detection*, which is important for traffic flow control on limited access facilities such as freeways;

(f) *excess energy consumption* caused by delays and queuing;

(g) *additional emissions of gaseous and noise pollution* caused by delays and queuing, and

(h) the possible increase in *accident potential* due to reduction in manoeuvring space and increased frustration and anxiety of driving in congested conditions.

4.2 Link congestion functions

In traffic network modelling it is often necessary to describe the overall traffic performance of an element (e.g. a route, link or junction) by a single function, rather than to apply separate functions for free flowing and interrupted traffic. The relationship between the amount of traffic using a network element and the travel time and delay incurred on that element is known as a *congestion function*. The discussions of waiting times and delays based on gap acceptance, queuing theory and traffic signals operations presented earlier provide the basis for these considerations. The total travel time to traverse a network element is directly related to the traffic volume using that element. As volume increases, so delay, and hence travel time, increases. The rate of increase in travel time accelerates as volume approaches the capacity of the element.

Traffic movement along a link in a network may be seen as consisting of two components. The first component is cruising, with traffic moving along the link largely uninterrupted (except for the possibility of side friction, say due to vehicle parking manoeuvres). Travel along the link may also be punctuated by points of interruption, say pedestrian crossings, bus stops and, most importantly, road junctions. For example, the junction at the downstream end of the link may dictate the traffic progression along the link. Movement through the interruption points can be handled using the methods for intersection analysis and queuing theory described previously. A typical congestion function may be seen in Figure 4.1.

The most convenient way to represent a congestion function is in terms of the travel time on a link, e.g. t = f(q, b) where t is the travel time on the link when it is carrying traffic at a flow rate of q, and the vector b represents a set of parameters that describe the characteristics of the link. The function starts with a finite travel time (T_o) at zero flow, and the actual travel time then increases with volume. The rate of increase is small for low volumes, but accelerates once volumes build up towards the capacity of the link. The excess travel time for finite volumes above T_o may be taken as a measure of the system delay (see Section 3.2) on the link, and reflects the state of congestion on the link. A typical congestion function is that developed by Davidson (1978):

$$T = T_0\left(1 + \frac{Jx}{1-x}\right) \qquad (4.1)$$

in which x = q/C is the degree of saturation of the network element, J is an environmental parameter that reflects road type, design standard and abutting land use development, and C is the absolute capacity for the link.

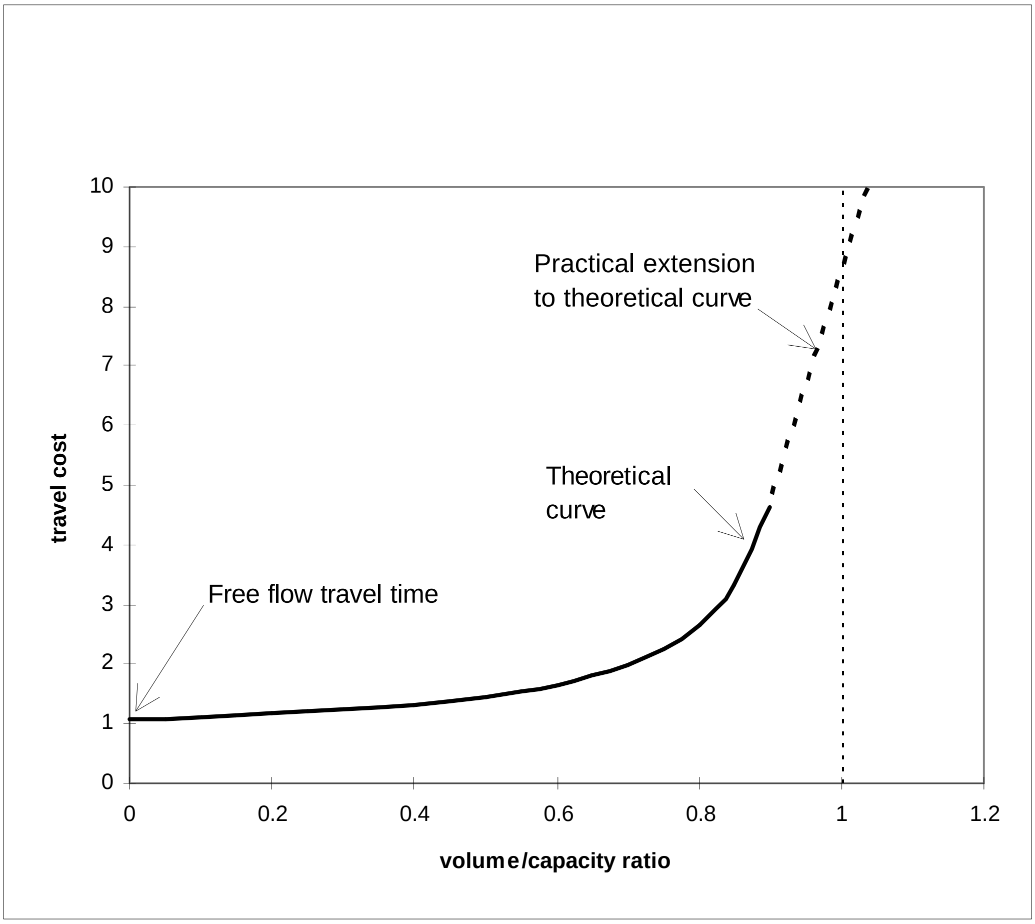


Figure 4.1: Typical form of a link congestion function

The Davidson function has proved popular in economic analysis and travel demand modelling for road networks, mainly because of its flexibility, and its ability to suit a wide range of traffic conditions and road environments. However, the original form (equation (4.1)) has one serious flaw. It cannot define a travel time for link volumes which exceed the capacity (C). This can provide computational problems in (say) a traffic network model which determines link volumes in an iterative manner, and may consequently occasionally overload some links in computing its intermediate solutions. A modification involving the addition of a linear extension term as a second component to the function has thus been proposed and used in transport planning practice (Tisato, 1991). The modified Davidson function is then

$$T = T_0\left(1 + \frac{Jx}{1-x}\right) \qquad x \le \mu$$

$$T = T_0\left(1 + \frac{Jm}{1-m} + \frac{J(x-m)}{(1-m)^2}\right) \qquad x > \mu \tag{4.2}$$

where μ is a user-selected proportion, usually in the range (0.85, 0.95), as discussed by Taylor (1984). This proportion sets a value of q after which the travel time increases as a linear function of q, and this removes the computational difficulties associated with the original function. It should be noted that all of the above congestion functions are 'steady-state' functions, in that they are based on an assumption that the flow q will persist indefinitely.

More recently Akçelik (1991) compared the Davidson function to the delay equations found in traffic signals analysis, and proposed a new link congestion function, better able to model link travel time when intersection delay provides a significant part of the total link travel time. Further, this function also has a 'time-dependent' form as well as the steady state form. The time period over which the volume q (the 'travel demand' to use the route or link) is maintained has an important bearing on the level of delay experienced. The longer this time period T_f persists, the higher the delays will be. Further, the time-dependent equation is designed to cope with periods of oversaturation, as defined by a degree of saturation (or volume/capacity ratio) $x = q/C$ which is greater than one. In this way oversaturation is regarded as a normal condition, which may exist for a finite time period as it does in the real world.

The steady-state form of Akçelik's congestion function is

$$T = T_0\left(1 + \frac{Ax}{CT_0(1-x)}\right) \qquad (4.3)$$

where A is Akçelik's environmental delay coefficient defined as $A = I\kappa$ where I is a factor representing the intensity of delay elements along the link (e.g. the junction density in intersections per kilometre) and κ is a delay parameter reflecting the level of randomness (or regularity) of the arrival and service processes at the interruption points along the link. Appropriate values for κ suggested by Akçelik (1991) were $\kappa = 0.6$ for isolated traffic signals, $\kappa = 0.3$ for coordinated signals, and $\kappa = 1.0$ for roundabouts or other unsignalised intersections.

The time-dependent form of Akçelik's congestion function is

$$T = T_0 + \frac{1}{4}T_f\left\{(x-1) + \sqrt{(x-1)^2 + \frac{8A}{CT_f}x}\right\} \qquad (4.4)$$

Akçelik (1991) provided a set of representative parameter values for use with this model, as shown in Table 4.1. The column T_m/T_o in Table 4.1 is the ratio of link travel time (T_m) for $x = 1$ to the zero-flow travel time T_o. This ratio provides an indication of the additional travel time and delay on the link when it is saturated, and may be taken as one measure of the effects of congestion on that class of road. For instance, the value of 1.587 for a freeway suggests that the travel time at saturation is about 59 per cent higher than the zero-flow travel time, whereas on an arterial road with interruption points (e.g. traffic signals) along it, this percentage increase is 104 per cent, meaning that the travel time has effectively doubled.

Table 4.1
Representative parameters for Akçelik's congestion function

Road Class	Description	t_o (min/km)	C (veh/h/lane)	A	t_m/t_o
1	freeway	0.500	2000	0.1	1.587
2	arterial (uninterrupted)	0.600	1800	0.2	1.754
3	arterial (interrupted)	0.750	1200	0.4	2.041
4	secondary (interrupted)	1.000	900	0.8	2.272
5	secondary (high friction)	1.500	600	1.6	2.439

5. THEORY AND ANALYSIS FOR TRAFFIC INTERSECTIONS

In urban road networks, traffic performance at intersections dictates the performance of the network at large, especially with regard to travel times for journeys. An indication of this influence was provided in Section 3.2, where the nature and definition of traffic delay times were discussed.

Analysis of intersection performance, and the delays and queuing associated with intersection manoeuvres, is thus an important part of the investigation of travel times and journey speeds. Traffic performance of an intersection is a combination of the intersection geometry and the provision for different turning movements, the traffic control system installed at the intersection, the total traffic volume, and the pattern of traffic movements that make up those volumes. A journey through a network, as described in Chapter 3, is very much a movement between a succession of stopping points. Intersections comprise a substantial proportion of these stopping points.

Traffic flows at intersections are examples of interrupted traffic flows, where different traffic streams are obliged to yield to or wait for other traffic streams to complete their manoeuvres. The issues to be addressed in intersection analysis are primarily those relating to the amount of traffic that can pass through a flow constriction over a given period of time and the degree of difficulty (measured in terms of queuing and delays) accompanying that passage.

Two broad type of traffic control system apply at intersections. These are:
1. signalisation, in which the number of movements able to pass through the intersection in a given time period is restricted. Other movements are held up to allow some to use the intersection. In subsequent time periods these positions are reversed, so that over a complete cycle of the signals each movement has a slot of time in which it can operate freely, and
2. major-minor control, in which all traffic streams can operate simultaneously but some streams, such as major road flows or through traffic movements, have priority over others, e.g. traffic on minor intersecting streets or turning movements. Intersections operating under this control regime include major-minor road junctions (with minor road traffic controlled by ‘stop’ or ‘give way’ signs) and roundabouts, and may be termed ‘unsignalised’ intersections as a generic class.

Different models and analytical techniques are applied to the two classes of intersection control. ‘Gap acceptance’ theory, based on a simple model in which minor stream flow unit look for suitably large gaps in the major traffic stream, is used for unsignalised intersection analysis. Signalised intersections are treated using continuity of flow models in which the maximum flow performance of each traffic movement is taken into account in setting an overall cycle time and allocations of green time within that cycle.

6. ANALYSIS OF UNSIGNALISED INTERSECTIONS

Modelling of the traffic performance of unsignalised intersections is done on the basis of gap acceptance theory.

Many traffic manoeuvres involve an individual road user (e.g. driver, rider or pedestrian) selecting a break or *gap* in a traffic stream of suitable size for the safe accomplishment of the manoeuvre. Typical examples are:

(a) a pedestrian crossing a road;
(b) a vehicle entering a road from a car park
(c) a vehicle making a filtered turn through an oncoming traffic stream, and
(d) one vehicle overtaking another.

Gap acceptance methods are based on the assumed distribution of gaps in a traffic stream and assumed rules of human behaviour when searching for suitable gaps to complete a manoeuvre. They permit the prediction of the likelihood of delay and the probable duration of delays at unsignalised intersections.

6.1 Distribution of headways between vehicles

The simplest realistic models of gap acceptance behaviour result when the headways in a major traffic stream are statistically distributed according to the negative exponential distribution. [A headway is the time period between successive vehicles passing a point by the road.] The negative exponential distribution has an important place in traffic flow theory. Its probability density function (pdf) is

$$f(x) = \frac{1}{q} e^{-qx} \qquad (6.1)$$

where q is the mean value, which is equal to the average rate of traffic flow for the time period under consideration. The variance of this distribution is q^2, and the corresponding cumulative density function is

$$F(x) = 1 - e^{-qx} \qquad (6.2)$$

which is the probability of occurrence of an ‘event’ (e.g. a headway) less than or equal to x. The negative exponential distribution is the distribution of headways that represents the Random Traffic Model. The Poisson distribution is the discrete analog of the negative exponential distribution under the Random Traffic Model. This model has provided the basis for the most commonly applied results of traffic flow theory, including expressions for gap sizes and delays. The Random Traffic Model depends on two assumptions:

(1) the time of arrival of a vehicle at a fixed point on a road is independent of the time that the previous vehicle arrived, and
(2) the probability of a vehicle arriving in a small interval of time (δt) is $q\delta t$, where q is the traffic volume.

The negative exponential distribution may also be used to represent the distribution of vehicle spacing under the same conditions. The Random Traffic Model has been widely used in traffic engineering since the 1930s, and continues to provide useful results, despite the restrictive nature of its two basic assumptions. In principle and in practice, these can break down under many common traffic situations. The model is generally considered to apply for uninterrupted flow rates of say:

(a) 500 veh/h total flow on two-way two-lane roads, and
(b) 400 veh/h/lane one-way on a multi-lane carriageway.

As overtaking and lane changing become more difficult due to traffic and environmental factors on a given road, so the Random Traffic Model becomes less applicable. The assumption of independence of vehicle arrival times (hence of vehicle headways) is a particular problem. Firstly, the model assumes that zero headways are possible. Indeed the maximum (or modal) frequency of headways is zero in the negative exponential distribution. Zero headways can occur on a multi-lane carriageway where vehicles can travel side by side, but cannot occur in single lane flows, where the finite length of each vehicle will set a minimum headway greater than zero - remember that the definition of headway is that of the time duration between the arrival of the same physical point (e.g. the front bumper bar) on two successive vehicles. Zero headway between two vehicles in a single lane thus implies that the vehicles have zero length. In urban networks with many signalised intersections, traffic flows are often cyclic in nature, as platoons are released from upstream signals. Cyclic flows will not be random. Thus some care is needed in applying the Random Traffic Model in practice. Nevertheless, the model is easy to use and gives useful if approximate results.

Troutbeck (1986) suggested that a more general form of the negative exponential distribution can provide a more useful description of traffic flow on urban roads. This distribution is based on the idea that a traffic stream can be broken into two distinct populations of vehicles - free-flowing vehicles and tracking vehicles. Free-flowing vehicles move independently of other traffic in the stream, while tracking vehicles are constrained to follow behind the vehicle in front of them. A tracking vehicle follows its leader at a constant headway of τ is, and the tracking vehicles form a proportion (θ) of the total traffic volume in the stream. The free-flowing vehicles are the remaining ($1 - \theta$) proportion of the traffic, and follow a shifted exponential distribution with headways $t > \tau$. The cdf of this distribution is

$$
\begin{aligned}
F(x) &= 0 && x < \tau \\
F(x) &= \theta && x = \tau \qquad (6.3) \\
F(x) &= 1 - (1 - q)\exp[-a(x - t)] && x > \tau
\end{aligned}
$$

where the coefficient α is given by $\alpha = q(1 - \theta)/(1 - q\tau)$.

6.2 Gap acceptance mechanisms

Consider the case of pedestrians waiting to cross a road, as the simplest example of a gap acceptance problem. The mechanism for gap acceptance by an individual pedestrian is as follows:

(a) the pedestrian arrives at the kerbside, and begins to scan the gaps (breaks) in the traffic stream. The first ‘gap’ in the traffic is the time from the arrival of the pedestrian to the arrival of the next vehicle to pass by. This gap is known as the *lag*;

(b) if the lag is of an acceptable size, the pedestrian crosses the road immediately and continues, unimpeded;

(c) if the lag is too small, the pedestrian is delayed and must wait for a subsequent gap that is large enough to allow the crossing manoeuvre to take place, and

(d) a delayed pedestrian may have to wait for r (= 1, 2, 3, ...) gaps before a suitable gap arrives. This means being delayed while a sequence of r successive gaps less than the acceptable gap occur, followed by a gap which is of acceptable size. The time incurred while waiting for the r gaps to pass (which equals the sum of those gap sizes) is the waiting time delay experienced by the pedestrian.

Assuming that individuals behave consistently, then for a given traffic situation they will require a gap greater than or equal to their critical gap before proceeding. Gaps less than the critical gap will be rejected. Gap acceptance is based on a process of human observation and estimation, in which a person observes an approaching vehicle, makes an assessment of the distance to that vehicle and its speed of approach, and then performs a mental calculation to see if there is sufficient time to make the required manoeuvre in safety, or at least with an acceptable minimum perceived risk. This time is the critical gap. It is most likely that different individuals will have different critical gaps, and, further, that as humans are imperfect measuring instruments, they may occasionally misjudge the size of a gap or end up accepting a gap smaller than one previously rejected. In most cases it is acceptable to use a mean value of the critical gap (t_a) to describe the behaviour of all individuals. Note that a group of pedestrians may all use the same gap simultaneously, as they can stand side-by-side (i.e. queue 'in parallel'), so that all of the group can take advantage of the entire gap when it appears.

On the other hand, vehicles in a minor stream must queue one behind the other in a lane, so that only one vehicle at a time can use an acceptable gap. The next vehicle may be able to use the residual part of that gap once it reaches the head of the queue. An additional parameter is needed in this case. This is the 'move-up time' (or 'follow-up headway') t_f, which is the minimum headway between minor stream units. When the vehicle at the head of the queue departs, the next vehicle will reach the stop line t_f seconds later. This translates into the following set of possible events for the vehicles in the minor stream, assuming that t_a and t_f are constants:

(a) gaps less than t_a will not be accepted;

(b) gaps between t_a and $t_a + t_f$ will be used by one minor stream vehicle;

(c) gaps between $t_a + t_f$ and $t_a + 2t_f$ will be used by two minor stream vehicles, and

(d) in more general terms, gaps between $t_a + (i-1)t_f$ and $t_a + it_f$ will be used by $i = 1, 2, 3, \ldots$ minor stream vehicles,

on the assumption that there are always vehicles queued in the minor stream to take full advantage of every possible gap. This means that, in theory, there is then an infinite queue on the minor stream.

In practice the critical gap is difficult to measure (see Taylor, Young and Bonsall (1996), pp.243-244, for a description of possible measurement techniques for critical gaps). If a driver rejects a number of gaps before accepting one then all that can be said is that, assuming consistent behaviour, the driver's critical gap is larger than the largest rejected gap and smaller than the accepted gap. The theoretical results that follow are based on the assumption of known mean acceptable gap t_a and follow-up time t_f.

6.3 Basic results

For random traffic, the cumulative density function (cdf) defined by equation (6.2) defines the probability that a gap will be less than a certain size. The probability of being delayed is thus the probability that the first gap (the lag) is less than t_a. This probability is, from equation (6.2),

$$\Pr\{\text{delay}\} = \Pr\{\text{lag} < t_a\} = 1 - \exp(-q_p t_a)$$

where q_p is the flow rate for the priority stream. The probability of no delay is the probability that the lag is greater than or equal to t_a. This probability is

$$\Pr\{\text{no delay}\} = \Pr\{\text{lag} \geq t_a\} = \exp(-q_p t_a).$$

These results indicate the proportion of minor stream road users that will be undelayed, or will suffer some delay. In addition, the gap acceptance model can be used to estimate the maximum rate at which minor stream units can complete their manoeuvres, and the delays incurred by those units.

The theoretical maximum rate at which minor stream units can enter or cross the major stream is known as the 'absorption capacity' (C). It is found by considering an infinite queue on the minor stream, and assuming that each and every suitable gap will be used to its maximum potential (i.e. the maximum possible number of minor stream units will use every gap which exceeds t_a and in large gaps vehicles follow-up at headways of t_f). Absorption capacity in the Random Traffic Model is then given by the equation

$$C = \frac{q_p \exp\left(-q_p t_a\right)}{1 - \exp\left(-q_p t_f\right)} \tag{6.4}$$

Field values of absorption capacity may be higher or lower than this theoretical value at a given site, usually due to site conditions, the inaccuracy of the Random Traffic Model, or pressure' on drivers due to the degree of saturation at which the intersection is operating. Nevertheless, the result of equation (6.4) provides a useful indication of the capacity of a given traffic movement subject to a gap acceptance filter.

The amount of delay under gap acceptance can also be predicted. The mean delay (w_h) in waiting for a suitable gap under the gap acceptance mechanism described above is given by Adams's formula (Adams, 1936)

$$w_h = \frac{1}{q_p} \exp\left(q_p t_a\right) - \frac{1}{q_p} - t_a \tag{6.5}$$

This result provides the average delay to a minor stream unit in looking for gaps in a major stream. Note that it does not include any queuing delays (i.e. time spent in a queue before the unit reached the head of the queue and can start scanning for gaps). Not all minor stream vehicles are delayed, and the mean delay to those who are delayed (w_{hd}) is equal to $w_h/\Pr\{\text{delay}\}$, which may be expressed as

$$w_{hd} = \frac{1}{q_p}\exp\left(q_p t_a\right) - \frac{t_a}{1-\exp\left(-q_p t_a\right)} \qquad (6.6)$$

Vehicles on a side street are obliged to queue to move up to the stop line and take their turn in scanning for gaps. This queuing process adds another component to the total delay time. Thus the mean delay to minor stream vehicles when queuing is included (w_m) is given by the equation

$$w_m = \frac{w_h + \eta\rho}{1-\rho} \qquad (6.7)$$

where ρ is the degree of saturation and is equal to the minor stream flow rate q_m divided by the maximum absorption capacity C (i.e. $\rho = q_m/C$). The factor η is given by the expression

$$\eta = \frac{\exp(q_p t_f) - q_p t_f - 1}{q_p(\exp(q_p t_f) - 1)} \qquad (6.8)$$

Equation (6.7) provides a useful form of the delay equation because it relates the actual mean delay to minor stream vehicles to the Adams' delay and to the absorption capacity of the minor stream.

6.4 Multi-lane flows

The previous discussions have implied that the major stream flow is in a single lane (or direction) in which minor stream units seek acceptable gaps. These results can be extended to cover the situation of multi-lane flows, with ease when the Random Traffic Model is held to apply.

Consider the case of two major road streams (either two-way traffic or two-lane one-way traffic) with flow rates q_1 and q_2, each with negative exponential headway distributions. The probabilities of occurrence of suitable headways in each of the streams are, from equation (6.2)

(a) stream 1, $\Pr\{t_h \geq t_a\} = \exp(-q_1 t_a)$, and

(b) stream 2, $\Pr\{t_h \geq t_a\} = \exp(-q_2 t_a)$.

Assuming that the two streams move independently of each other, the probability of the simultaneous occurrence of a suitable headway in both streams is given by the product of the above probabilities, i.e. for the combined stream,

$$\Pr\{t_h \geq t_a\} = \exp(-q_1 t_a)\exp(-q_2 t_a) = \exp[-(q_1 + q_2)t_a].$$

This result is in the same form as that for a single lane stream, implying that when the Random Traffic Model is employed for the purposes of gap acceptance analysis, a number of separate major road traffic streams may be combined into a single stream with overall flow rate equal to the sum of the flow rates of the individual streams.

The other situation where multi-lane flows are considered is when there are two or more lanes available for the minor stream. In this case the total absorption capacity for the stream is equal to the sum of the absorption capacities of the individual lanes.

6.5 Combined lanes

Often more than one minor stream may share the same lane on an approach. For example, on a single lane approach to a T-junction, left and right turners may share a single lane, as in Figure 6.1.

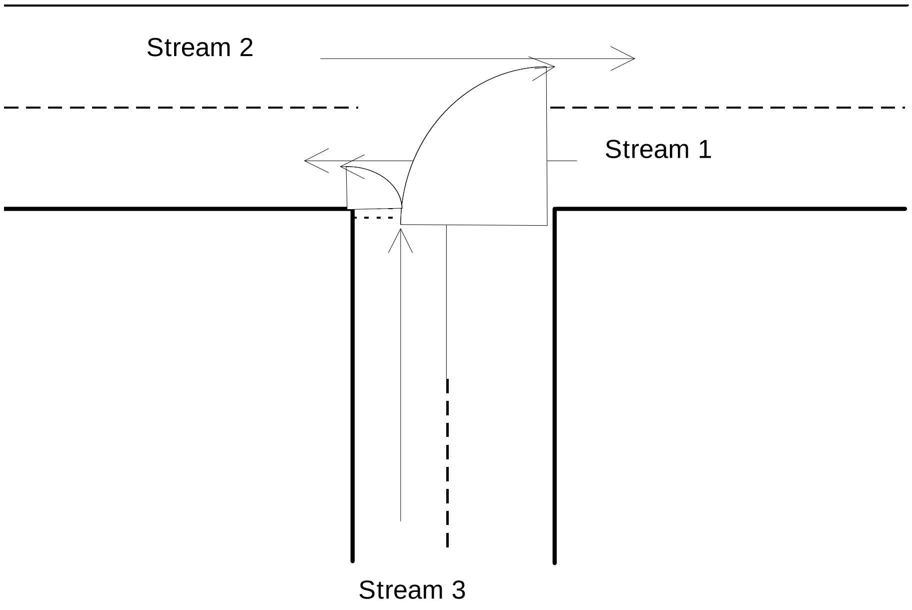


Figure 6.1. Major and minor traffic streams at a T-junction

Now, each of these movements yields to different combinations of traffic streams. In Figure 6.1 the left turners in stream 3 yield to stream 1 only, whereas the right turners give way to streams 1 and 2. Thus the absorption capacities for the two movements are different. In addition, the presence of right turning vehicles in the queue ahead of a left turner reduces the opportunities for the left turner to find and utilise acceptable gaps. For the general case where there are multiple movements sharing a lane, each looking for suitable gaps in a major stream, the overall absorption capacity C of the lane is given by equation (6.9), in which C_i is the absorption capacity of the ith movement in the stream and p_i is the proportion of the total flow in the lane that is making movement i:

$$\frac{1}{C} = \sum_i \frac{p_i}{C_i} \qquad (6.9)$$

7. ANALYSIS OF SIGNALISED INTERSECTIONS

Interrupted flow is a hallmark of urban traffic systems. The major factors affecting urban traffic are those related to the intersections of cross roads in the transport network, where sets of traffic streams must compete for limited resources of road space and time. The effects are two-way, for the operation of the intersections affects the nature of the traffic flow, and the traffic flow affects the operation of intersections. Traffic signals have become the major form of urban traffic control and management, especially for major intersections where large volumes converge. The basis of traffic signals operation is that each of the intersecting traffic streams will be offered a window of time (the 'green time') during which it will have the opportunity to traverse the intersection. The design task is to determine the lengths of these green times so that the intersection can provide efficient operation for all of the intersecting traffic streams. Traffic control technology based on the separation of traffic streams in time but not space, such as traffic signals, automatically builds some delay into the operation of the traffic system, and this delay may have significant effects on overall journey times through a road network, especially in networks where signalised intersections are the predominant form of traffic control.

The cycle of operation of a traffic signal is the total time required before the repetition of the same sequence of the traffic signal *phases*. A phase is a distinct part of the signal *cycle* in which one or more movements receive right of way (i.e. have the green light). A stage (phase) is identified by at least one movement which gains right of way at the start of it and at least one movement which loses right of way at the end of it.

A *movement* is a separate queue leading to the intersection (e.g. right or left turning traffic, or the separate lanes available for passage through a junction). Figure 7.1 shows an intersection plan and simple staging (phasing) diagram for a T-junction. There are six vehicle movements (1-6) and two pedestrian movements (7 and 8) possible. The staging (phasing) diagram shows the movements given right of way in the three phases (A, B and C) of the signal cycle. An important part of the analysis and design of a signal cycle is to determine the set of *critical movements* which determine the capacity and timing requirements at the junction.

7.1 Basic parameters for signal operation

Five main factors influence the capacity of a single movement at a signalised intersection:

(1) the *saturation flow* (s) for each of the lanes used by the movement;

(2) the *number of lanes* available for the movement;

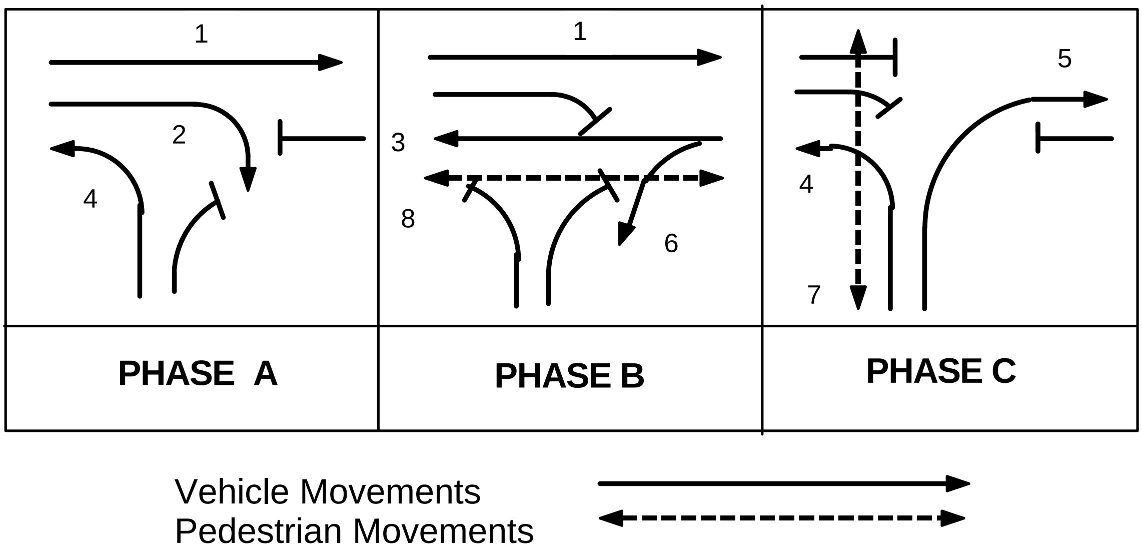
(3) the *lost time* (l) for the movement;

(4) the *cycle time* (c) for the intersection, and

(5) the *green time* (g) for the movement.

Capacity is the maximum volume of traffic that can be discharged through the intersection over an extended time period (i.e. more than one cycle). The saturation flow is the maximum instantaneous flow rate, and is only possible during the green time for the movement.

PHASING DIAGRAM



INTERSECTION PLAN

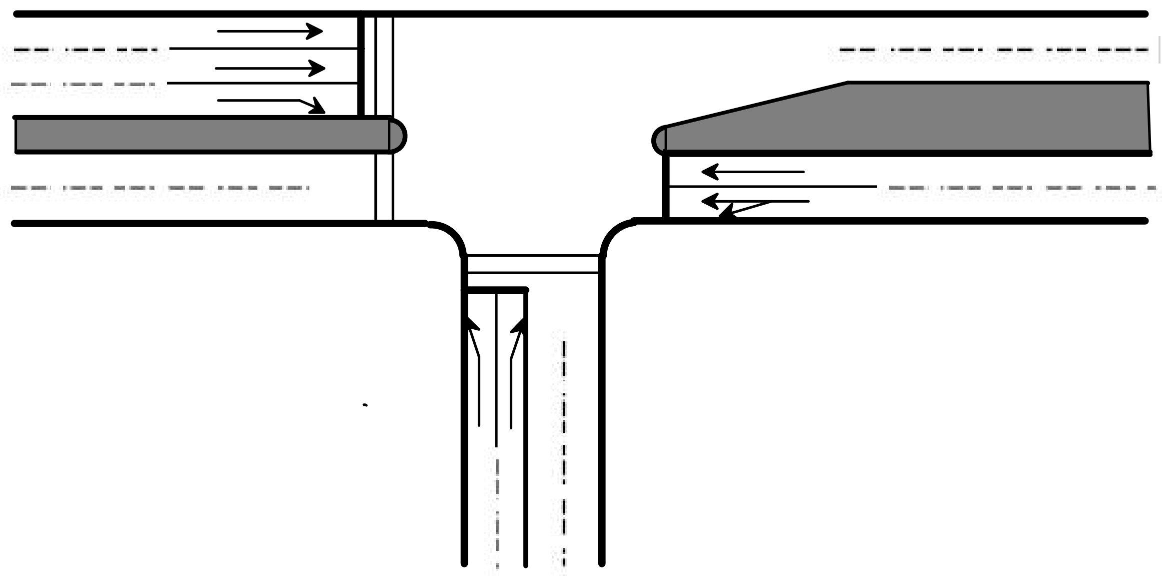


Figure 7.1: Sample junction plan and signal phasing

7.2 Wardrop-Webster model

The Wardrop-Webster model was developed in the UK during the 1950s. It provides the basis of all signal calculations, although the modern implementations of the model are somewhat more complicated than the original version (e.g. see Akçelik, 1981) provides a useful description of the model. Modern design methods based on it are usually implemented as computer software packages, such as SIDRA. (Akçelik, 1985). The procedures described in Akçelik (1981) are incorporated in TrafikPlan.

The basic model is formulated in the following way. Consider a simple four-arm junction, controlled by two phases (one for east-west traffic, the other for north-south traffic) as in Figure 7.2. This figure shows the flow of traffic over time on the east-west road, under the assumption that this road is carrying a sufficiently high volume such that there is always a queue present (i.e. the east approach is *saturated*). The top flow diagram in Figure 7.2 shows the observed flow at the stop line for one lane of traffic on each approach. Once the signal changes to green on one approach, traffic starts to move through the intersection: the rate of discharge (flow rate) builds up rapidly to a value of about 0.5 veh/s (1800 veh/h, implying an average headway of two seconds between successive vehicles), at which it levels off. Flow at this rate will continue across the stop line of the approach until:

(a) the queue is cleared, after which time it would fall back to the arrival flow rate of traffic on the approach, or

(b) the end of the green period, when the flow will reduce to zero again, depending on which event happens first. The pattern is repeated in each cycle - one way to view this periodic phenomenon is as a series of pulses (waves) of traffic released through the signal.

The starting point for traffic signals analysis is the assumption that for saturated flows on an approach to a traffic signal, the observed flow profile as shown in the top diagram of Figure 7.2 could be replaced by a simple rectangular volume-time profile, as indicated in the bottom section of the figure. The area of the rectangle is equated to the area under the observed profile. Now, as the height of the rectangle is determined by the saturation flow, the width of the rectangle is taken as the *effective green time* (g_A) for phase A.

There is a second effective green time (g_B) for phase B. But note from the lower part of Figure 7.2 that g_A and g_B do not add up to the cycle time (c). There is an amount of lost time at the end of each phase. In reality, this lost time is useful because it provides a buffer between the major flow movements (that can be used, for example, to allow vehicles on one road to clear the intersection before the cross-traffic movement starts, and/or to allow vehicles making opposed turns to clear the intersection). The amounts of lost time for each phase in Figure 7.2 are l_A and l_B. Total lost time (L) is an important parameter for the intersection. This time is defined as the difference between the cycle time and the sum of the effective green times for all of the phases:

$$L = c - \sum_i g_i = \sum_i l_i \qquad (7.1)$$

The amount of lost time for a given intersection depends on the number of phases and the intersection geometry.

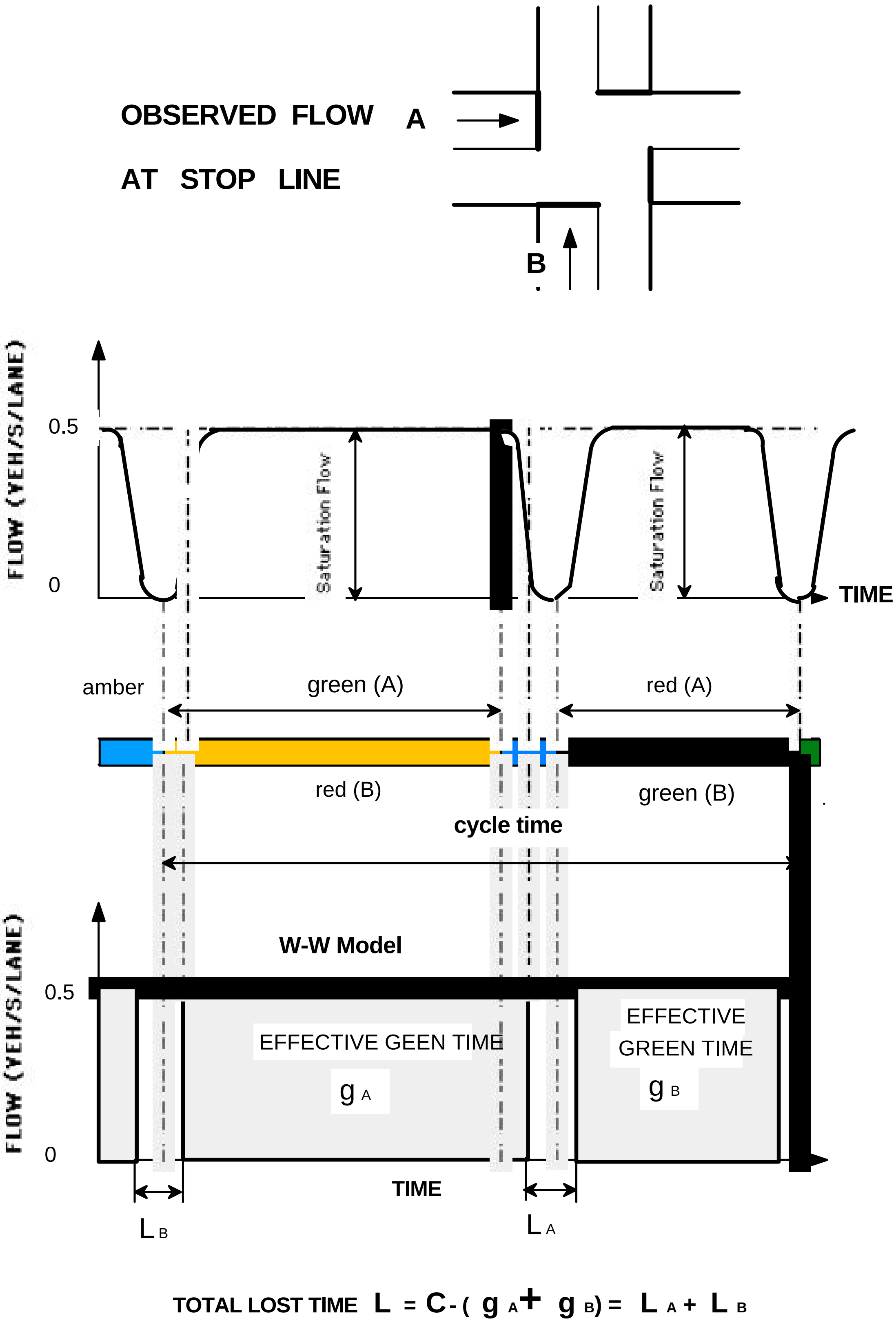


Figure 7.2: Basic Wardrop-Webster model for a junction

The Wardrop-Webster model provides the basis of all traffic signal calculations. There are, however, some known deficiencies in it, such as:

(a) opposed turn lanes with no separate turning phase allowed;

(b) a shared lane, i.e. one used by more than one movement (e.g. a lane shared by through traffic and turning traffic), and

(c) parking in the vicinity of an intersection, where there may be short lengths of kerbside lane available for traffic flow near the stop lines, but these lanes are then blocked further away from the intersection.

A number of modifications and extensions to the basic theory have been made to overcome these problems (Akçelik, 1981), whilst software packages such as SIDRA extend the model further, to cater for cases involving shared lanes and multiple movement phases.

7.3 Capacity of one movement

The capacity of a movement is the maximum number of vehicle that can be discharged through that movement over an extended period of time. The maximum number of vehicles than can be discharged for one movement per cycle (of length c) is equal to the area of the flow rectangle (e.g. $s_A g_A$ for phase A in Figure 7.2 is the total number of vehicles that can be discharged from approach A during one cycle). Capacity is normally expressed as a flow rate (in veh/s or veh/h) so that, in general, the capacity (C_i) of movement i is given as

$$C_i = \frac{s_i g_i}{c} = s_i u_i \qquad (7.2)$$

The ratio $u_i = g_i/c$ is the proportion of time that movement i has the green signal. Now consider the relationship between the arrival rate of traffic (the traffic demand) on a movement and the departure rate. If the arrival flow rate for the movement is q_i, then the number of vehicles arriving per cycle is $q_i c$. For the signals to function satisfactorily (i.e. be able to cope with the demand), adequate capacity must be provided to clear the vehicles arriving in one cycle, i.e.

$$C_i = s_i g_i \geq q_i c \qquad (7.3)$$

and relation (7.3) provides a first indication of the minimum required green time.

7.4 Capacity of entire intersection

The overall capacity of the intersection is found by determining the capacities of the critical movement phases in a full cycle. First consider the simple two-phase system at a cross-road, as seen in Figure 7.2, looking at each of the approach roads (or movements) that operate in the signal cycle (Figure 7.3).

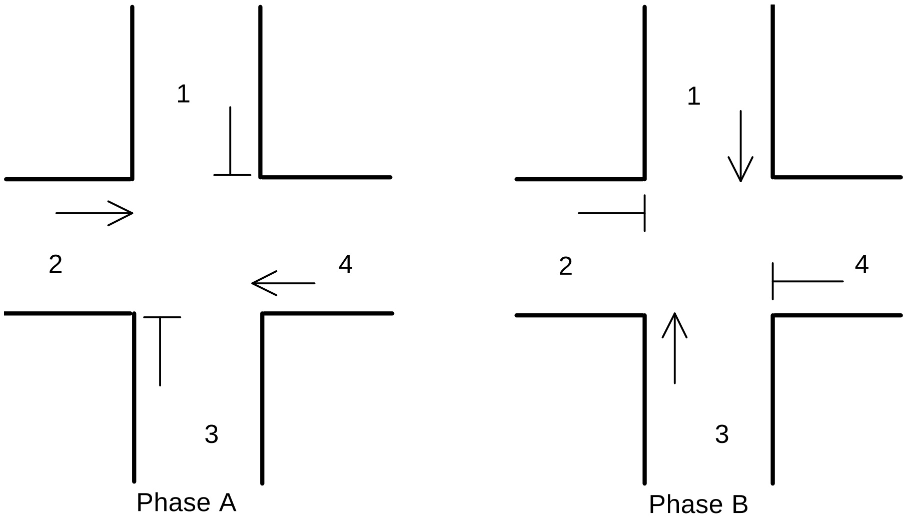


Figure 7.3: Simple phasing arrangement at a cross intersection

Let q_i be the flow on approach (movement) i. The green time for Phase A is g_{24}, and the green time for Phase B is g_{13}. From relation (7.3) we can write the following set of inequalities that must be satisfied if the intersection is to have sufficient capacity to handle the traffic demand:

Phase A:

$$\begin{aligned} s_2 g_{24} &\geq q_2 c \\ s_4 g_{24} &\geq q_4 c \end{aligned} \tag{7.4}$$

Phase B:

$$\begin{aligned} s_1 g_{13} &\geq q_1 c \\ s_3 g_{13} &\geq q_3 c \end{aligned} \tag{7.5}$$

subject to the constraint that

$$c = g_{13} + g_{24} + L \tag{7.6}$$

where L is the total lost time. Note that there are two inequalities to be satisfied in each of the phases A and B. Within each phase, one inequality will dominate, and the other will be redundant. The dominant inequality for a stage defines the critical movement for that stage. This result will apply in the general case, where there will be N movements in any one stage. The critical movement will be the one with the dominant inequality, and there will be (N - 1) redundant inequalities. Define new variables: (1) (the 'y-value') for each movement, where $y_i = q_i/s_i$, and (2) (the 'u-value') for each movement, where $u_i = g_i/c$, and rewrite relation (7.4) for phase A as:

$$u_A = \frac{g_{24}}{c} \geq y_2 \tag{7.7}$$

$$u_A = \frac{g_{24}}{c} \geq y_4$$

and relation (7.5) for phase B as:

$$u_B = \frac{g_{13}}{c} \geq y_1$$

$$u_B = \frac{g_{13}}{c} \geq y_3 \qquad (7.8)$$

Then select the largest y-value for each phase as the representative value for the phase, and this identifies the critical movement, i.e. $y_A = \max\{y_2, y_4\}$ and $y_B = \max\{y_1, y_3\}$. Providing adequate capacity for each critical movement will also provide enough capacity for all of the other movements in each stage. Note that the critical movement is not necessarily the one with the heaviest traffic flow, it is the ratio of traffic flow to saturation flow that is important. The constraint equation (7.6) may now be rewritten as

$$c = g_{13} + g_{24} + L \geq L + c(y_A + y_B)$$

which may be rearranged into

$$c \geq \frac{L}{1-(y_A + y_B)}$$

In general, when there are n phases in a cycle, the *minimum practical cycle time* is given by

$$c \geq \frac{L}{1-\sum_i y_i} = \frac{L}{1-Y} \qquad (7.9)$$

where $Y = \Sigma y_i$. Now, $Y \leq 1$, otherwise it will not be possible to find signal settings that provide sufficient capacity for the intersection. Rearranging relation (7.9) yields

$$Y \leq 1 - \frac{L}{c}$$

which suggests that Y will always be less than one, for L is always positive, never zero. In practice $Y \leq 0.75$, otherwise operation of the intersection will be unsatisfactory. The higher the value of Y, the higher the degree of saturation, and hence the greater the delays expected. A number of measures might be considered to reduce an excessively high Y-value, by reducing the y-value(s) on one (or more) movements.

7.5 Degree of saturation

The degree of saturation of a movement (x_i) and of the intersection (x_p) are the fundamental design parameters in traffic signals design, and are widely used as measures of intersection performance. The movement degree of saturation is defined as the ratio of the traffic demand on the movement (the number of vehicles arriving a cycle) to the capacity of the movement (the maximum number of vehicles that could be discharged during the green period in the cycle):

$$x_i = \frac{q_i}{C_i} = \frac{q_i c}{s_i g_i} = \frac{y_i}{u_i} \qquad (7.10)$$

and the intersection degree of saturation $x_p = \max\{x_i\}$, i.e. the maximum of the movement degrees of saturation. A maximum value of $x_p \geq 0.9$ is often used in intersection design.

7.6 Saturation flows

The saturation flow is the maximum instantaneous flow rate (possible during green time only). The primary factors affecting saturation flow include the environment class of the intersection, lane type, lane width, gradient and traffic composition. Australian practice (Akçelik, 1981) is to estimate saturation flow using the following steps:

(1) for each lane allocated to a given movement, choose a base saturation flow (s_b) value on the basis of environment class and lane type (see Table 7.1). This base saturation flow will be in units of *through car units per hour* (tcu/h);

(2) adjust this base saturation flow to allow for the various factors that impinge on the particular movement (these will include effects of intersection geometry, gradient and traffic composition), to obtain an estimate of lane saturation flow in units of *vehicle per hour* (veh/h), and

(3) add the lane saturation flows to determine the total saturation flow of the movement.

Table 7.1
Base lane saturation flows (s_b tcu/h) for Australian conditions

Environment	Lane Type		
Class	Type 1	Type 2	Type 3
A	1850	1810	1700
B	1700	1670	1570
C	1580	1550	1270

Table 7.1 shows the values of base saturation flows. The two dimensions of the table are the *intersection environment class* and the *lane type*. Three classes of intersection environment are defined:

Class A: *ideal* or near ideal conditions for free movement of vehicles on both approach and departures sides of the movements, good visibility, little interference from pedestrians or parked vehicles. This environment is typically that occurring in a suburban residential or parkland setting.

Class B: *average* conditions, which means adequate intersection geometry, small to moderate numbers of pedestrians, some interference by parked vehicles or goods vehicles. These conditions might be expected in an industrial or shopping area.

Class C: *poor* conditions, involving large numbers of pedestrians and considerable interference from parked vehicles, goods deliveries and perhaps buses. Restrictions on visibility could also be expected. Such conditions are typical of central city precincts.

Lane type is also defined as three categories:

Type 1: *through lane*, containing only through vehicles

Type 2: *turning lane*, containing any type of turning traffic (exclusive left turn or right turn lane, or a shared lane from which vehicles may turn left or right or continue straight through. Adequate turning radius and negligible pedestrian interference to turning vehicles

Type 3: restricted lane, as for Type 2, except that turning vehicles face a small turning radius or some pedestrian interference.

The base saturation flows from Table 7.1 need to be adjusted to fit the particular characteristics of a given intersection, to allow for factors such as lane width, gradient and traffic composition. Adjustments are made by the selection of values of the *adjustment factors*:

(a) f_w, the lane width adjustment factor, given by $f_w = 1$ if $3.0 \le w \le 3.7$, $f_w = 0.55 + 0.14w$ if $w < 3.0$, and $f_w = 0.83 + 0.05w$ if $w > 3.7$, where w is the lane width in metres.

(b) f_g, the gradient adjustment factor, defined by

$$f_G = 1 \pm \frac{G_r}{200}$$

where G_r is the per cent gradient: $+G_r$ for downhill gradients, to increase the saturation flow, and $-G_r$ for uphill gradients, to decrease the saturation flow, and

(c) f_c, the traffic composition adjustment factor, defined by

$$f_c = \frac{1}{q}\sum_m e_m q_m$$

where q_m is the flow rate (veh/h) in the movement for turn or vehicle type m, q (veh/h) is the total vehicle flow rate for the movement, i.e. $q = \Sigma q_m$, and e_m is the *through car equivalent* for turn or vehicle type m. The 'unit' for e_m is tcu/veh, i.e. the number of tcu's equivalent to a single type m vehicle. The through-car-equivalent factors are given as:

(a) *opposed* left turning cars, $e_{LT} = 1.25$;

(b) *unopposed* left turn, $e_{ULT} = 1.00$;

(c) *unopposed* right turning cars, $e_{URT} = 1.00$ [except for a one-way street where the value of $e_{URT} = 1.25$, and

(d) *opposed* right turning cars. This requires knowledge of the opposing through movement and the amount of green time that may be available. The opposed right turn equivalent e_{RT} is given by

$$e_{RT} = \frac{0.5g}{s_u g_u + n_f}$$

where q is the green time for the movement with the opposed turn, s_u is the opposed turn saturation flow (see Akçelik (1981) for methods of estimating s_u), and g_u is the *unsaturated* part of the opposing movement green time. This time is given by

$$g_u = \frac{sg - qc}{s - q}$$

The term n_f represents the number of turning vehicles from the movement who depart the movement *after* the green time, and

(e) trucks and buses, $e_t = 2.0$ cars/truck, except that $e_t = e_{RT} + 1$ for an opposed right turn.

The adjusted saturation flow for a lane or movement (s veh/h or veh/s) is then given by:

$$s = \frac{f_w f_G}{f_c} s_b \tag{7.11}$$

The saturation flows for each lane in a movement are then summed to yield the total saturation flow.

7.7 Measures of intersection performance

A number of measures of performance can be defined to help assess the worth of a particular signal design. These fall into two broad classes:

(a) the performance of traffic using the intersection (e.g. the delays experienced by the traffic), and

(b) the performance of the traffic control system and the junction itself (e.g. the amount of idle time, such as time that any one movement has the green but there is no traffic to take advantage of it).

An assessment of performance in terms of traffic behaviour may include considerations of: delay time (e.g. stopped delay, time in queue, and maximum delay), number of stops, queue length, pedestrian delay, fuel usage and pollutant emissions, and ‘overflow’ queuing (the presence of a residual queue at the end of a *green* period). Sometimes, composite measures of performance are used, for example traffic signals design and analysis software package SIDRA (Akçelik, 1985) provides a *Performance Index* combining delays, number of stops, fuel consumption and queue length.

In terms of intersection performance, important measures include: the degree of saturation (x_p), intersection flow ratio (Y), intersection green time ratio ($U = \Sigma u_i = G/c$, where $G = \Sigma g_i$ is the total amount of green time for the intersection), and cycle failure rate (the proportion of cycles that do not clear queues). Queue length is an important performance measure, e.g. if queues build up so far that they begin to block neighbouring junctions. Queue management is of increasing importance in urban traffic control.

7.7.1 Optimal cycle time

Equation (7.9) defined a practical cycle time, by indicating the minimum cycle time required to provide adequate capacity for the intersection. If optimum performance of an isolated intersection is sought, e.g. for minimum delay, fuel consumption or vehicle operating cost, then Akçelik (1981) indicates that the optimum cycle time (c_o) is given by

$$c_o = \frac{(1.4 + k)L + 6}{1 - Y} \tag{7.12}$$

where k = 0.4 for minimum fuel consumption, k = 0.2 for minimum operating cost, and k = 0 for minimum delay. Note that a different optimum cycle time can exist for each of these measures of performance. A further operational constraint on cycle time is that c should not be too long. Excessive cycle times (e.g. 150 seconds or longer) may cause drivers to suspect a fault with the signals, and then to start to disregard them!

7.7.2 Queue length and delay

The formation of queues (and thus the imposition of delay on an least some of the traffic) is an integral part of signalisation. Each cycle must contain some time when each movement is prevented from travelling through the intersection, although traffic may continue to arrive to make that movement. Thus queues build up. Further, once the signal changes to green, only the vehicles at the head of the queue can start moving. A finite amount of time that will elapse before vehicles at the back of the queue can move off. New arrivals during the green also have to join the back of the queue, unless this has cleared. The trajectory diagram in Figure 7.4 provides a simple illustration of this process.

Consider the trajectory of vehicle I in Figure 7.4. This vehicle drives up to the intersection during the red time (r_i) for the movement, where r_i is indicated by the thick horizontal line AC. Vehicle I joins the end of the existing queue. When the signal changes to green, the queue starts to move, with vehicles being discharged at a constant rate (the saturation flow.) Thus vehicle I starts to move forward some time after the start of the green phase. The horizontal line segment in the trajectory for vehicle I is the delay time experienced by that vehicle, and the total area of the *delay triangle* ACB in Figure 7.4 is the total delay experienced by all of the vehicles in the queue. The queue length at the start of the green period is $N_i = r_i q_i$.

Now, Figure 7.4 indicates the continued existence of the queue some time after the end of the red period. For instance, vehicle J arrives after the start of the green period, but still has to join the queue. The total number of vehicles that pass through the queue formed as a result of the red period is N_{mi}, which is known as the *back of queue*:

$$N_{mi} = \frac{N_i}{1 - y_i} \qquad (7.13)$$

An alternative representation of the queuing process at a signal is given in Figure 7.5. The top part of this figure shows the flow rate of arrivals and departures over time, the bottom part shows the cumulative numbers of arrivals and departures. The queue length N(t) at time t is given by the difference between the cumulative number of arrivals and the cumulative number of departures. Thus N_i is the vertical distance between the two cumulative flow lines at the start of the green period. Notice the step form of the departure flow in Figure 4.7. This indicates that the movement is under-saturated ($x_i < 1$). Discharge at the saturation flow rate only continues until the queue has cleared. Once the queue has cleared, the subsequent arrivals to the movement during the remainder of the green proceed straight through the intersection, and the departure rate for that residual green period is equal to the arrival rate q_i.

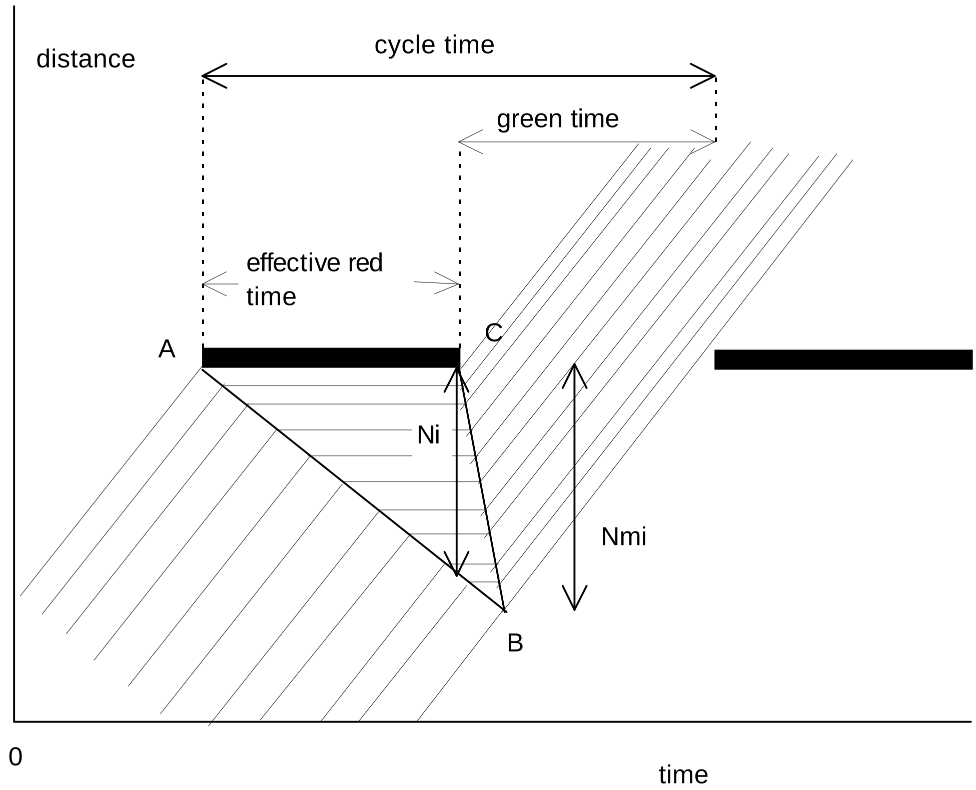


Figure 7.4: Trajectory diagram of arrivals and departures for one movement

In the limit (as $x_i \rightarrow 1$), the time at which the queue clears approaches the finish of the green period. The point of time during the green interval at which the queue will clear can be obtained by equating the areas of rectangles OABC and FEDB in Figure 7.5. The back of queue is given by

$$N_{mi} = N_i + \Delta N$$

Total delay time over the time interval 0 to t incurred by vehicles on the movement is given by

$$D = \int_0^t N(z)dz$$

The mean uniform delay per vehicle (for uniform arrivals and the Wardrop-Webster model applied to an isolated intersection) is then

$$d_u = \frac{r_i^2}{2c(1 - y_i)} \tag{7.14}$$

Coordination between signals along a road can be used to modify the mean uniform delay, and opens up the possibility to reduce this significant component of the total delay. A model for estimating the uniform delay under signal coordination is presented in Chapter 10.

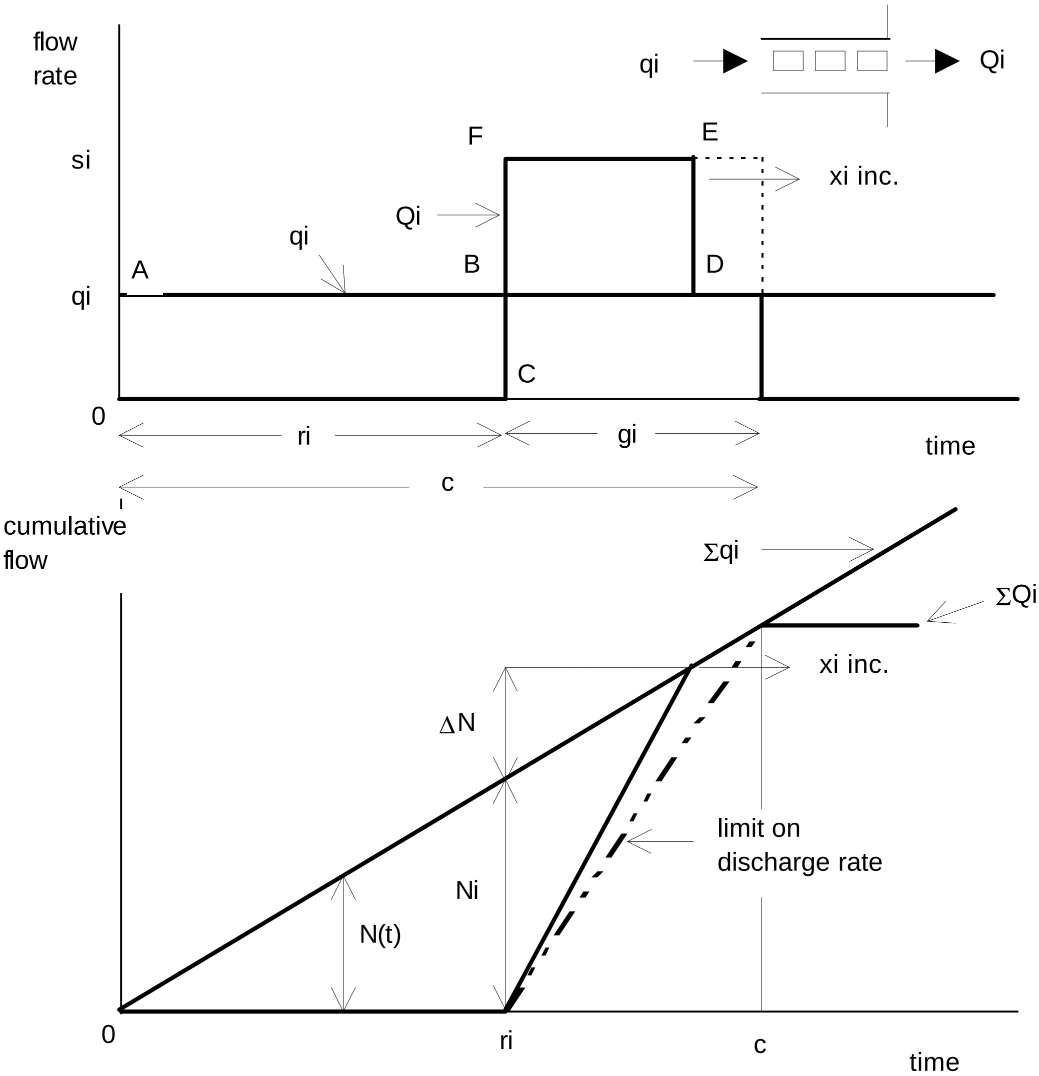


Figure 7.5: Queue formation by comparison of arrival and discharge rates

This average delay term (d_u) is an *under-estimate* of the total real world delay as it does not account for random fluctuations in the arrival rate.

A component representing 'overflow' delay (d_f) to account for these fluctuations is thus usually added to d_u, to yield a total mean delay (d):

$$d = d_u + d_f \tag{7.15}$$

An important consideration in determining f_f is the time period (T_f) over which the traffic demand (q) persists. Average delays are expected to be higher, the longer that high volume demand occurs at an intersection. This results from the differences in volume between peak and off-peak conditions, and the greater opportunity for overflow queues to occur as time period (and hence number of cycles) increases. Akçelik (1981) gave the following expression for d_f

$$d_f = \frac{1}{4} T_f \left\{ (x-1) + \sqrt{(x-1)^2 + \frac{12}{qT_f}\left(x - 0.67 - s\frac{g}{100} \right)} \right\} \qquad (7.16)$$

where s is in veh/h and g is in seconds. Equation (7.16) applies for $x \geq 0.7$. These delay expressions are applied in TrafikPlan to each movement at a signalised intersection.

7.7.3 Number of stops

The average number of stops per vehicle for a movement, including both vehicles that join the queue or are able to pass unimpeded, is given by h_i where

$$h_i = \frac{N_{mi}}{cq_i} = \frac{1-u_i}{1-y_i}$$

Now this is a theoretical result and in practice many drivers will avoid coming to a full stop by decelerating well before reaching the back of the queue, so that they never quite come to rest. Thus Akçelik (1981) suggested applying a correction factor to cope with these ‘partial stops’. This correction is:

$$h_i = \frac{0.9(1-u_i)}{1-y_i} \qquad (7.17)$$

7.8 Implementation in TrafikPlan

The basic traffic signals capacity and timing model developed by Akçelik (1981) and described in this chapter is implemeted in TrafikPlan.

8. MODELLING FUEL CONSUMPTION AND EMISSIONS

The analysis of travel times and delays (see Chapters 6 and 7) can be extended to include models for the estimation of fuel consumption and air pollutant emissions by traffic along a route, if the types of vehicles in the traffic streams are known. This permits the inclusion of environmental impacts into a traffic analysis. The following set of models is used in TrafikPlan.

8.1 Segmentation of vehicle flows

Changing fleet composition and the contributions of different vehicle types and trip classes to fuel usage and pollution are important in transport planning and road traffic flow analysis, e.g. to see how such changes might affect pollution levels. [The differences in energy and environmental performance between pre-1986 and post-1986 Australian vehicles is one such issue. Trip class might include different categories of travellers, e.g. through traffic and local traffic, private, commercial and business travel, etc.] If q(e) is the total vehicle volume on link e then

$$q(e) = \sum_k q_k(e) \qquad (8.1)$$

where $q_k(e)$ is the volume of trip class k vehicles on e. If p_{km} is the proportion of type m vehicles in trip class k then the flow $q_m(e)$ of type m vehicles is given by equation (8.2):

$$q_m(e) = \sum_k p_{km} q_k(e) \qquad (8.2)$$

It therefore follows that if $E_m(X)$ is the mean rate (per unit length) of emission (consumption) of pollutant (fuel) X by a type m vehicle then $TE_e(X)$, the total rate of emission (consumption) of X on link e is given by equation (8.3).

$$TE_e(X) = \sum_{km} E_m(X) p_{km} q_k(e) \qquad (8.3)$$

In the common situation where trip class data are not readily available or cannot be accommodated in the computations, then an equivalent formulation can be used

$$TE_e(X) = \sum_{km} E_m(X) p_m q(e) \qquad (8.4)$$

where p_m is the proportion of type m vehicles in the traffic stream.

Thus given models to predict $E_m(X)$ for a range of traffic conditions then total pollution loads and fuel consumption can be estimated. These models will have the ability to suggest differences in energy and environmental impacts for changes in levels of traffic flow and congestion and for changes in vehicle fleet composition. Until recently only limited data (for a restricted number of vehicle types) has been available (Akçelik, 1990), but recent research at

TSC has enlarged the database of available vehicle types (e.g. Taylor and Young (1996ab), including data for unleaded petrol (ULP) cars.

8.2 Emission/consumption models for traffic streams

Four levels of fuel consumption and emissions modelling were proposed by Biggs and Akçelik (1986). Their models are:

(a) an *instantaneous model*, that indicates the rate of fuel usage or pollutant emission of an individual vehicle continuously over time;
(b) an *elemental model*, that relates fuel use or pollutant emission to traffic variables such as deceleration, acceleration, idling and cruising, etc. over a short road distance (e.g. the approach to an intersection);
(c) a *running speed model*, that gives emissions or fuel consumption for vehicles travelling over an extended length of road (perhaps representing a network link), and
(d) an *average speed model*, that indicates level of emissions or fuel consumption over an entire journey.

The instantaneous model is the basic (and most detailed) model. The other models are aggregations of this model, and require less and less information but are also increasingly less accurate. The running speed model is suitable for application in strategic networks, for it can be used at the network link level.

8.2.1 Instantaneous model

This model is suitable for the detailed assessment of traffic management schemes for individual intersections or sections of road. It may be used for comparisons of the behaviour of individual vehicles under different traffic conditions. The variables in the model include instantaneous values such as speed v(t) and acceleration a(t) at time t. The instantaneous model gives the rate of emission/consumption (E/C) of X, including components for:

(a) the fuel used or emissions generated in maintaining engine operation, estimated by the idle rate (a);
(b) the work done by the vehicle engine to move the vehicle, and
(c) the product of energy and acceleration during periods of positive acceleration.

The energy consumed in moving the vehicle is further divided into drag, inertial and grade components. Part (c) allows for the inefficient use of fuel during periods of hard acceleration. The model is

$$\frac{dE(X)}{dt} = a + b_1 R_T v + \left[\frac{b_2 M a^2 v}{1000}\right]_{a>0} \qquad \text{for } R_T > 0$$

$$\frac{dE(X)}{dt} = a \qquad \text{for } R_T \leq 0 \qquad (8.5)$$

where v = speed (ms^{-1}),
a = instantaneous acceleration in ms^{-2},

R_T	=	total tractive force required to drive the vehicle, which is the sum of the drag, inertial and grade forces
M	=	vehicle mass in kg;
a	=	idling fuel consumption or pollutant emission rate;
b_1	=	engine efficiency parameter (mL or g per kJ), relating E/C to energy provided by the engine, and
b_2	=	engine efficiency parameter (mL or g per (kJ.ms^{-2})) relating E/C during positive acceleration to the product of inertia energy and acceleration.

R_T is given by

$$R_T = b_1 + b_2 v^2 + \frac{Ma}{1000} + gMG \times 10^{-5} \qquad (8.6)$$

where	g	=	gravitational acceleration in ms^{-2};
	G	=	percentage gradient (negative downhill);
	b_1	=	drag force parameter relating *mainly* to rolling resistance, and
	b_2	=	drag force parameter relating *mainly* to aerodynamic resistance.

Both of the drag force parameters also reflect some component of internal engine drag. The model has been found to estimate the fuel consumption of individual vehicles to within five per cent (Biggs and Akçelik, 1986). Its accuracy for emissions modelling remains to be established but a similar level could be expected. The five parameters a, b_1, b_2, b_1 and b_2 are specific to a particular vehicle, and the idling rate and energy efficiency parameters (a, b_1 and b_2) depend on the type of fuel or emission as well.

8.2.2 Elemental model

The most suitable model for estimating fuel consumption and emissions of traffic at an intersection or on a road section is the elemental model. This model is included in the TrafikPlan package. It considers the trajectories of vehicles traversing the section. It can be used to estimate the additional emissions or fuel usage incurred compared to the case of an equivalent road section without intersection or traffic control device. This is done by considering the speed-time profile of vehicles using the section, and describing this profile in terms of the following five elements (hence the name of the model):

(1) cruising, the vehicle enters the road section at a constant speed;
(2) deceleration, the vehicle has to brake to join the back of a queue;
(3) idling, the vehicle waits in the queue with engine idling;
(4) acceleration, the vehicle accelerates as the queue moves off; and
(5) cruising, the vehicle resumes cruising as it leaves the section.

The elemental model thus considers the incremental effects of delays, queuing and numbers of stops and starts due to the traffic controls, for a defined section of road. The required input data include cruise speed (v_c), number of stops, stopped time (T_i), road section distance (x_s) and average gradient of the road over the section prior to, and after, the intersection.

The total volume of fuel consumed or pollutant emitted per vehicle over the section ($E_s(X)$) is composed of the consumption or emission over the cruise-deceleration-idle-acceleration-cruise cycle. The model is constructed by summing the fuel consumption or pollutant emission in each element of this cycle:

$$E_s(X) = f_{c1}(x_{s1} - x_d) + F_d + aT_i + F_a + f_{c2}(x_{s2} - x_a) \qquad (8.7)$$

where f_{c1}, and f_{c2} are cruise E/C rates per unit distance for the initial and final cruise speeds v_{c1} and v_{c2}; x_{s1}, and x_{s2} are section distances on approach and departure, respectively; x_d, and x_a are deceleration and acceleration distances, respectively; F_d, and F_a are the total deceleration and acceleration E/C, respectively; α is the idle E/C rate; and T_i is the idle or stopped time (sec). Taylor and Young (1996ab) provide parameter values for the model.

The elemental model provides estimates of fuel consumption within ten per cent of observed values. Indeed, given that it is computationally easier to apply than the instantaneous model, its performance is commensurate with its more detailed cousin. The elemental model is included in TrafikPlan, where the focus is generally on the fuel and emissions effects of an element of the road system (e.g. an intersection) rather than on those of individual vehicles traversing that element.

8.2.3 Running speed model

This model may be used for estimation of fuel consumption or emissions along a network link, and is thus the most suitable model for application in a transport network model. The data required to apply the model are travel time T_s (seconds), trip distance x_s (km), and stopped time T_i (seconds) over the route section. The vehicle is then assumed to travel at a constant running speed v_r (km/h), where

$$v_r = \frac{3600 x_s}{T_s - T_i} \qquad (8.8)$$

while moving. The model predicts the mean rate of pollution emission or fuel consumption E_s (g or mL per km per vehicle) as

$$E_s = \max\left\{ f_r + \frac{\alpha T_i}{x_s}, \frac{\alpha T_i}{x_s} \right\} \qquad (8.9)$$

where

f_r = fuel consumption or pollutant emission per unit distance (mL/km or g/km) excluding stopped time effects (i.e. while cruising at constant speed v_r), and is given by

$$f_r = \frac{3600\alpha}{v_r} + A + Bv_r^2 + k_{E1}\beta_1 + ME_{k+} + k_{E2}\beta_2 ME_{k+}^2 + gk_G\beta_1 M\frac{G}{100} \qquad (8.10)$$

E_{k+} = sum of *positive* kinetic energy changes per unit mass per unit distance along the road section (ms^{-2}), which may be estimated from

$$E_{k+} = \max\{0.35 - 0.0025 v_r, 0.15\} \qquad (8.11)$$

as described by Bowyer, Akçelik and Biggs (1986). The calibration parameters k_{E1}, k_{E2} and k_G may be estimated from

$$k_{E1} = \max\left\{0.675 - \frac{1.22}{v_r}, 0.5\right\} \quad (8.12)$$

$$k_{E2} = 2.78 + 0.0178 v_r \quad (8.13)$$

$$k_G = 1 - 1.33 E_{k+} \quad \text{for } G < 0$$
$$k_G = 0.9 \quad \text{for } G > 0 \quad (8.14)$$

A prediction of running speed is needed to complete this link-based model of emissions and consumption, and if this cannot be observed directly then [from Bowyer, Biggs and Akçelik (1986)] an estimate of the running speed v_r (km/h) may be made from equation (8.15), given knowledge of the overall average link travel speed v_s (km/h).

$$v_r = \max\{8.1 + 1.14 v_s - 0.00274 v_s, v_s\} \quad (8.15)$$

This model provides estimates of fuel consumption within 10-15 per cent of observed values for travel over road sections of at least 0.7 km (Biggs and Akçelik, 1986). Road gradient plays a major role in determining the accuracy because of the non-compensatory effects of positive and negative gradients. Longer section lengths will give improved accuracy. The accuracy of this formula for emissions modelling remains to be determined, but is the subject of current research.

8.2.4 Journey speed model

This model is useful for estimating total fuel consumption or emissions over long journeys in large networks, or when no explicit node-link network description is being used (e.g. in a sketch planning application). It would be apply to impact assessment for a regional transport systems management scheme likely to affect mean travel speeds, door-to-door travel times, and the level of traffic demand. It is most applicable for situations in which mean door-to-door travel speeds are 50 km/h or less. Mean travel speed is total travel distance divided by total travel time. The data required are travel distance x_s, and either total travel time T_s or mean speed v_s. The average speed model relates f_x, the mean consumption or emission rate per unit distance, to the mean travel speed, as follows:

$$f_x = \frac{f_i}{v_s} + b \quad (8.16)$$

where f_i is the idle E/C rate (= 3600α if v_s is in km/h) and b is a composite parameter accounting for the drag, inertial and grade components of the E/C rate, averaged over the whole journey, written as

$$b = A + B v_r^2 + k_{E1} b_1 M E_{k+} + k_{E2} b_2 M E_{k+}^2 + g k_G b_1 M \overline{G} \quad (8.17)$$

where $\overline{G}$ is the mean gradient over the journey and the other parameters are as defined above. The difference between the running speed model and the average speed model is that the latter is a simplified model which assumes that the energy terms contribute a constant amount to fuel consumption and emissions per unit distance. This assumption is valid at low speeds (less than 50 km/h), but breaks down at higher speeds where the v_r^2 term dominates. Thus the average speed model tends to underestimate E/C rates at higher speeds. Suitable accuracy in the higher speed range can be achieved by using the running speed model with v_r estimated using equation (8.15).

8.3 Model implementation in TrafikPlan

The running speed model is presently included with TrafikPlan, using the model defined by equations (8.8) - (8.15), with parameter values as given in Taylor (1996) and Taylor and Young (1996ab). The current implementation of TrafikPlan can provide reports on the consumption of leaded and unleaded petrol, and on the emissions of carbon monoxide and carbon dioxide.

A set of traffic performance parameters for evaluation of different speed limits is given in Chapter 13. Consumption of unleaded petrol and emissions of carbon monoxide are included amongst these performance factors.

9. AREA-WIDE TRAFFIC CONTROL

Modern traffic control systems such as the SCATS (and its localised variants), TRACS and BLISS systems presently used in Australia, attempt to provide for free-flowing traffic (or at least minimum disruption) on major traffic routes. Linking of the signals at successive junctions (i.e. so that the start of the green periods at successive signals are coordinated to allow for the free movement of platoons) is one way to minimise delays and stop-start driving. The purpose of a coordinated signal system is to allow the maximum volume of traffic to pass without stopping, while catering for the demands of cross street traffic. Further, coordination can be used to prevent queues from extending back to interfere with upstream intersection movements.

The basics of signal coordination are shown in Figure 9.1. Platoons of vehicles are aimed at each 'green window'. Signals at intersection no. 2 turn green ϕ_{12} seconds after those at intersection no. 1. This time is known as the *offset* between the signals. There is constant (critical) cycle time for the system (the *common cycle time*), which is set to provide sufficient capacity at the *critical intersection* in the group. The green times at all intersections are set to provide adequate capacity and to give opportunity for free passage of the platoons. A coordination cruise speed (tanθ in Figure 9.1) is determined as a design parameter - this speed will be closely related to the speed limit. The essence of coordination is to keep platoons together but the nature of the driving task is for platoons to break up.

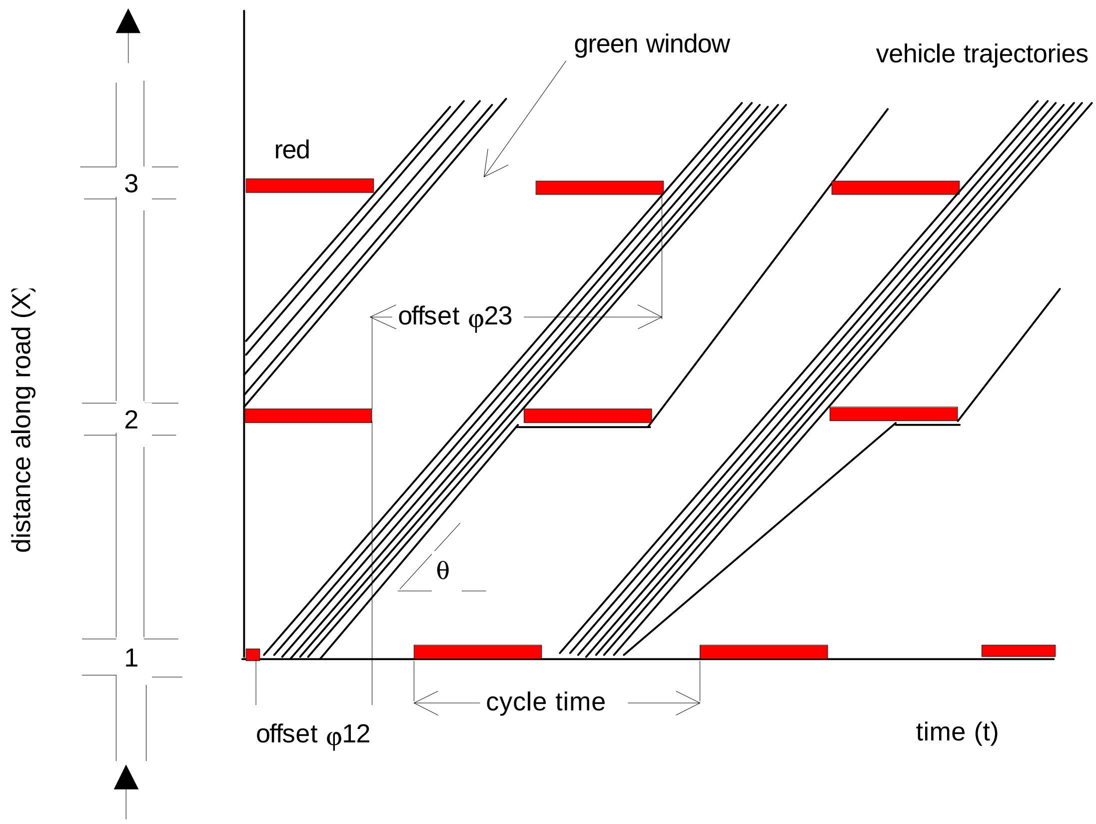


Figure 9.1: Signal coordination principles and the 'green wave'

9.1 Platoon dispersion

A platoon is a cluster of vehicles, travelling at short headways and moving at about the same speed. Platoons form naturally in the process of vehicles moving away from a standing queue, as happens at a traffic signal. Once formed, the natural tendency of the platoon is to disperse. The headways between vehicles in the platoon will tend to increase as the speed of the platoon increases, so that the size of the platoon grows. In addition, some drivers and vehicles may be unable or unwilling to maintain speed at the level set by the platoon leaders. Thus as the platoon moves down the road towards the next set of signals it will occupy an increasing physical length of road and an increasing time band. This is the process of platoon dispersion. Dispersion makes the task of setting signal offsets for traffic control more difficult. Methods are needed to estimate the size of the platoon (i.e. the physical space it occupies) as it disperses so that offsets and green times at downstream signals can be designed to accommodate the bulk of, if not all of, the platoon.

The most common method for modelling platoon dispersion is Robertson's recurrence formula. This is an integral part of the TRANSYT computer model (Robertson, 1969) for designing network-wide signal settings. The method is based on an assumed relationship between the flow passing a point in one time interval and a previous time interval. This relationship is

$$q_2(i+BT) = Fq_1 + (1-F)q_2(i+BT-1) \qquad (9.1)$$

where $q_2(i)$ is the predicted flow downstream in the ith time interval, $q_1(i)$ is the flow of the initial platoon in the ith time interval, B is the 'travel time factor' (the ratio of the platoon leader travel time to the average travel time of the entire platoon), T is the average travel time of the entire platoon, over the distance for which the platoon dispersion is being calculated, and F is a smoothing factor. The value of F is given by

$$F = \frac{1}{A+BT} \qquad (9.2)$$

where A is a dispersion factor to account for the degree of platoon dispersion. A and B are empirical parameters, ranging from 0 to 1. If A = 0, then no dispersion occurs. If B is constant and A increases from zero, the level of dispersion increases. Values of A and B may be determined from field data, but this is often difficult. Typical values used as defaults are A = 0.5 and B = 0.8.

Other models for platoon dispersion exist, and the interested reader is referred to Seddon (1972), Tracz (1975), Axhausen (1987) and Young, Taylor and Gipps (1989).

Consideration of platoon dispersion in terms of a relatively simple model such as that described above can then be used to formulate analytical models of traffic flow under signal coordination. Gartner, Little and Gabbay (1975) developed a model of signal coordination that can be used to determine optimal offsets. Their model, known as the GLB model, is described in Chapter 10. It is implemented in TrafikPlan to account for area-wide traffic control.

10. THE GLB MODEL OF ARTERIAL ROAD SIGNAL COORDINATION

A useful mathematical model of traffic progression along a road with signal progression was developed by Gartner, Little and Gabbay (1975). This model, described herein as the GLB model, was subsequently incorporated into the TrafikPlan dense network traffic model (e.g. Young, Taylor and Gipps (1989), Taylor (1992a)) and provides the basis for an analytical model of arterial road traffic flow that can accommodate alternative speed limit regimes.

10.1 Definitions

Consider an arterial road made up of a connected set of road links, where link e = (i, j) is the directional link connecting intersection node i to node j. Traffic flow along the road is in platoons of vehicles, and the nodes (i, j) are signalised junctions. For the link e = (i, j), define the traffic control factors

X_{ij} = the distance between node i and node j

T_{ij} = travel time of a platoon leader between i and j

V_{ij} = average travel speed of a platoon leader between i and j, given by X_{ij}/ T_{ij}

c = common cycle time for the signals, i...e. the cycle time applying at each node

g_{ij} = the effective green time on the approach leg e =(i, j) at node j

r_{ij} = the effective red time for the approach leg e = (i, j) at node j, equal to $c - g_{ij}$

ϕ_{ij} = the signal offset between i and j, i.e. the time interval between the start of a green phase at node j relative to the start of the green phase at node i

γ_{ij} = the arrival time of the platoon leader at node j relative to the start of the green at j

It then follows that

$$f_{ij} = T_{ij} - g_{ij} \tag{10.1}$$

Further, it is only necessary to consider $- r_{ij} \leq \gamma_{ij} \leq g_{ij}$ as the process is cyclic, with period c.

The travel time T_{ij} is determined by the average speed of travel along the link (i, j), i.e. V_{ij}, which is in turn set by the speed limit on the road link.

10.2 Performance functions

A link performance $Z_e = Z(\phi_{ij})$ can be defined, based on mean delay time, travel time, stop rate, fuel consumption or pollutant emissions. This function provides a measure of the level of performance of the link under a given set of traffic and traffic control parameters. An overall route performance function (Z_R) may then be constructed, by summing the individual link performance functions for the route:

$$Z_R = \sum_{e=1}^{n} Z_e = \sum_{j=1}^{n} Z(f_{ij}) \quad (10.2)$$

Now consider some traffic flow relationships for the link.

Let $q_{ij}(t)$ be the arrival rate of vehicles on the link at time t, where $-r_{ij} \le t \le g_{ij}$.

$A_{ij}(t)$, the cumulative number of arrivals between the start of the red time and time t is then given by

$$A_{ij}(t) = \int_{-r_{ij}}^{t} q_{ij}(t)dt \quad (10.3)$$

Then $A_c = A(g_{ij})$ is the total number of arrivals in one cycle. Let s_{ij} be the saturation flow for the approach (i, j). The operation of the signals at each node (j) can be described using the established methods described in Akçelik (1981), with the following assumptions applied:

- traffic arrivals are periodic, so that $q(t) = q(t \pm c)$
- the analysis is for free-flowing traffic conditions, and thus the signals are undersaturated. This implies that $A_c \le g_{ij}s_{ij}$
- the arrival traffic flow rate never exceeds the saturation flow rate s_{ij}, i.e $q_{ij}(t) \le s_{ij}$
- at some point in the past, a zero queue has occurred on the approach (i, j)

The queue length $N_{ij}(t)$ on the link is given by:

$$\begin{aligned} N_{ij}(t) &= A_{ij}(t) && -r_{ij} \le t < 0 \\ N_{ij}(t) &= A_{ij}(t) - ts_{ij} && 0 \le t < t_o \\ N_{ij}(t) &= 0 && t_o \le t \le g_{ij} \end{aligned} \quad (10.4)$$

where t_o is the point during the green time when the queue disappears. It is given by the solution of

$$A_{ij}(t_0) - t_o s_{ij} = 0 \quad (10.5)$$

The total delay time incurred during one cycle of the signals is thus

$$D_{ij} = \int_{-r_{ij}}^{g_{ij}} N_{ij}(t)dt = \int_{-r_{ij}}^{t_0} N_{ij}(t)dt \quad (10.6)$$

and the mean delay per vehicle (d_{ij}) is thus

$$d_{ij} = \frac{D_{ij}}{A_c} \quad (10.7)$$

10.3 Traffic flow model

The assumed model for traffic arrivals at the start of the link is flow in platoons, which may be described mathematically as a square wave pulse. Thus the traffic flow function $q_{ij}(t)$ is given as a cyclic pulse of length (time duration) p, such that 0 < p < c, and height q_{ij}. Each pulse contains pq_{ij} vehicles. If f is the mean flow rate over a complete cycle, and is made up of a primary component f_1 (from the upstream link of the arterial road) and a secondary component f_2 (turning traffic from side streets along the link), with an upstream green time g_{li}, then the platoon length of the primary flow is

$$p_1 = g_{li}$$

and

$$q_{0ij} = f_1 \frac{c}{g_{li}},$$

i.e. q_{oij} is the equivalent uniform flow rate through the upstream green.

Secondary flow is added with the same height (q_{ij}) as the primary flow, in order to preserve the platoon shape - the effect of secondary flow is to increase the length of the platoon. The additional length required is p_2, where

$$p_2 = f_2 \frac{c}{q_{ij}}$$

for a total platoon length of

$$p_{0ij} = p_1 + p_2 = g_{li}\left(1 + \frac{f_2}{f_1}\right) \qquad (10.8)$$

Note that this formulation assumes $f_1 > f_2$, with the assumption that the arterial road flow dominates the traffic flow on the link concerned. In the case that $f_2 > f_1$, the roles of primary and secondary components may have to be interchanged.

Now, p_{oij} in equation (10.8) refers to the upstream condition. We need value for p and q at the downstream end of the link, i.e. at the approach to node j. To account for platoon dispersion over the link length X_{ij}, we can write

$$p(X_{ij}) = k(X_{ij})\,p_{0ij} \qquad \text{if } k(X_{ij})p_{oij} < c$$

$$(10.9)$$

$$p(X_{ij}) = c \qquad \text{otherwise}$$

The effective flow rate for the platoon at the downstream end of the link, $q(X_{ij})$, is found by applying the principle of conservation of vehicles along the link (i.e. we assume that the vehicular flow along the link is conserved). Thus

$$q(X_{ij}) = \frac{q_{0ij}\,p_{0ij}}{p(X_{ij})} \qquad (10.10)$$

The platoon dispersion factor $k(X_{ij})$ is dependent upon the characteristics of the link, including the number of lanes and other traffic parameters as well as its length X_{ij}. A widely-used empirical model of platoon dispersion (Robertson's model) was described in Section 9.1. Young, Taylor and Gipps (1989) provides a description of some of the alternative formulations for the platoon dispersion factor. A precise definition of the dispersion factor is not required at this stage in model formulation, although it must be noted that platoon length cannot exceed cycle time in a periodic system.

10.4 Link performance function for rectangular platoons

The function $Z(\phi_{ij})$ is non-linear, and can be reduced to a piecewise linear form for use in a signal optimisation problem. Results can be found for the maxima and minima of $Z(\phi_{ij})$, and these may be used as 'anchor points' for an overall performance optimisation.

Delay is minimal when the tail of a platoon arrives at the signal just before the signal turns to red and the platoon can thus just clear the intersection without being broken up. This occurs for

$$\gamma = \bar{\gamma}_{ij} = g_{ij} - p(X_{ij})$$

If

$$y_{ij} = \frac{q_{ij}}{s_{ij}}$$

then Gartner *et al* (1975) showed that

$$\begin{aligned} z_{\min} &= 0 && p(X_{ij}) \le g_{ij} \\ z_{\min} &= \frac{\bar{\gamma}_{ij}^2}{2p(X_{ij})(1-y_{ij})} = \frac{(g_{ij} - p(X_{ij}))^2}{2p(X_{ij})(1-y_{ij})} && p(X_{ij}) > g_{ij} \end{aligned} \qquad (10.11)$$

Maximum delay occurs when the head of the platoon arrives at the stop line immediately after the signal turns red. This occurs for

$$g_{ij} = \tilde{g}_{ij} = -r_{ij}$$

and it then follows that

$$\begin{aligned} z_{\max} &= r_{ij} + \frac{p(X_{ij})(y_{ij} - 1)}{2} && p(X_{ij}) \le t_o + r_{ij} \\ z_{\max} &= \frac{r_{ij}^2}{2p(X_{ij})(1-y_{ij})} && p(X_{ij}) > t_o + r_{ij} \end{aligned} \qquad (10.12)$$

and it may also be noted that the delay for $\gamma_{ij} = g_{ij}$ is the same as for $\gamma_{ij} = -r_{ij}$, which is expected given the cyclic nature of the traffic flow. The value of delay (Z) determined through this model is the uniform delay component applying to coordinated signals - see also Section 7.7.2.

The GLB model is incorporated in TrafikPlan, along with the models for the performance of individual signalised intersections. In this way the computer model is able to analysis traffic networks including the effects on delays and travel times of traffic signal coordination.

A fundamental input parameter for the GLB model is the average speed of the platoon leader, see Section 10.1, and this parameter includes the effects of the posted speed limits on the road links in the signal progression.

10.5 Application of the GLB model

Some examples of the application of the GLB model for signal coordination provide an illustration of the effect of signal offset on traffic progression along an arterial road. Consider the coordination between the signals at two neighbouring intersections. Let these intersections be 1000 m apart, and assume that a common cycle time of 100 seconds applies at each intersection. The saturation flow rate (s, in passenger car units per hour, the maximum instantaneous flow rate across the stop line) for the road connecting the two intersections is 3600 pcu/h. The green time at the downstream intersection (g, in seconds) and the traffic volume (q, pcu/h) on the road may be varied along with the offset, and mean delays calculated for each offset, green time and traffic volume. Comparison of the values of the computed delays for each offset shows the possible effects of signal coordination, and the opportunity to reduce overall travel times by selecting offsets which minimise the delay experienced by the platoons.

Two examples are presented. In the first, green time is varied between 20 s and 50 s for the flow direction considered. A traffic volume of 1080 pcu/h is used in all cases. This means that the y-value for the traffic stream is $y = q/s = 1080/3600 = 0.3$, a value indicating a reasonable level of service for the traffic stream (Akçelik, 1981). A further (and perhaps more useful performance parameter in traffic signals analysis is the degree of saturation (x), defined by equation (7.10), which for this case is equal to 30/g. Thus the degree of saturation varies from 0.6 (g = 50 s) - light to moderate traffic - to 1.5 (g = 20 s) - heavily congested traffic. The peak period is assumed to last for one hour.

Table 10.1 shows the calculated mean delay and its uniform and overflow components for a green time of 30 seconds and a range of offsets from 0 to 100 seconds. As indicated in Section 10.4, the delays are periodic with respect to offset, with a period equal to the cycle time. The table also indicates the expected delays for an isolated signalised intersection under the same traffic conditions and signal settings (expected total delay 41.5 s). It clearly shows that choice of offset leads to considerable variation in expected delays. Poor offset selection may increase the delay above that predicted for the isolated signal case, whereas a range of offsets providing greatly reduced delays is available (e.g. offsets between 50 and 80 seconds for the case shown in Table 10.1).

Figure 10.1 shows the variations in expected total delay for a range of green times (20 to 50 seconds).

Table 10.1
Calculated delays for coordinated signals with c = 100 seconds, g = 30 seconds, s = 3600 pcu/h and q = 1080 pcu/h

Offset (s)	Uniform delay component (s)	Overflow delay component (s)	Total delay (s)
0	32.32	4.58	36.90
10	44.18	4.68	48.76
20	48.92	4.58	53.50
30	34.64	4.58	39.22
40	20.36	4.58	24.95
50	6.09	4.58	10.67
60	0.34	4.58	4.92
70	0.62	4.58	5.20
80	8.61	4.58	13.19
90	20.46	4.58	25.05
100	32.32	4.58	36.90
Isolated signal delays	35.00	6.48	41.48

Similar effects may be seen when traffic volumes vary. Figure 10.2 shows the variations in delays for different traffic volumes (1620, 2160 and 2700 pcu/h) for a green time of 50 seconds. The degrees of saturation for these flows are 0.9, 1.2 and 1.5 respectively, indicating heavy to oversaturated traffic conditions. However, even under the worst traffic condition (q = 2700 pcu/h) there is still a small window of offsets which offers some reduction in delay.

It is this phenomenon that offers some possibilities for improving overall travel times by intelligent application of appropriate offsets in signal coordination. Individual intersection capacity is not increased, but the utilisation of that capacity may be accomplished in ways that reduce intersection delays compared to those experienced at isolated intersections, even under heavy traffic flows. There are complications engendered by the interference of minor traffic movements with the platoons (e.g. vehicle turning into the main road that are then caught at downstream signals and form queues in the path of the advancing platoons) and it is for these reasons that a network traffic model such as TrafikPlan has to be considered.

Figure 10.1
Expected total delays for coordinated signals over a range of green times and offsets (c = 100 seconds, s = 3600 pcu/h and q = 1080 pcu/h)

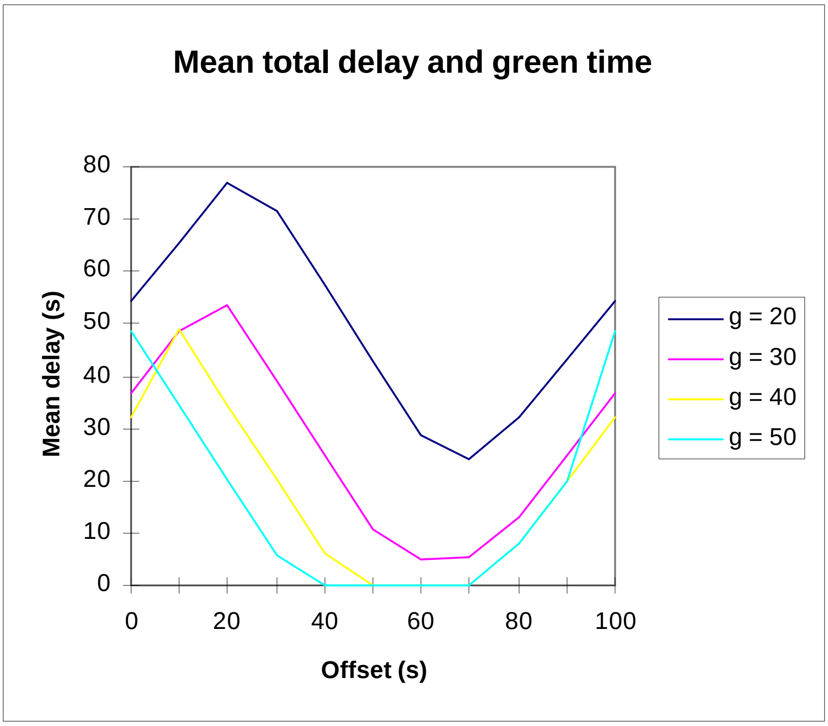
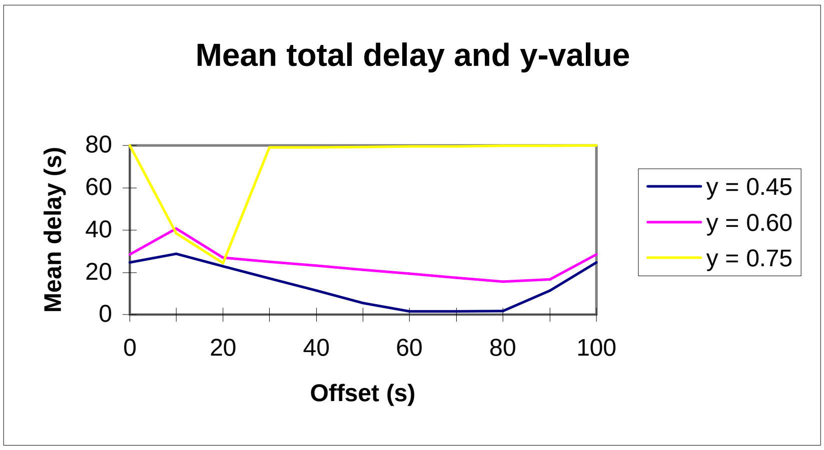


Figure 10.2
Expected total delays for coordinated signals a range of traffic volumes (y-values) and offsets (c = 100 seconds, g = 50 seconds , and s = 3600 pcu/h)



10.6 Summary

The inclusion of the GLB model in TrafikPlan allows the network model to be used to examine the effects of signal coordination and specific offset settings on delays and queuing in a road network, under different traffic conditions which can include alternative speed limits. The network model is then applied in this way to a number of synthetic networks, each under different levels of traffic demand, to test the effects on overall travel times and don delays under different speed limits.

11. EXPERIMENTAL DESIGN AND MODELLING PLAN

The first part of this report is concerned with the traffic modelling capabilities of the TrafikPlan package and how these can be related to speed limits. The second part, beginning with this chapter, looks at the outputs of a series of network model runs in which the speed limit was a principal independent factor.

TrafikPlan is a ‘dense network’ traffic model created for the design and evaluation of traffic management schemes and for the assessment of the traffic impacts of land use developments. It can distinguish between different road classes and design standards, traffic control systems and devices, and traffic management schemes. The model can also incorporate the effects of different speed limits on roads, assuming ‘weak’ level compliance, i.e. where drivers chose a cruising speed at about the speed limit, with a noticeable proportion of drivers exceeding the limit, as is the general rule for observed driver behaviour[1].

Use of TrafikPlan can then be made to test the effects of alternative speed limits on arterial roads and local streets, on overall journey times in a road network and on other factors related to traffic progression. These other factors include the delay time component of the overall travel time, fuel consumption, and pollutant emissions. Use of a computer model such as TrafikPlan allows the effects of the speed limits to be isolated from other traffic variables (which can be held constant during the model runs). In addition, different levels of traffic demand can be applied to the selected networks, to see how any effects change with levels of traffic congestion.

The remainder of this report describes the results obtained from modelling of a selection of a synthetic (but representative) road networks, under different levels of traffic demand, with different geometric design standards and speed limit regimes applied to them.

11.1 Experimental design

A number of small ‘stereotype’ networks were devised, and are described in Chapter 12. In addition, levels and patterns of traffic demand (trip movements) for these networks are also proposed, to offer a controlled method of testing the effects of variations in the level and spatial distribution of travel in them. These are also described in Chapter 12.

The experimental design for the study involved the selection of three basic network configurations (A, B and C, increasing in size and complexity), with two different design standards applied to each network, yielding a set of six networks in all. Each network was modelled at four different levels of traffic congestion, ranging from light to moderate traffic, to oversaturated conditions. For each combination of network and traffic congestion level, each of three speed limit regimes (60, 50 and 40 km/h) were applied and modelled using TrafikPlan. This yields a full factorial set of 72 different conditions.

[1] A ‘strong’ compliance would assume that the speed limit represented a maximum possible speed, exceeded by only a few drivers.

A number of parametric measures of traffic performance were used to assess the networks under the different operating conditions of traffic congestion level and speed limit regime. These parameters are described below.

11.2 Traffic performance parameters

The evaluation of the relative performance of a given traffic network under different traffic management schemes and control regimes may be gauged on the basis of a number of different parameters. These parameters relate to measures of overall traffic load, measures of capacity, and measures of delay. Delay-oriented parameters have to be assessed in terms of the selected definition of delay, with the definition of ‘system delay’, the total excess travel time over the minimum or ‘free flow’ travel time for a given link, intersection or road section. For the situation where alternative speed limits are being considered, the ‘free flow’ travel time will be different for different speed limits, and so a datum free flow travel time is needed. That stemming from the *status quo*, i.e. for a 60 km/h general urban speed limit would seem most appropriate. Differences in travel times (and ‘free flow’ travel times) depending on the presence or absence of traffic signal coordination also have to be considered. The *status quo* datum may be assumed to be isolated signal control, i.e. no signal coordination imposed on the network. The delay-oriented parameters may also include measures of the smoothness of traffic progression, perhaps termed ‘quality of traffic stream flow’.

11.2.1 Parameters of traffic load

These parameters indicate the volumes of traffic using the network and the travel distances involved. Specific parameters are:

1. Vehicle-kilometres of travel (VKT)
2. Vehicle-hours of travel (VHT)
3. Vehicle-hours of delay (VHD)
4. Mean journey speed ($\bar{v}$)
5. Mean travel time ($\bar{t}$)
6. Mean delay time ($\bar{d}$)

Each of these parameters may be disaggregated by link type. Given a network of links e, for which the link length is x_e, the link traffic volume is q_e, the unit travel time on the link is u_e, the free flow unit travel time is $u_0(e)$, and l is the link type. Define a delta function δ_{el} such that

$d_{el} = 1$ if link e is of type l

$d_{el} = 0$ otherwise

Then the VKT, VHT and VHD may be defined as follows.

For link type l,

$$VKT(l) = \sum_e d_{el} q_e x_e \qquad (11.1)$$

$$VHT(l) = \sum_e d_{el} q_e x_e u_e \qquad (11.2)$$

$$VDT(l) = \sum_e \delta_{el} q_e x_e (u_e - u_0(e)) \qquad (11.3)$$

The overall traffic parameters for all links in the network are then given by

$$VKT = \sum_l VKT(l) \qquad (11.4)$$

$$VHT = \sum_l VHT(l) \qquad (11.5)$$

$$VDT = \sum_l VDT(l) \qquad (11.6)$$

If N_t is the total number of trips made in the network over the period, then the mean travel parameters per trip may be defined as

$$\bar{v} = \frac{VKT}{VHT} \qquad (11.7)$$

$$\bar{t} = \frac{VHT}{N_t} \qquad (11.8)$$

$$\bar{d} = \frac{VDT}{N_t} \qquad (11.9)$$

11.2.2 Delay, congestion and quality of flow parameters

The control variables for each link (e) in a given network under a specified traffic control regime are:

- the congestion factor (σ), which has the values 0, 1, 2 and 3
- the speed limit (V), given as 60, 50 or 40 km/h
- the signal coordination status (Z), defined as 0 for isolated signal control and 1 for coordinated control

Given the link and traffic variables link length x_e, unit travel time u(e, σ, V, Z), and free flow travel time u_0(e, σ, V, Z), the delay, congestion and flow quality parameters may be defined as follows.

Mean system delay on a link, d(e, σ, V, Z), is given by

$$d(e,s,V,Z) = x_e\left[u(e,s,V,Z) - u_0(e,V,Z)\right] \qquad (11.10)$$

The mean delay may also be expressed in terms of a dimensionless congestion index, which may be termed CI(e, σ, V, Z) where

$$CI(e,s,V,Z) = \frac{d(e,s,V,Z)}{x_e u_0(e,V,Z)} = \frac{u(e,s,V,Z)}{u_0(e,V,Z)} - 1 \qquad (11.11)$$

Note that CI(e, σ, V, Z) cannot take negative values.

These parameters are for a given network with its specified traffic control regime (signal coordination plan and speed limit).

In order to make comparisons between networks under different traffic control regimes, other parameters must be introduced. The assumed datum is a network with a 60 km/h general speed limit and uncoordinated signals. The parameters could then include the following.

The delay time with base u_0(e, 60, 0), described as d60(e, σ, V, Z), the difference in travel time on a link in the specified network and on that link with a 60 km/h general speed limit and uncoordinated signals:

$$d60(e,s,V,Z) = x_e\left[u(e,s,V,Z) - u_0(e,60,0)\right] \qquad (11.12)$$

The congestion index relative to base u_0(e, 60, 0), i.e. CI60(e, σ, V, Z), is defined as

$$CI60(e,s,V,Z) = \frac{d60(e,s,V,Z)}{x_e u_0(e,60,0)} = \frac{u(e,s,V,Z)}{u_0(e,60,0)} - 1 \qquad (11.13)$$

which may be compared to CI(e, σ, V, Z) - see equation (11.11). Note that CI60(e, σ, V, Z) can take both negative and non-negative values.

In addition, comparisons between link travel times may be useful. Two such parameters are:

- the change in free flow travel time compared with a 60 km/h speed limit and isolated signal control, Δt_0(e, V, Z)

$$\Delta t_0(e,V,Z) = x_e\left(u_0(e,V,Z) - u_0(e,60,0)\right) \qquad (11.14)$$

- the change in actual travel time compared with a 60 km/h speed limit and isolated signal control, Δt(e, σ, V, Z)

$$\Delta t60(e,s,V,Z) = x_e\left(u(e,s,V,Z) - u(e,s,60,0)\right) \qquad (11.15)$$

Several of the above parameters are strongly related to each other. The link-based parameters u, CI, CI60 and Δt_0 may be taken as an overall representative set of parameters. For this set:

- the unit travel time (u, min/km) directly represents travel times on links in the network, and thus travel time for journeys through the network. In addition, the reciprocal of u is the average journey speed on each link, and thus u also represents travel speeds (a value of u of 1.0 min/km corresponds to a speed of 60 km/h);

- the congestion index CI (dimensionless) indicates the proportion of travel time on the link that is delay time (i.e. excess travel time above the free flow travel time). This provides a representation of system delay as a parameter that can be applied over each link in a network and between networks;
- the relative congestion index CI60 (dimensionless) indicates the different between the travel time on a link for a given speed limit and the free flow travel time on the link when a speed limit of 60 km/h is applied. This is a representation of traffic delay relative to the 60 km/h speed limit situation, and
- the difference in free flow travel time (Δt_0, min) for each link between the case of a specified speed limit and the 60 km/h limit provides a measure of the change in minimum possible travel times.

11.2.3 Traffic impact factors

In addition to the traffic parameters described above, the impacts of the traffic system on the surrounding areas need to be considered in the analysis. Two factors, the average fuel consumption and the mean emissions of carbon monoxide were selected to represent the impacts - these factors were already included in the TrafikPlan outputs. The running speed model formulation described in Section 8.2.3 was thus applied to the estimation of these variables in each network tested. Fleet composition could not be included as a control variable in the modelling, so unleaded petrol consumption and carbon monoxide emissions for a modern 4-cylinder car with EFI (a Toyota Camry sedan) were the specific parameters computed (see Taylor and Young (1996ab) for details of the specific model parameters applying to this vehicle).

11.3 Summary

The experimental design and the modelling plan outlined in this chapter were then applied to a set of synthetic networks. Modelling runs for each network were to be undertaken with different speed limits (40, 50 and 60 km/h), over a range of traffic congestion levels and under two different traffic control strategies (isolated (‘uncoordinated’) signal control and coordinated signal control). Comparisons of network perforamnce under the different speed limits were made at two levels:

1. the network level, where the overall parameters mean journey speed, mean travel time and mean delay were to be considered, and
2. at the link level, where the link-based parameters defined under 11.2.2 and 11.2.3 above were to be examined and compared.

These comparisons would then yield an informative picture of the traffic impacts of the different speed limits.

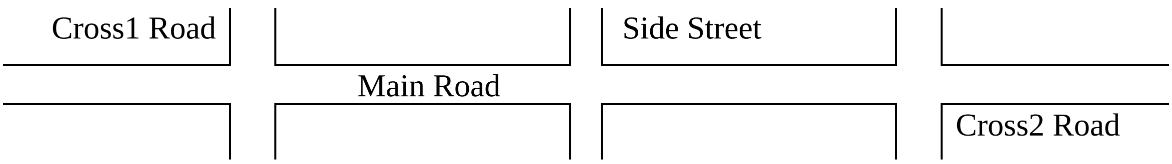
12. THE TEST NETWORKS

Three synthetic test networks were devised for the study, based on a grid pattern of main roads and the ‘one-mile’ (1.6 km) spacing between major roads characteristic of urban Australia (Brindle, 1989). These networks, ‘A’, ‘B’ and ‘C’ are similar, and of increasing size and complexity. Two variants of each network were developed, of differing geometric design standards. Thus, for example, network ‘A1’ is a basic network with few traffic engineering design and traffic management measures in place, whereas network ‘A2’ has substantial traffic engineering treatments (e.g. fully channelised intersections, separate turn phases at traffic signals) on the same spatial network pattern. The network patterns are given below. In addition, a tidal pattern of traffic origin-destination movements through these networks, which can be scaled to simulate increasing levels of traffic congestion, was also devised, for use in the modelling of the traffic flows in the networks and in the determination of the levels of traffic congestion.

12.1 The synthetic networks

Network A is a simple one-dimension network, comprising an arterial road with two major intersections along it. These intersections are 1.6 km apart. A minor road (‘Side Street’) intersects with the arterial road midway between the two major intersections. A schematic plan of the network is given in Figure 12.1. Traffic signals are installed at the two major intersections (of ‘Main Road’ with ‘Cross1 Road’ and ‘Cross2 Road’. The intersection with ‘Side Street’ is a major-minor intersection, with ‘stop’ signs on ‘Side Street’. The arterial road is taken in two alternative geometric configurations, firstly as a four-lane single carriageway road, and secondly as a dual carriageway road with two lanes on each carriageway.

Figure 12.1: Test network A - single arterial road

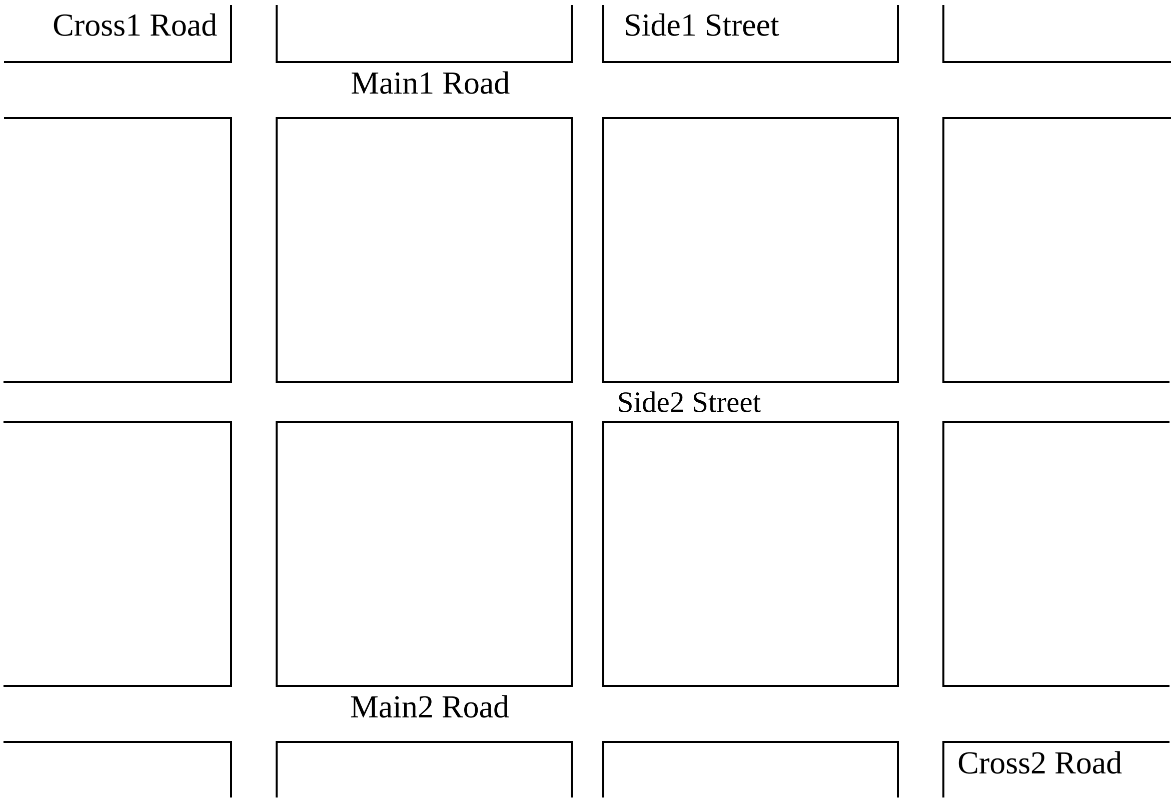


In its basic form (network A1) there are two lanes for flow in each direction along Main Road, and on Cross1 Road and Cross2 Road. These two lanes are all that is provided at the stop lines of the signalised intersections, where they are shared (T-L and T-R) between the through, left and right turning movements. In the network A2, a higher design standard has been applied. Main Road is a dual carriageway arterial road, still with two-lanes for flow in each direction. The major intersections are fully channelised, with separate left and right turning lanes, the left turn being catered for by a free-running slip lane and separate right turn phases included in the signal cycle.

Network B comprises a grid of two east-west and two north-south arterial roads, again with spacing at 1.6 km between parallel arterial roads. Figure 12.2 indicates the network layout. All major intersections are signalised. The intersections of the side streets with the arterial roads are major-minor intersections, with 'give way' control on the side streets. The intersection inside the local area (i.e. the intersection of 'Side1 Street' and 'Side2 Street' is a single-lane roundabout. 'Main2 Road' and 'Cross2 Road' are single carriageway roads with two lanes in each direction. 'Main1 Road and 'Cross1 Road' are treated (1) in network B1 as four-lane single carriageway roads, then (2) in network B2 as six-lane dual carriageway roads.

Network B2 has full channelisation at all main road intersections. The minor road intersection of Side1 Street and Side2 Street is a single lane roundabout.

Figure 12.2: Test network B - a box of four arterial roads

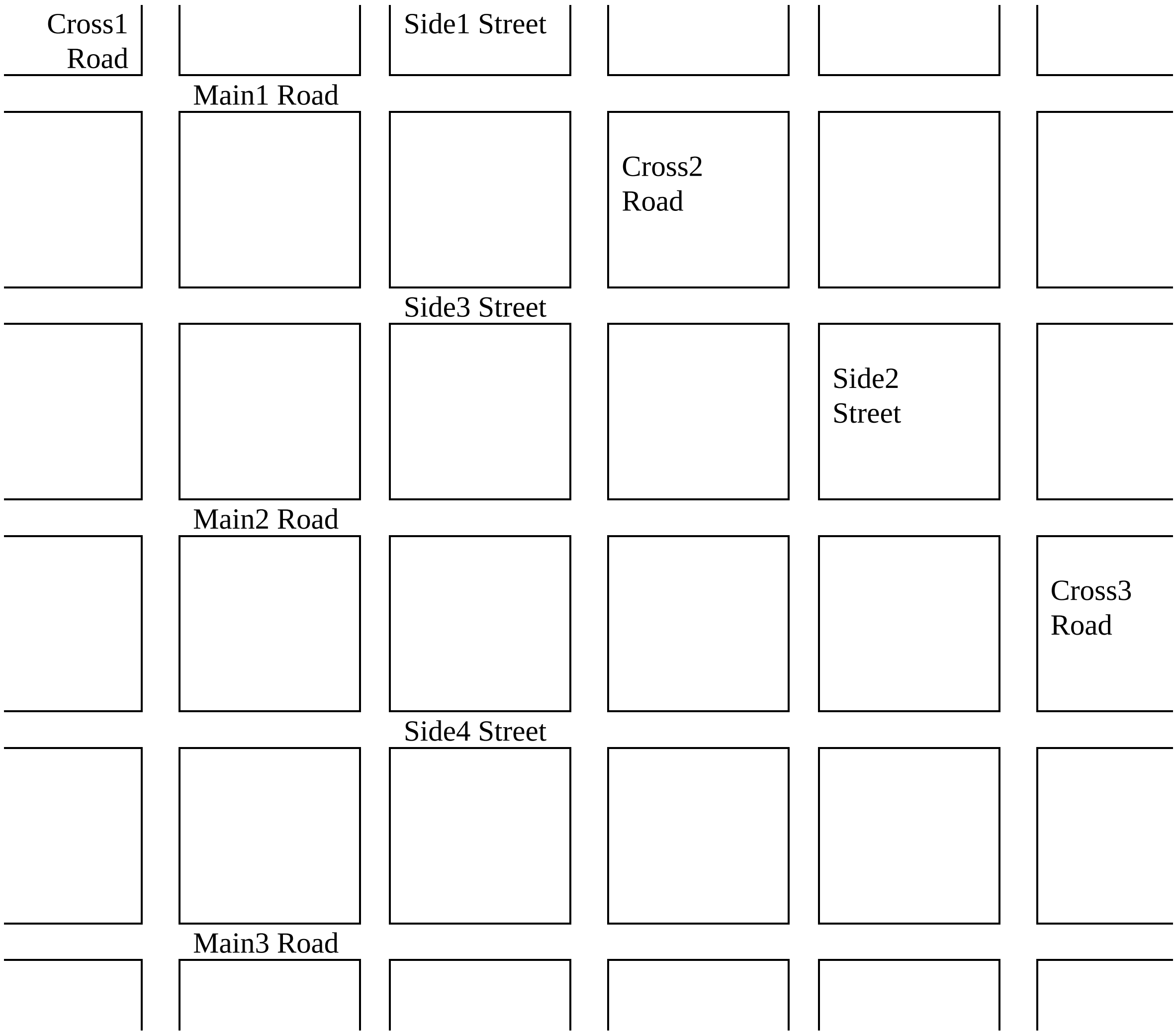


Network C comprises a grid of three north-south and three east-west arterial roads, at 1.6 km spacings, with local streets in between. A sketch of the network is shown in Figure 12.3. All major intersections are signalised, and all intersections of the main roads with the side streets are major-minor, with ‘give way’ signs on the side streets. The intersections of the minor roads are all single-lane roundabouts.

In network C1 all main roads are four-lane single carriageway arterial roads. All main road intersections have signals, while all intersections of main roads with side streets have ‘give way’ control on the side streets. All side street intersections are single lane roundabouts.

In network C2, all main roads are dual carriageway. Main2 Road and Cross2 Road have two lanes for flow in each direction, while all of the other main road in C2 have three lanes for each direction. Full channelisation is employed at all main road intersections, with separate turning phases in place at all signalised intersections.

Figure 12.3: Test network C - grid of 3 east-west and 3 north-south arterial roads



12.2 Travel demand patterns

The spatial distribution of travel demand and the total demand on the network must also be accounted for in investigations of the test networks. Delays at intersections and hence travel times on links are significantly affected by the patterns of turning movement flows at the intersections. These patterns in turn depend on the spatial distribution (origin-destination movements) of travel in the network.

Thus modelling of the test networks must allow for a range of origin-destination patterns, or at least be undertaken and reported with qualification to note the range of patterns investigated.

In all networks, an origin-destination (O-D) matrix of trip movements has to be defined. This is required input for the trip assignment module in TrafikPlan. For the general case, assume that a base matrix of trips $\{M_{ij}^b\}$ from origin i to destination j can be defined. The total number of trips in this matrix, $\sum M^b$, is then given by summing over all of the elements in the matrix, i.e.

$$\sum M^b = \sum_{ij} M_{ij}^b \tag{12.1}$$

P_i^b is the total number of trips in the base O-D matrix starting at origin i (i.e. the trip production of origin i), given by the row sum in the matrix, i.e.

$$P_i^b = \sum_j M_{ij}^b \tag{12.2}$$

and the number of trips bound for destination j - the trip attraction of j A_j^b - is given by

$$A_j^b = \sum_i M_{ij}^b \tag{12.3}$$

which is the column sum in the matrix.

12.2.1 Level of traffic congestion

Different levels of traffic demand in the test network may be modelled by scaling the base O-D matrix. Using a scaling factor $0 \le \sigma \le 2$, the level of traffic demand may be varied up to three times the total traffic in the base O-D matrix by applying the equation

$$\{M_{ij}\} = (1 + s)\{M_{ij}^b\} \tag{12.4}$$

as discussed in Wigan and Luk (1976). This equation scales the total traffic demand, whilst maintaining the spatial distribution of travel on the network.

12.2.2 Spatial distribution of travel

For the case study networks (see Section 12.3), origin-destination matrices can be found by observation or by estimation from observed link volumes, using the methods described in Taylor, Young and Bonsall (1996, pp.247-255 and pp.107-114).

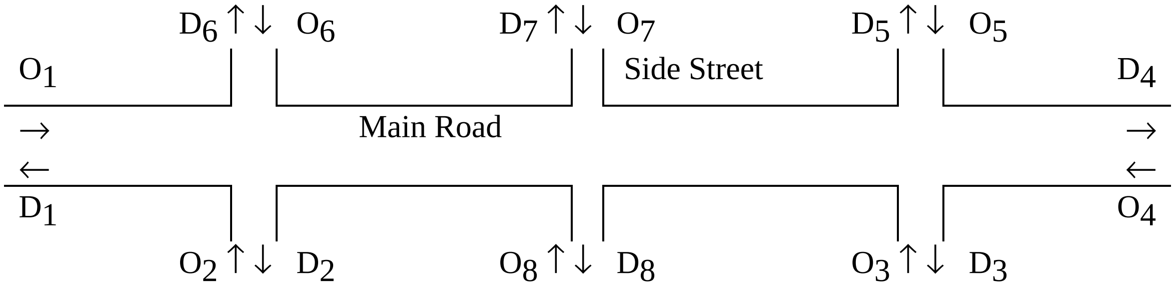
For the synthetic networks (networks A, B and C), suitable O-D matrices have to be defined. The following method was proposed, based on the specification of sets of origins and destinations, and the assumption that the total number of trips originating at origin number 1 is equal to a given number of trips Q. Thus

$$P_1^b = \sum_j M_{ij}^b = Q \qquad (12.5)$$

Then all other trip productions and trip attractions are set as scaled values of P_1^b.

For the case of network A, eight origins and eight destinations are defined, as shown in Figure 12.4.

Figure 12.4: Origins and destinations in test network A



The O-D matrix for this network is then set up as shown in the spreadsheet in Table 12.1. This table indicates the row and column sums (the trip productions and attractions), with the trip production for Origin 1 as the main independent variable. All other trip productions are scaled relative to that for Origin 1. For network A the various factors for the trip productions and attractions on all roads intersecting with Main Road are determined by using Hauer's mean turning proportion factors for urban arterial roads and collector roads (Hauer, Pagitsas and Shin, 1981) determined for Toronto and validated for Australian cities by Luk (1989) and Taylor, Young and Bonsall (1996).

In addition, the spreadsheet includes a value for the congestion factor, which can be modified to scale the O-D matrix according to the desired level of traffic congestion in the synthetic network.

Table 12.1
General origin-destination matrix for network A

O-D Matrix for FORS network NetA

Entry vol from O#1=	500	Congestion factor =	0.00
O#2 flow as propn of O#1 =	0.3		
O#3 as propn of O#2 =	0.5		
O#4 flow as propn of O#1 =	0.3		
O#5 as propn of O#4 =	0.5		
O#6 flow as propn of O#1 =	0.4		
O#7 flow as propn of O#1 =	0.1		
O#8 as propn of O#7 =	0.5		

				Destination						Trip
Origin	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	[Row Sum]	**Prodn.**
1	0	60	60	41	41	263	19	15	500	500
2	18	0	114	2	2	12	1	1	150	150
3	9	57	0	1	1	6	0	0	75	75
4	12	2	2	0	114	18	1	1	150	150
5	6	1	1	57	0	9	0	0	75	75
6	105	17	17	24	24	0	6	8	200	200
7	13	2	2	1	1	6	0	24	50	50
8	3	1	1	1	1	7	12	0	25	25
Trip Attrn	167	139	196	128	185	322	40	49	1225	1225

Uses Hauer's turning proportions at intersections (see tables below)

Turning proportions

1. Left turns

	To:	
From:	Arterial	Collector
Arterial	0.12	0.04
Collector	0.35	0.18

2. Right turns

	To:	
From:	Arterial	Collector
Arterial	0.12	0.05
Collector	0.17	0.20

3. Through movements

	To:	
From:	Arterial*	Collector*
Arterial	0.76	0.91
Collector	0.48	0.62

A similar (12×12) O-D matrix was constructed for network B (in which the first eight origins and destinations are major road entry and exit links, and the last four are minor road entry and exit links). O-D sites are numbered symmetrically starting at the bottom lefthand corner, with Main2 Road as O-D site number 1 (Figure 12.2), and the sites then being numbered anticlockwise. O-D site number 8, Main1 Road at the top lefthand corner of the network (Figure 12.2) is the major generator in the network, and the principal flow movement is along Main1 Road from west to east (i.e. from O-D site number 8 to O-D site number 5).

Table 12.2 shows the spreadsheet developed to generate the origin-destination matrix for network B. This is similar in concept to that for network A (Table 12.1), but there are more degrees of freedom in this table and the method thus adopted to set trip productions and attractions was to select a ‘representative’ trip movement in each row of the O-D matrix, and scale this from the principal flow movement. The row sum (total trip production of the O-D site) is then scaled up from the representative movement. Only those trip movements corresponding to a turning movement at an intersection (e.g. the movement from O-D site number 8 to O-D site number 7 at the top left hand corner of Figure 12.2) is defined by a turning proportion (as in network A).

The spreadsheet in Table 12.2 again allows for the use of the congestion factor, to scale the travel demand for different traffic congestion conditions.

The origin-destination table for network C was developed in similar fashion to that for network B, with 20 O-D sites in this case. Table 12.3 shows the spreadsheet used to generate the origin-destination data.

The origin-destination data tables were then used in TrafikPlan, which assigned the trips in the table to routes through the specific networks to build up the traffic volumes on the links, and to model the turning flows at each intersection. TrafikPlan also allows the user to see the origin-destination data drawn as desire lines on maps of the study network. Figure 12.5 shows the full set of desire lines for network C (congestion factor 3). The interactive capabilities of the TrafikPlan graphics displays may also be used to show parts of the desire lines. For example, Figure 12.6 shows the travel desire for Origin site 11 (Main1 Road, west) in network C. This display indicates the intended destinations of the trips originating at this point and the band widths on the map show the relative importance of each trip movement.

Table 12.2 General origin-destination matrix for network B (page 1 of 2)

O-D Matrix for FORS network NetB

12 peripheral stations (8 on arterial and 4 on collectors)

Congestion Factor =	3.00

Main flow is W to E on Main1 Road

For movement from station 8 to 5, base flow =	300
The actual flow (scaled by congestion factor) =	1200
Opposing flow factor =	0.50
Secondary flow direction is S to N. S to N factor =	0.60

Primary arterials are Main1 Road and Cross1 Road

Secondary arterial factor =	0.70

There are: no movements between adjacent peripheral stations
no movemnts from arterials to collectors

Collector road factor =	0.10
For movement from station 9 to 11, scaled base flow =	120
Secondary collector road flow factor (E to W)=	0.20
N to S collector road factor =	0.40

Turning proportions, using Hauer's results:

	1. Left turns To:			2. Right turns To:	
From:	Arterial	Collector		Arterial	Collector
Arterial	0.12	0.04	Arterial	0.12	0.05
Collector	0.35	0.18	Collector	0.17	0.20

Total trips originating at arterial road station = (1 + B)*Qrep where Qrep is the identified movement for the origin
B is the trip scaling factor, defined by X*Rsum + Qrep + n*qmin = (1 + B)*Qrep
X is the turn factor for the direct t.m. flow
For non-negativity, B > X/(1 - X)

Thus B(min) = 0.273 for R and L turns from arterial. Set B =	0.40
n is the no. of minor movements (each of qmin) =	4
Then qmin = Qrep*(B - X - B*X)/n = KX*Qrep. For R turn KR =	0.058
for L turn KL =	0.058

Table 12.2 General origin-destination matrix for network B (page 2 of 2)

The O-D matrix for NetB Congestion factor = 3.00

	Destination												
Origin	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**Row sum**
1	0	141	49	840	49	49	49	0	0	0	0	0	**1176**
2	121	0	0	42	42	42	720	42	0	0	0	0	**1008**
3	42	0	0	121	42	720	42	42	0	0	0	0	**1008**
4	420	24	71	0	0	24	24	24	0	0	0	0	**588**
5	35	35	35	0	0	101	35	600	0	0	0	0	**840**
6	21	21	360	21	60	0	0	21	0	0	0	0	**504**
7	21	360	21	21	21	0	0	60	0	0	0	0	**504**
8	0	70	70	70	1200	70	202	0	0	0	0	0	**1680**
9	0	10	0	10	10	0	10	0	0	24	120	48	**230**
10	10	0	0	10	0	10	10	0	24	0	48	72	**182**
11	5	0	5	0	0	5	0	5	60	48	0	48	**175**
12	0	5	5	0	5	0	0	5	48	36	48	0	**151**
Col. sum	**674**	**665**	**614**	**1133**	**1428**	**1020**	**1090**	**799**	**132**	**108**	**216**	**168**	**8047**

Table 12.3 General origin-destination matrix for network C (page 1 of 3)

O-D Matrix for FORS network NetC

20 peripheral stations (12 on arterials and 8 on collectors)

Congestion Factor =	0.00

Main flow is W to E on Main1 Road

For movement from station 11 to 7, base flow =	300
The actual flow (scaled by congestion factor) =	300
Opposing flow factor =	0.50
Secondary flow direction is S to N. S to N factor =	0.60

Primary arterials are Main1 Road and Cross1 Road

Secondary arterial factor =	0.70

There are: no movements between adjacent peripheral stations
no movements from arterials to collectors

Collector road factor =	0.10
For movement from station 19 to 16, scaled base flow =	30
Secondary collector road flow factor (E to W)=	0.20
N to S collector road factor =	0.40

Turning proportions, using Hauer's results:

	1. Left turns			2. Right turns	
	To:			To:	
From:	Arterial	Collector		Arterial	Collector
Arterial	0.12	0.04	Arterial	0.12	0.05
Collector	0.35	0.18	Collector	0.17	0.20

Total trips originating at arterial road station = (1 + B)*Qrep where Qrep is the identified movement for the origin
B is the trip scaling factor, defined by X*Rsum + Qrep + n*qmin = (1 + B)*Qrep
X is the turn factor for the direct t.m. flow
For non-negativity, B > X/(1 - X)

Thus B(min) = 0.273 for R and L turns from arterial. Set B =	0.40		
n1, n2 are the no. of minor movements (each of qmin). n1 =	4	n2 =	8
Then qmin = Qrep*(B - X - B*X)/n = KX*Qrep. For R turn KR(n1) =	0.058	K(n2) =	0.05
for L turn KL(n1) =	0.058		

Table 12.3 General origin-destination matrix for network C (page 2 of 3)

The O-D matrix for NetC Congestion factor = 0.00

	Destination											
Origin	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
1	0	35	12	12	210	12	12	12	12	12	0	0
2	30	0	0	0	10	10	10	10	10	180	10	10
3	9	0	0	0	9	9	9	9	180	9	9	9
4	10	0	0	0	30	10	10	180	10	10	10	10
5	126	7	7	21	0	0	0	7	7	7	7	7
6	5	5	5	5	0	0	0	5	5	5	5	105
7	9	9	9	9	0	0	0	25	9	9	150	9
8	6	6	6	105	6	6	15	0	0	0	6	6
9	5	5	105	5	5	5	5	0	0	0	5	5
10	6	105	6	6	6	6	6	0	0	0	15	6
11	0	17	17	17	17	17	**300**	17	17	50	0	0
12	0	11	11	11	11	210	11	11	11	11	0	0
13	1	0	0	0	1	1	1	1	1	1	1	1
14	1	0	0	0	1	1	1	1	1	1	1	1
15	1	1	1	1	0	0	0	1	1	1	1	1
16	1	1	1	1	0	0	0	1	1	1	1	1
17	1	1	1	1	1	1	1	0	0	0	1	1
18	1	1	1	1	1	1	1	0	0	0	1	1
19	0	1	1	1	1	1	1	1	1	1	0	0
20	0	1	1	1	1	1	1	1	1	1	0	0
Col. sum	**214**	**208**	**186**	**199**	**312**	**294**	**386**	**284**	**269**	**301**	**226**	**176**

Table 12.3 General origin-destination matrix for network C (page 3 of 3)

The O-D matrix for NetC Congestion factor = 0.00

13	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**Row sum**
0	0	0	0	0	0	0	0	**294**
0	0	0	0	0	0	0	0	**252**
0	0	0	0	0	0	0	0	**252**
0	0	0	0	0	0	0	0	**252**
0	0	0	0	0	0	0	0	**176**
0	0	0	0	0	0	0	0	**147**
0	0	0	0	0	0	0	0	**210**
0	0	0	0	0	0	0	0	**126**
5	0	0	0	0	0	0	0	**126**
0	0	0	0	0	0	0	0	**126**
0	0	0	0	0	0	0	0	**420**
0	0	0	0	0	0	0	0	**294**
0	0	6	6	6	6	3	1	**39**
0	0	12	12	6	6	3	1	**51**
3	6	0	0	6	6	6	1	**39**
3	6	0	0	3	3	15	6	**47**
3	3	3	2	0	0	6	1	**29**
3	3	3	2	0	0	6	1	**29**
6	6	12	**30**	12	12	0	0	**29**
1	1	1	12	1	1	0	0	**29**
24	**25**	**37**	**63**	**34**	**34**	**39**	**12**	**2966**

Figure 12.5
Desire lines for full O-D matrix in network C, congestion factor = 3

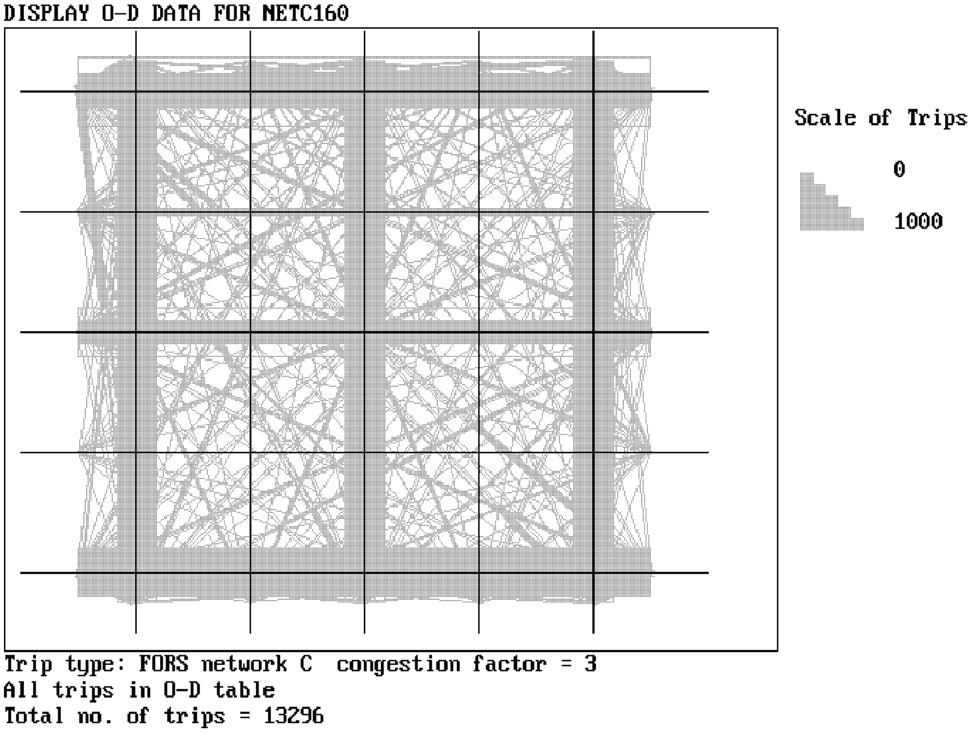
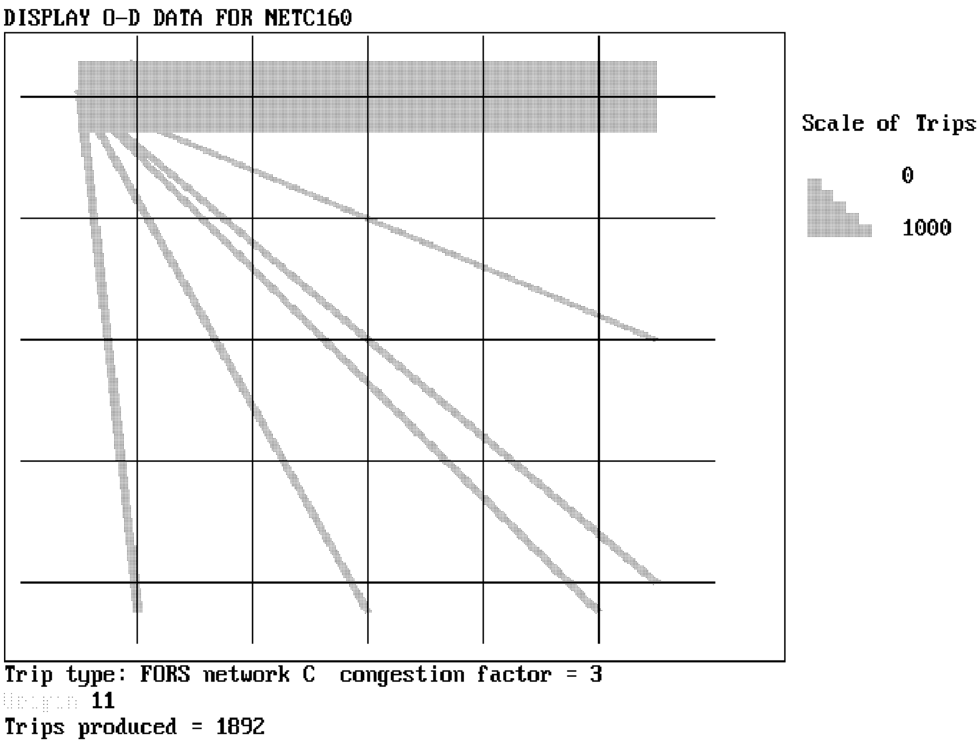


Figure 12.6
Desire lines for trips from Origin 11 in network C, congestion factor = 3



13. MODEL RESULTS

Model results available for this interim final report focus on the synthetic networks introduced in Chapter 12. These networks enabled the investigation of the effects of the three speed limits (60, 50 and 40 km/h) and the signal control strategy (isolated or 'uncoordinated' control and coordinated control) on travel times, delays, journey speeds, fuel consumption and emissions.

The experimental design outlined in Chapter 11 was used as the basis for the modelling study. The three synthetic networks were each considered with both isolated and coordinated signal control strategies. Traffic flows corresponding to four separate congestion levels were modelled. For each of these 24 combinations of network, signal control strategy and traffic congestion level, each of the three alternative speed limits were applied, giving a total of 72 separate cases.

The TrafikPlan model was then applied to each of the 72 cases, and outputs from the traffic model collated and analysed. In all cases for the synthetic networks, the default route choice model in TrafikPlan was applied. This is an 'equilibrium' assignment solution in which all trips are assigned to the minimum travel time routes for the particular trips, where the minimum paths take account of the congestion levels in the network and the actual travel times on congested links.

13.1 TrafikPlan outputs

A traffic network contains a large number of variables, including flows, travel times, delays and performance variables for each direction of flow on each link in the network and, in the case of a dense network model such as TrafikPlan, similar variables for each turning movement as well. Summary statistics including VKT, VHT, mean travel speed, mean travel time and mean delay are also available.

A variety of output forms, including both textual descriptions and graphical displays (both maps and diagrams) are available. Some examples of these are shown in the next set of figures. These figures (Figures 13.1 - 13.6) are all for network C1, with the maximum value (3.0) used for the congestion factor and a 60 km/h speed limit. Similar displays are available for the other five networks as well.

Figure 13.1 shows the link flows on network C1 under congestion factor 3 and 60 km/h speed limit. The broad nature of the tidal flow set in the origin-destination table for this network is from west to east and north to south, with the major input flows originating from the top righthand corner (Main1 Road and Cross1 Road).

Figure 13.2 is a textual display from TrafikPlan of the traffic volumes, mean journey speeds and travel times on Main1 Road, shown as a schematic strip map of the road. This traffic flow report shows directional flows and the travel times and speeds for the two directions of flow.

Figure 13.1
Traffic flows in network C1, congestion factor = 3

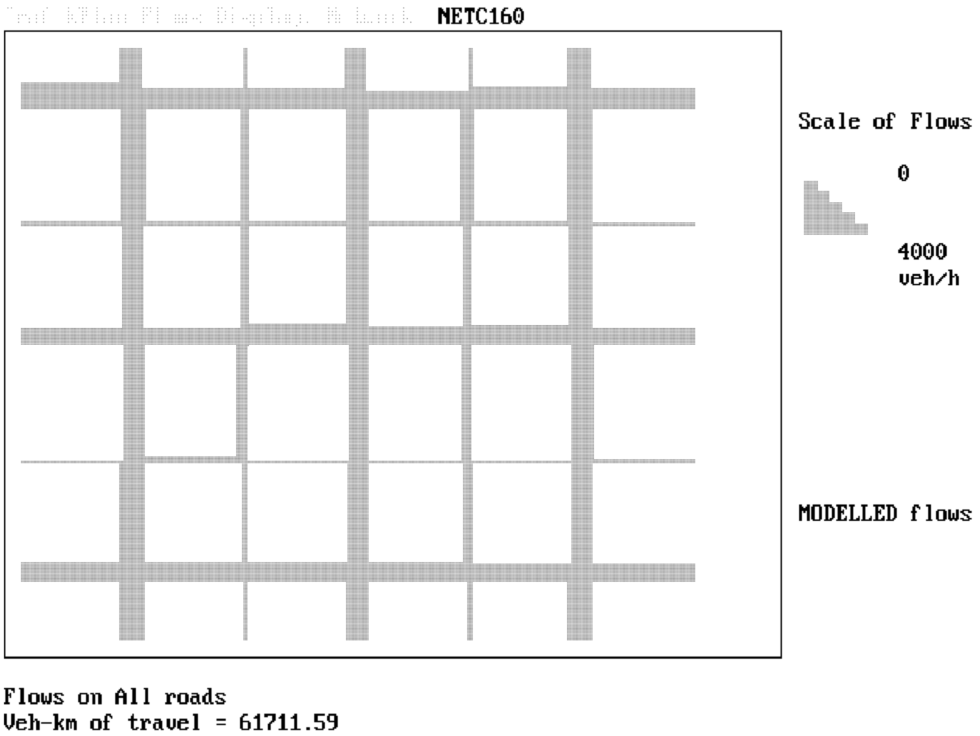


Figure 13.2
Traffic flow report for MAIN ROAD 1 in network C1, congestion factor = 3

```
TrafikPlan: REPORT ON TRAFFIC FLOWS. Network NETC160        14:58:30 04-13-1997
ROAD : MAIN1 ROAD                                              MODELLED flows
 flows
      Flow   Length  T-Time   Speed          Flow   Length  T-Time   Speed
    (veh/h)   (km)    (min)   (km/h)        (veh/h)   (km)    (min)   (km/h)
                                     * *
         0                           * *       0
               0.80    0.48   100.0  * *              0.80    4.18    11.5
       903                           * *    1891
                       CROSS1 ROAD        CROSS1 ROAD
      1045                           * *    1549
 ^             0.80    1.42    33.9  * *              0.80    0.82    58.9   |
/|\   1046                           * *    1548                             |
 |                     SIDE1 STREET       SIDE1 STREET                       |
 |     837                           * *    1546                            \|/
 |             0.80    0.81    58.9  * *              0.80    1.42    33.9   V
       838                           * *    1548
                       CROSS2 ROAD        CROSS2 ROAD
      1002                           * *    1303
               0.80    0.80    60.0  * *              0.80    0.82    58.9
      1002                           * *    1304
                       SIDE2 STREET       SIDE2 STREET

More data for this street. Press any key to continue
```

The next pair of figures (Figures 13.3 and 13.4) show details of the turning movement flows at a particular intersection (node 21, the intersection of Main1 Road and Cross1 Road) in network C1 (congestion factor 3, 60 km/h speed limit). Figure 13.3 is the basic intersection turning movement display, while Figure 13.4 is the turning movement display in which the flows from a particular approach leg (Main1 Road from the West) have been highlighted.

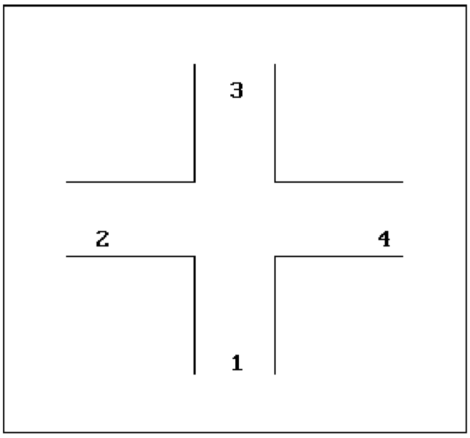
Similar interactive displays may be obtained for each intersection in a TrafikPlan network.

Figures 13.5 and 13.6 are examples of the statistical analysis of link-based variables available in TrafikPlan.

Figure 13.5 is a histogram of the one-way link traffic volumes in network C1 (congestion factor 3, speed limit 60 km/h), showing descriptive statistics for the histogram as well. Most of the links have small volumes (the modal frequency band is 0-200 veh/h), corresponding to the minor streets in the network (see also Figure 13.1), but there are link flows over the full range of 0 to 2000 veh/h.

A similar histogram is shown in Figure 13.6, with mean travel speed for each direction of flow on each link being the variable of interest. The modal frequency is for mean speeds between 56 and 64 km/h, corresponding to the lightly trafficked (local street) links in the network, and the 60 km/h speed limit. Other links show slower speeds, under the heavily congested main road conditions found in this network with congestion factor 3.

Figure 13.3
Turning movement flows at node 21 in network C1, congestion factor = 3



TrafikPlan 4.0: TURNING FLOWS

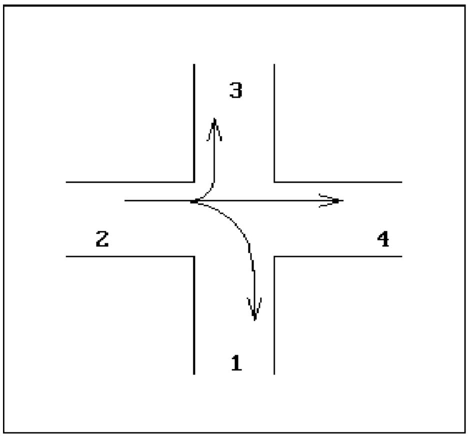
1 CROSS1 ROAD
2 MAIN1 ROAD
3 CROSS1 ROAD
4 MAIN1 ROAD

		LEG OUT				
		1	2	3	4	TOTAL
LEG IN	1	0	167	770	99	1036
	2	370	0	202	1319	1891
	3	457	60	0	131	648
	4	135	676	234	0	1045
TOTAL		962	903	1206	1549	4620

FLOWS DISPLAY OPTIONS
1. all flows leaving a leg
2. all flows entering a leg
3. single flow
4. switch Observed/Modelled
5. try another node Option

MODELLED traffic flows at node 21

Figure 13.4
Highlighted turning movement flows at node 21 in network C1, congestion factor = 3



TrafikPlan 4.0: TURNING FLOWS

1 CROSS1 ROAD
2 MAIN1 ROAD
3 CROSS1 ROAD
4 MAIN1 ROAD

		LEG OUT				
		1	2	3	4	TOTAL
LEG IN	1	0	167	770	99	1036
	2	370	0	202	1319	1891
	3	457	60	0	131	648
	4	135	676	234	0	1045
TOTAL		962	903	1206	1549	4620

Press any key to continue

MODELLED traffic flows at node 21

Figure 13.5
TrafikPlan histogram of link flows in network C1, congestion factor = 3

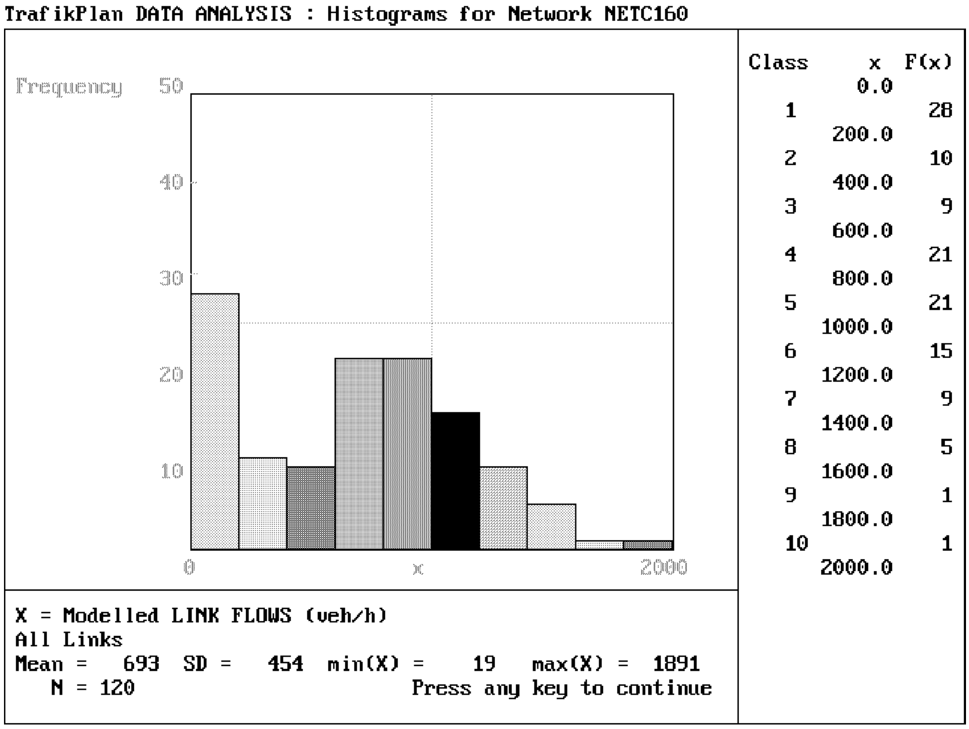
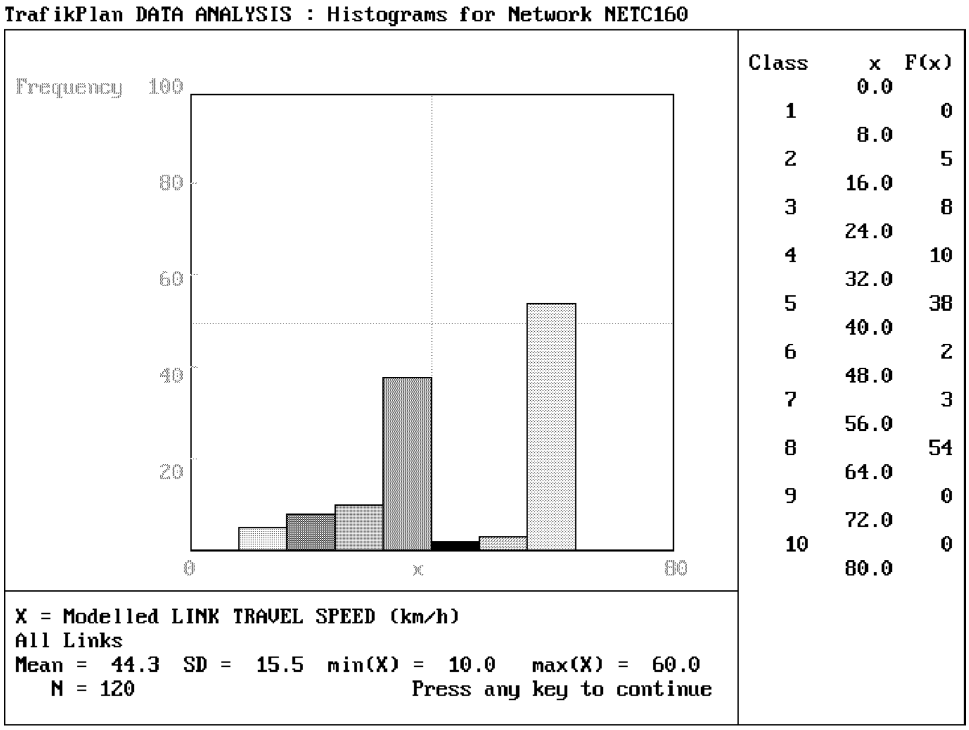


Figure 13.6
TrafikPlan histogram of link speeds in network C1, congestion factor = 3



13.2 Overall travel parameters for the networks

Some parameters of overall traffic performance for networks under different traffic control and speed limit regimes were introduced in Section 11.2.1. The three principal parameters introduced there were:

1. the mean journey speed in the network (km/h);
2. the mean overall travel time in the network, in min/veh, and
3. the mean overall delay in the network, in min/veh.

The modelled values for each of these parameters for the network, traffic control, traffic congestion and speed limit combinations are shown in the following tables (Tables 13.1 - 13.18).

Tables 13.1 - 13.6 show the mean journey speeds in the networks A1, A2, B1, B2, C1 and C2 respectively, for each congestion level, signal control strategy and speed limit.

Tables 13.7 - 13.12 show the values of the overall mean travel times for the same networks and traffic and network control variables, while Tables 13.13- 13.18 show the values for the overall mean delay times.

The model results for these parameters are more clearly seen graphically. Figures 13.7 - 13.12 provide comparisons of the mean journey speeds in each of the six networks, for each traffic congestion level, speed limit and signal control strategy.

The first overall result to emerge is that journey speeds through the networks are considerably less than the set speed limits. For example, Table 13.1 and Figure 13.7 indicate that, even for coordinated signal control, the mean journey speed in network A1 at congestion level 0 and 60 km/h speed limit is 41.5 km/h. This result was anticipated, for overall journey speed must take account of all time spent in queues and in accelerating to or decelerating from the cruise speed.

Secondly, the differences in overall travel speed for the different speed limits are somewhat less that the differences in the speed limits themselves. This, for network A1 (see Table 13.1) the overall mean speed for the 40 km/h speed limit with isolated signal control is 26.2 km/h, with that for the 60 km/h speed limit being 37.8 km/h - a difference of 11.6 km/h compared to the 20 km/h difference in speed limits. For network C2 the corresponding overall mean journey speeds are 40.0 km/h and 29.4 km/h, a difference of 10.6 km/h. The differences between speed limits decrease as the congestion level increases, as the opportunities to reach the higher speeds on the main roads are lessened and intersection delays increase.

Signal coordination leads to higher speeds in almost all cases, although signal coordination in networks C2, C1 (and perhaps B2) becomes less effective at congestion level 3.

Table 13.1
Mean travel speeds (km/h) in network A1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	37.8 41.5	37.2 41.7	36.8 41.7	36.0 41.2
50	isolated coordinated	31.9 34.8	31.7 34.9	31.4 34.9	30.9 34.2
40	isolated coordinated	26.2 28.2	26.1 28.3	25.9 28.3	25.6 27.8

Table 13.2
Mean travel speeds (km/h) in network A2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	35.3 38.5	35.2 39.5	35.1 40.4	34.7 39.8
50	isolated coordinated	30.3 34.6	30.3 34.6	30.2 34.6	29.9 33.5
40	isolated coordinated	25.2 28.1	25.2 28.2	25.1 28.2	24.9 27.5

Table 13.3
Mean travel speeds (km/h) in network B1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	43.7 50.1	41.0 50.4	40.1 48.7	40.5 37.7
50	isolated coordinated	37.5 42.0	37.2 42.2	36.8 41.2	35.4 33.6
40	isolated coordinated	31.4 34.2	31.3 34.3	30.9 33.5	29.9 29.0

Table 13.4
Mean travel speeds (km/h) in network B2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	42.4 48.2	42.0 49.1	41.4 48.9	40.7 43.4
50	isolated coordinated	36.6 40.9	36.5 41.5	36.2 41.0	35.5 38.3
40	isolated coordinated	30.8 33.7	30.8 33.9	30.5 33.4	30.0 31.6

Table 13.5
Mean travel speeds (km/h) in network C1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	42.2 49.1	41.4 48.7	39.2 43.6	36.9 33.7
50	isolated coordinated	36.1 41.1	35.7 40.5	34.9 37.9	33.3 31.2
40	isolated coordinated	30.2 33.3	30.0 33.0	29.6 31.2	27.9 28.1

Table 13.6
Mean travel speeds (km/h) in network C2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	40.0 46.2	40.1 47.8	40.0 48.4	39.8 48.0
50	isolated coordinated	34.6 40.3	34.6 40.4	34.6 40.4	34.6 40.4
40	isolated coordinated	29.4 32.9	29.4 33.0	29.4 33.0	29.2 32.9

Table 13.7
Mean travel times (min/veh) in network A1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	3.08 2.80	3.14 2.79	3.17 2.80	3.25 2.84
50	isolated coordinated	3.65 3.35	3.67 3.34	3.71 3.34	3.78 3.42
40	isolated coordinated	4.44 4.13	4.46 4.12	4.50 4.13	4.58 4.20

Table 13.8
Mean travel times (min/veh) in network A2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	3.30 38.5	3.31 39.5	3.33 40.4	3.38 39.8
50	isolated coordinated	3.84 3.37	3.85 3.37	3.86 3.37	3.91 3.49
40	isolated coordinated	4.61 4.15	4.63 4.14	4.64 4.14	4.69 4.26

Table 13.9
Mean travel times (min/veh) in network B1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	3.81 3.32	4.08 3.32	4.20 3.46	4.16 4.74
50	isolated coordinated	4.44 3.96	4.49 3.96	4.58 4.09	4.76 5.26
40	isolated coordinated	5.30 4.87	5.35 4.87	5.44 5.03	5.63 6.09

Table 13.10
Mean travel times (min/veh) in network B2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	3.93 3.46	3.99 3.42	4.06 3.46	4.13 4.01
50	isolated coordinated	4.55 4.08	4.59 4.04	4.66 4.12	4.74 4.56
40	isolated coordinated	5.40 4.95	5.45 4.94	5.51 5.05	5.61 5.52

Table 13.11
Mean travel times (min/veh) in network C1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	6.35 5.49	6.48 5.55	6.85 6.21	7.45 8.28
50	isolated coordinated	7.42 8.05	7.53 8.13	7.71 8.67	8.17 9.79
40	isolated coordinated	8.87 8.05	8.95 8.13	9.08 8.67	9.82 9.79

Table 13.12
Mean travel times (min/veh) in network C2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	7.42 6.49	7.48 6.32	7.54 6.29	7.56 6.30
50	isolated coordinated	8.58 7.39	8.64 7.41	8.71 7.48	8.69 7.46
40	isolated coordinated	10.13 9.04	10.16 9.07	10.25 9.16	10.30 9.16

Table 13.13
Mean delay times (min/veh) in network A1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.35 0.07	0.40 0.06	0.43 0.06	0.51 0.10
50	isolated coordinated	0.37 0.06	0.39 0.05	0.43 0.05	0.51 0.12
40	isolated coordinated	0.37 0.06	0.39 0.05	0.42 0.05	0.49 0.12

Table 13.14
Mean delay times (min/veh) in network A2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.56 0.28	0.57 0.21	0.58 0.14	0.62 0.19
50	isolated coordinated	0.54 0.08	0.55 0.07	0.56 0.07	0.60 0.18
40	isolated coordinated	0.53 0.06	0.54 0.05	0.55 0.06	0.59 0.16

Table 13.15
Mean delay times (min/veh) in network B1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.51 0.02	0.76 0.00	0.84 0.10	0.76 1.05
50	isolated coordinated	0.50 0.02	0.48 0.00	0.54 0.08	0.72 1.00
40	isolated coordinated	0.44 0.02	0.46 0.00	0.53 0.11	0.71 0.92

Table 13.16
Mean delay times (min/veh) in network B2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.60 0.09	0.63 0.02	0.64 0.03	0.71 0.41
50	isolated coordinated	0.56 0.07	0.55 0.01	0.60 0.05	0.69 0.32
40	isolated coordinated	0.52 0.03	0.54 0.00	0.59 0.08	0.69 0.40

Table 13.17
Mean delay times (min/veh) in network C1, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.94 0.07	1.05 0.11	1.36 0.65	1.48 2.31
50	isolated coordinated	0.91 0.02	0.95 0.11	0.98 0.51	1.31 2.71
40	isolated coordinated	0.74 0.03	0.80 0.10	0.94 0.55	1.49 1.44

Table 13.18
Mean delay times (min/veh) in network C2, under different levels of traffic congestion

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	1.32 0.29	1.33 0.09	1.36 0.02	1.37 0.05
50	isolated coordinated	1.18 0.02	1.16 0.01	1.18 0.02	1.25 0.05
40	isolated coordinated	1.08 0.02	1.11 0.01	1.16 0.02	1.26 0.06

Figure 13.7
Mean journey speeds in network A1

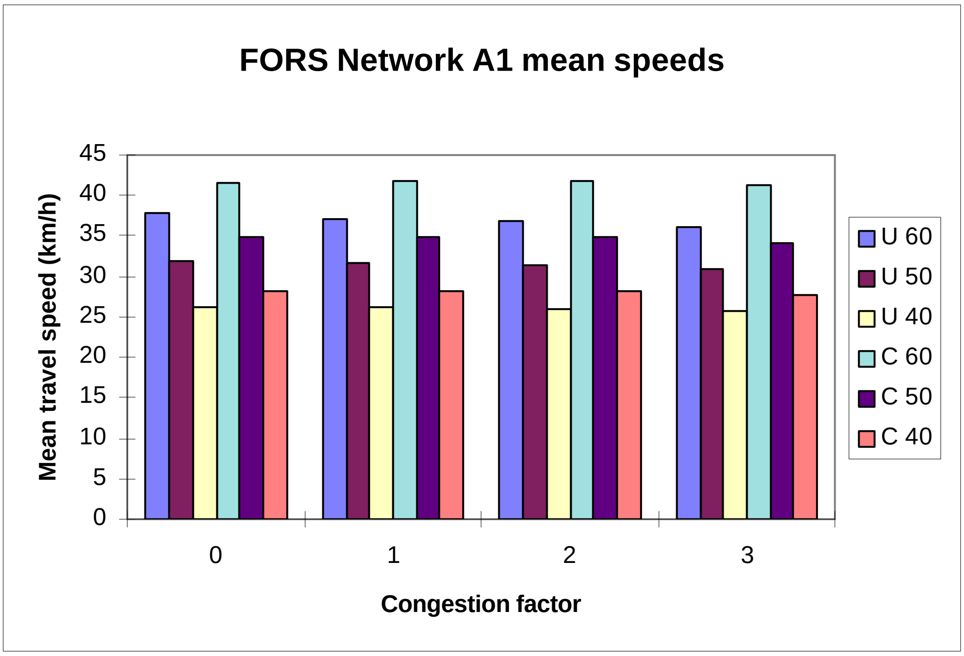


Figure 13.8
Mean journey speeds in network A2

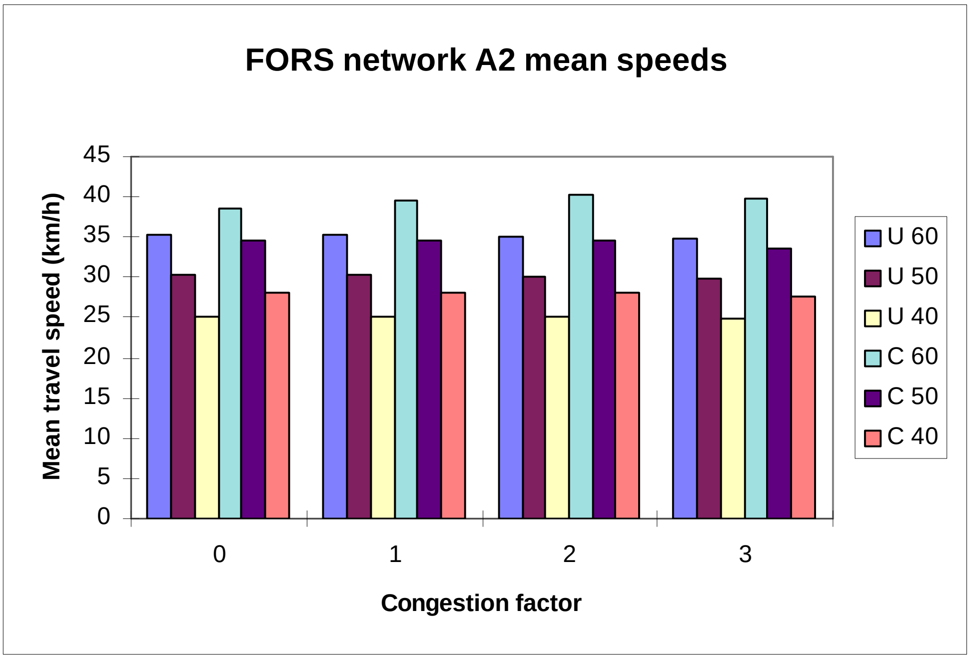


Figure 13.9
Mean journey speeds in network B1

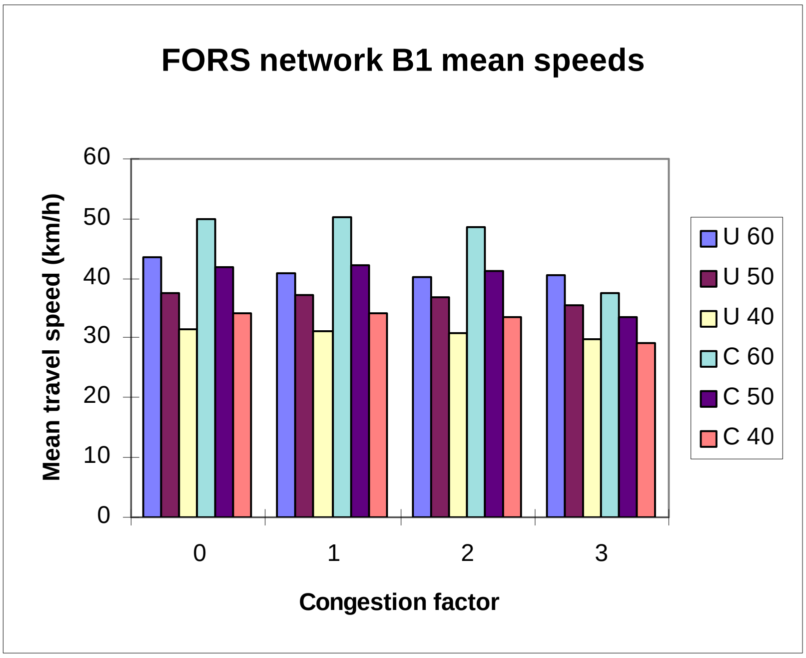


Figure 13.10
Mean journey speeds in network B2

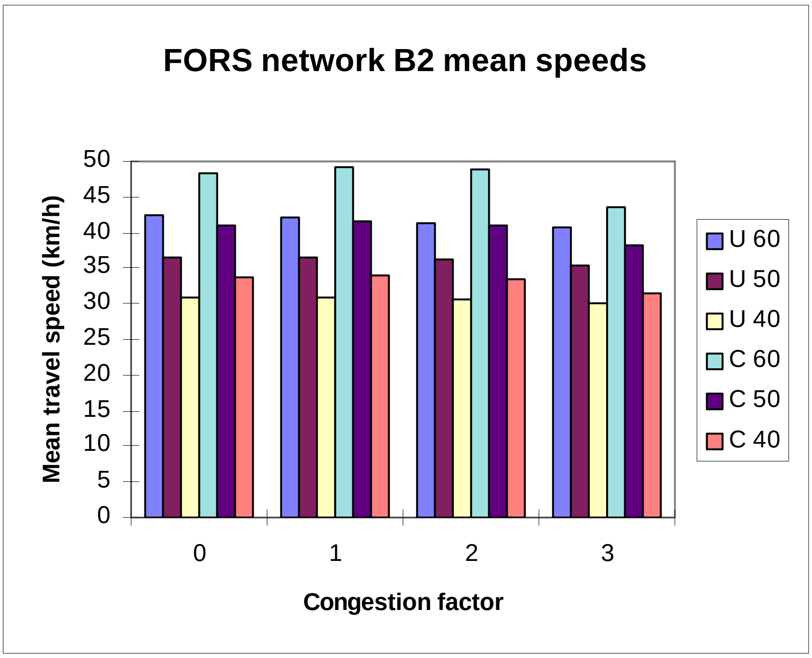


Figure 13.11
Mean journey speeds in network C1

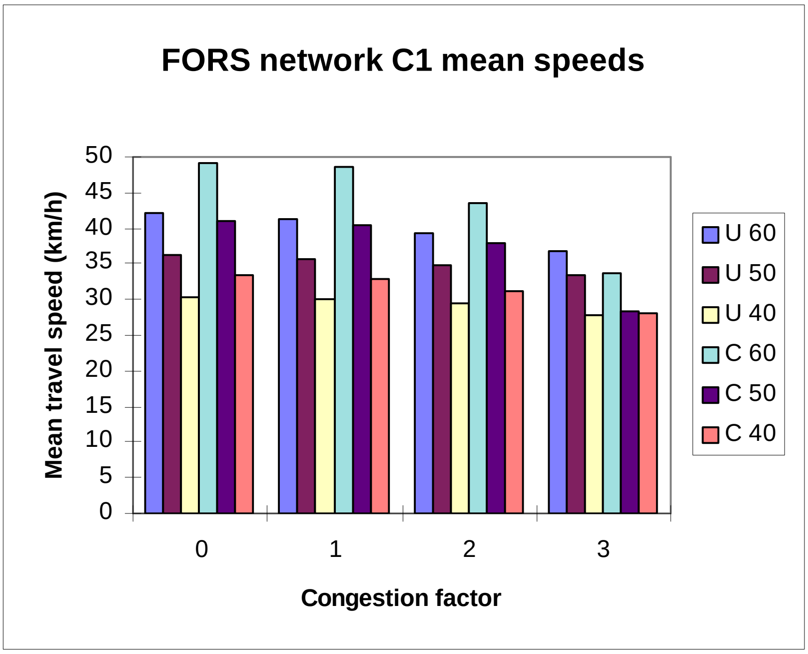
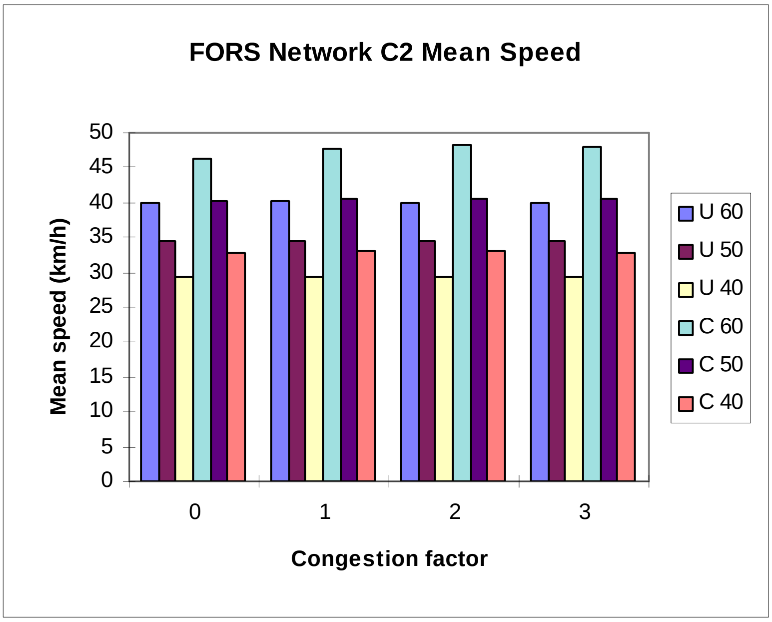


Figure 13.12
Mean journey speeds in network C2



The mean travel times in the networks are shown in Figures 13.13-13.14 (networks A1 and A2), Figures 13-15-13.16 (networks B1 and B2), and Figures 13.17-13.18 (networks C1 and C2). Clearly, the travel times under higher speed limits (e.g. 60 km/h) are less than those for the 40 km/h limit. Signal coordination improves travel times, although under the highest congestion levels the improvements become smaller (e.g. see Figures 13.16 and 13.17) or, in one case (Figure 13.15, for network B1 with congestion factor 3), may be inferior to the isolated signal control case. Signal coordination is generally of considerable value, and for instance in most cases coordinated signals with a 50 km/h speed limit yield superior travel times to those for isolated signal control and a 60 km/h limit.

The results for the mean journey delays are also of interest. Figures 13.19-13.24 show these, for the networks A1, A2, B1, B2, C1 and C2 respectively. A general result, seen for most of the model runs in all networks and at all congestion levels, is that delay times are reduced under the lower speed limits. Now, care needs to be taken to review the definition of delay time - see equation (3.11) and Section 3.2.2. The delay time reported by TrafikPlan is the system delay, which is the excess of the actual travel time for a link or route above the free flow travel time for that link or route. The model is thus suggesting that this amount of excess travel time compared to the free flow travel time, is less for the lower speed limits than it is for the 60 km/h limit. This result applies for both isolated and coordinated signal control, and the only exceptions from the model runs are seen in Figure 13.19 (network A1, isolated control at congestion factor 0 and coordinated control at congestion factor 3), Figure 13.21 (network B1, coordinated control at congestion factor 2), Figure 13.22 (network B2, coordinated control at congestion factors 2 and 3) and Figure 13.23 (network C1, coordinated control at congestion factors 2 and 3).

The inference is that, under the adopted definition of delay, delays are reduced at lower speed limits, even though travel times are higher. This suggests smoother progression of traffic flow is being achieved at the lower speed limits. The complication in using this results is one of driver perception. Driver compliance with a lower speed limit might be assisted by an indication of this improved evenness of progression along a road?

Figure 13.13
Mean travel times in network A1

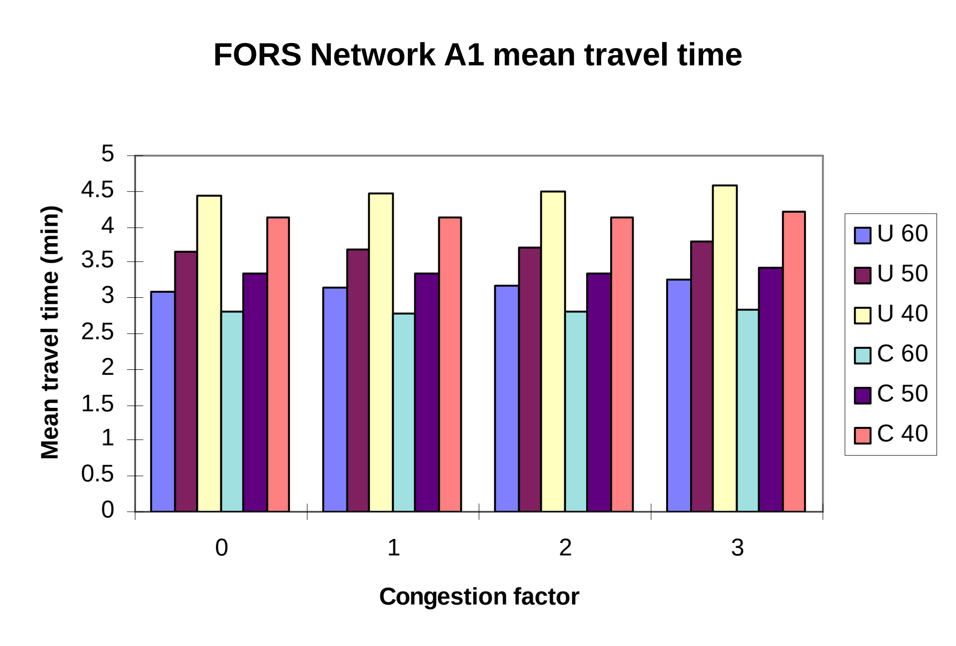


Figure 13.14
Mean travel times in network A2

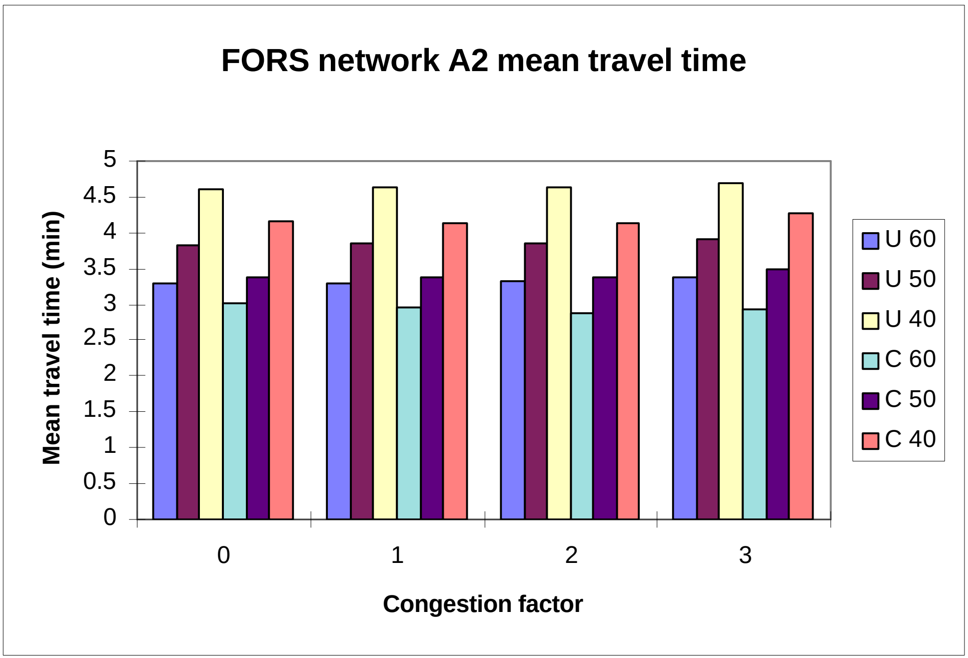


Figure 13.15
Mean travel times in network B1

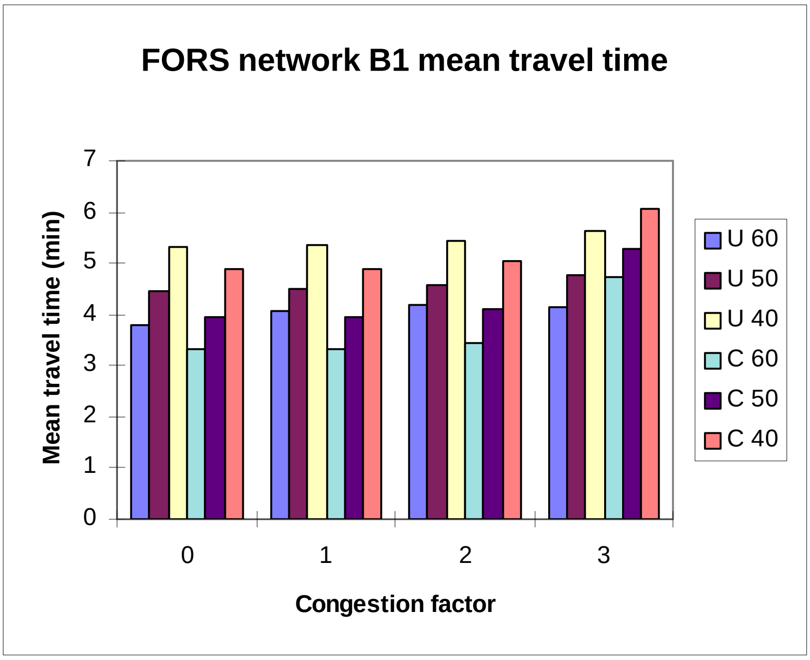


Figure 13.16
Mean travel times in network B2

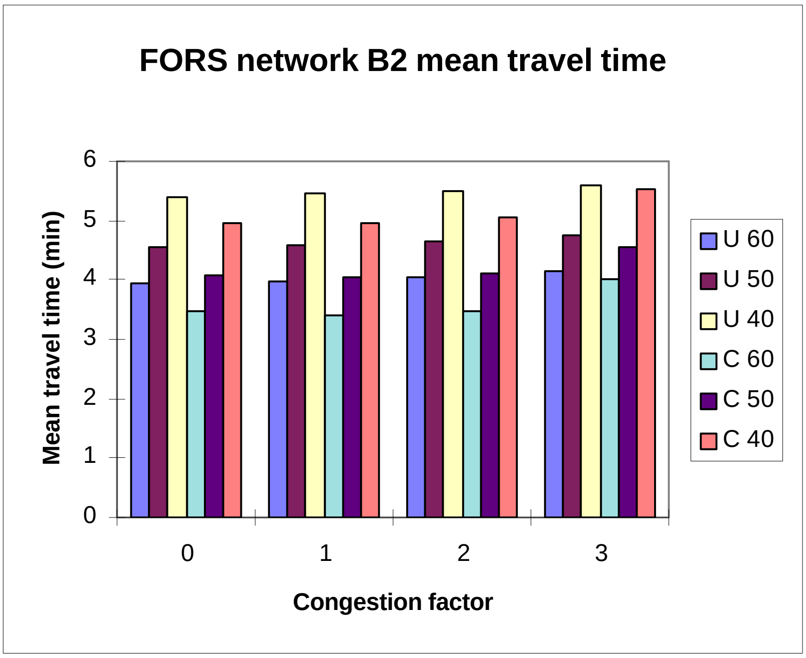


Figure 13.17
Mean travel times in network C1

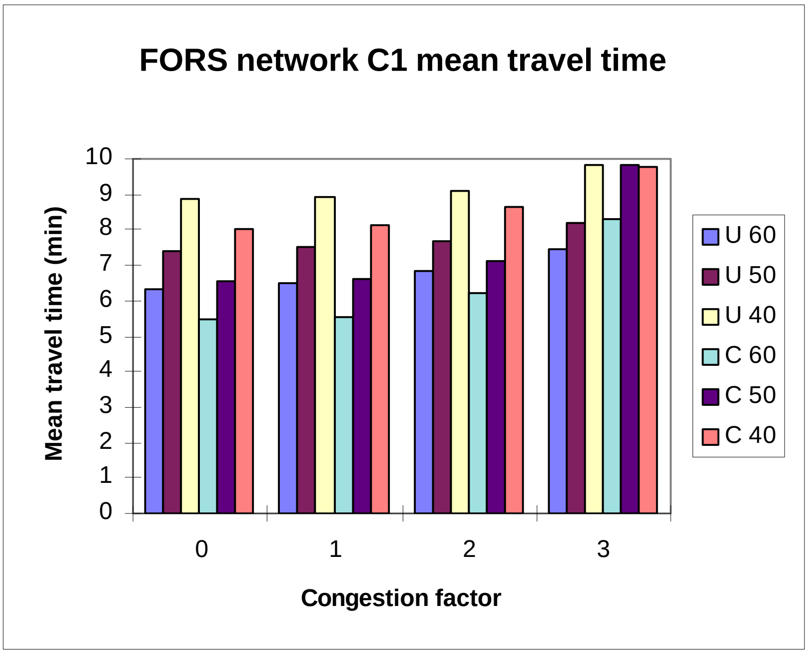


Figure 13.18
Mean travel times in network C2

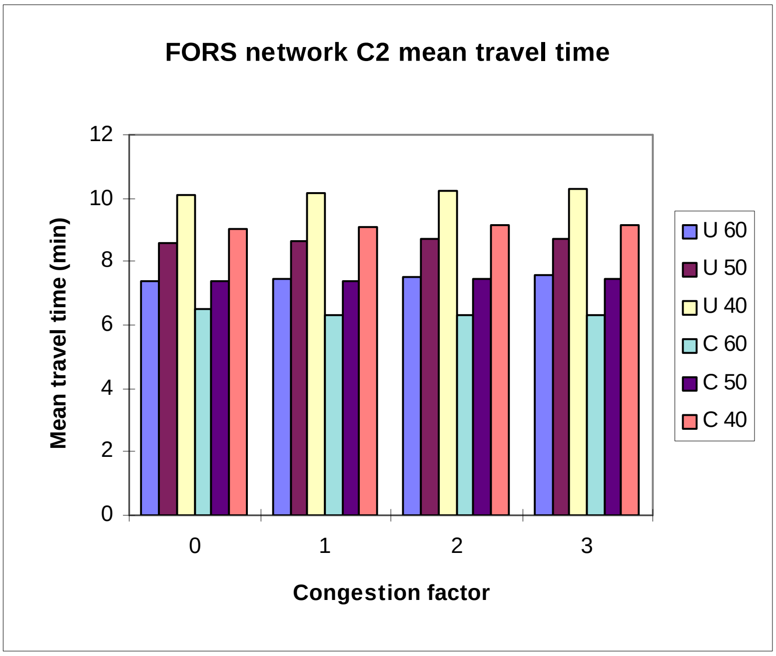


Figure 13.19
Mean vehicle delays in network A1

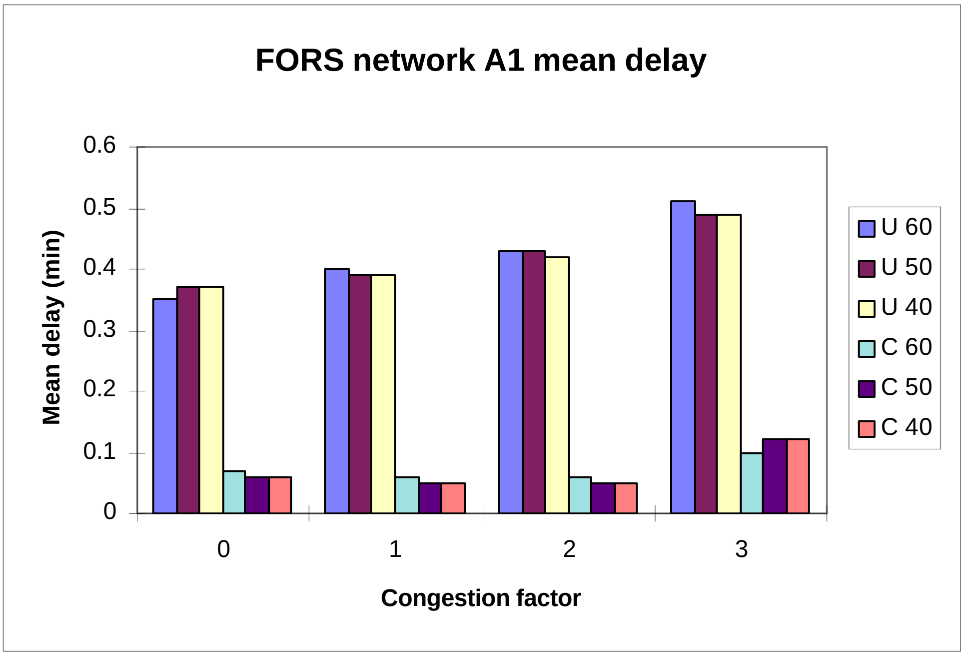


Figure 13.20
Mean vehicle delays in network A2

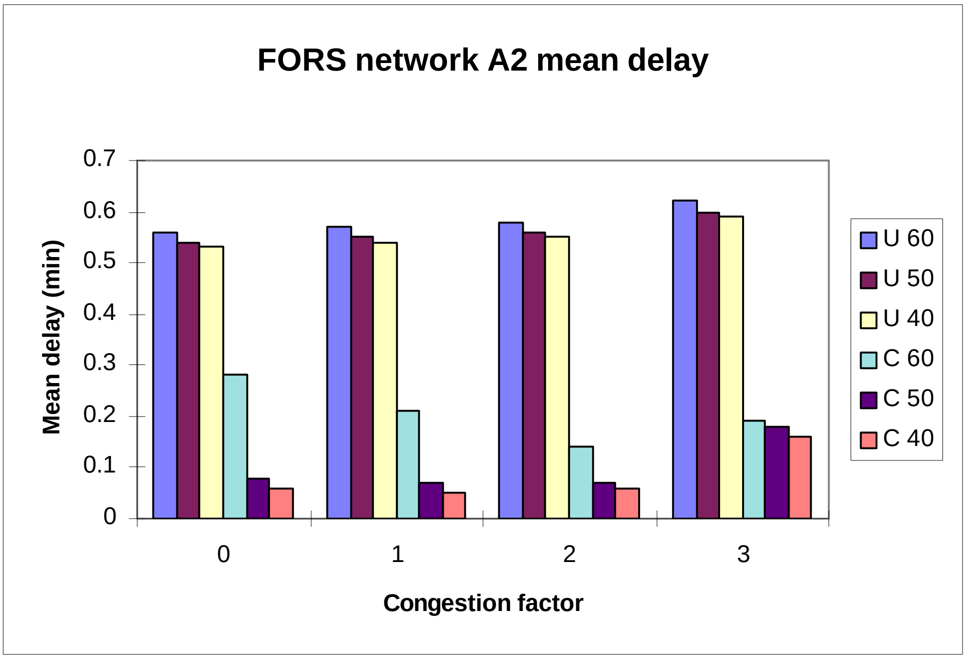


Figure 13.21
Mean vehicle delays in network B1

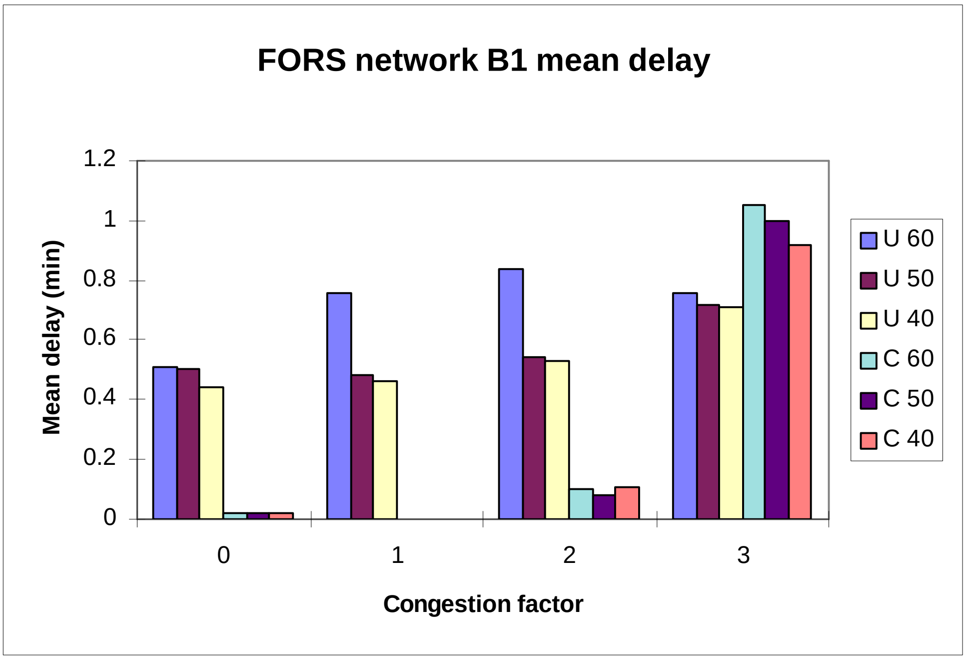


Figure 13.22
Mean vehicle delays in network B2

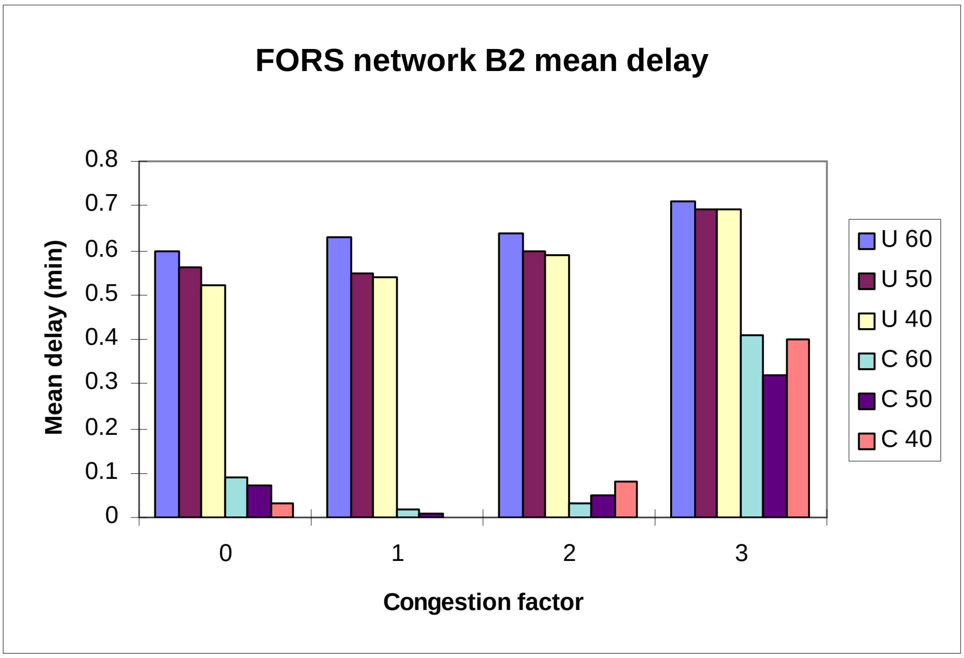


Figure 13.23
Mean vehicle delays in network C1

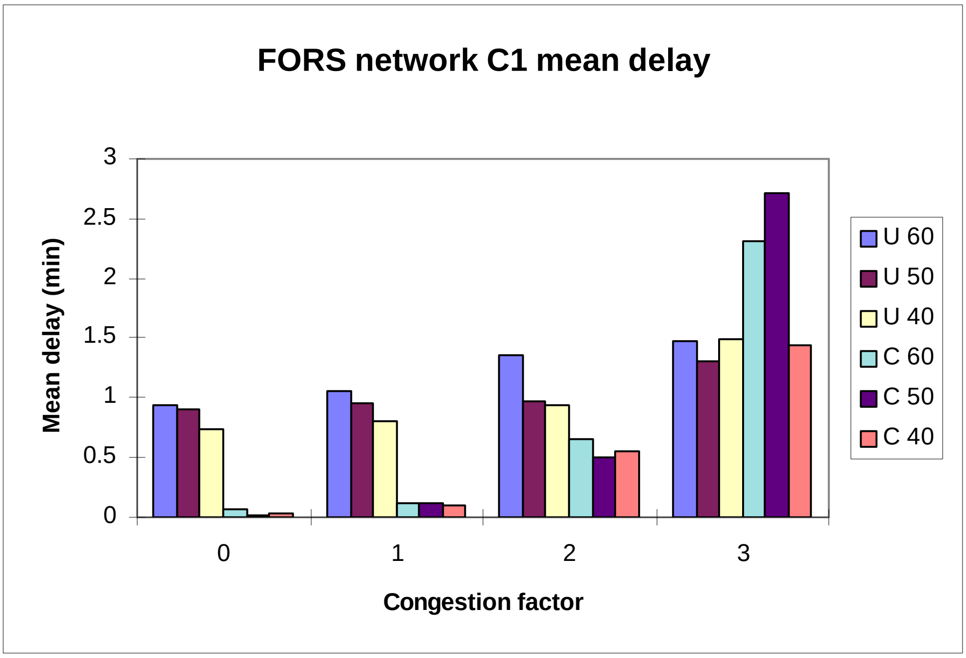
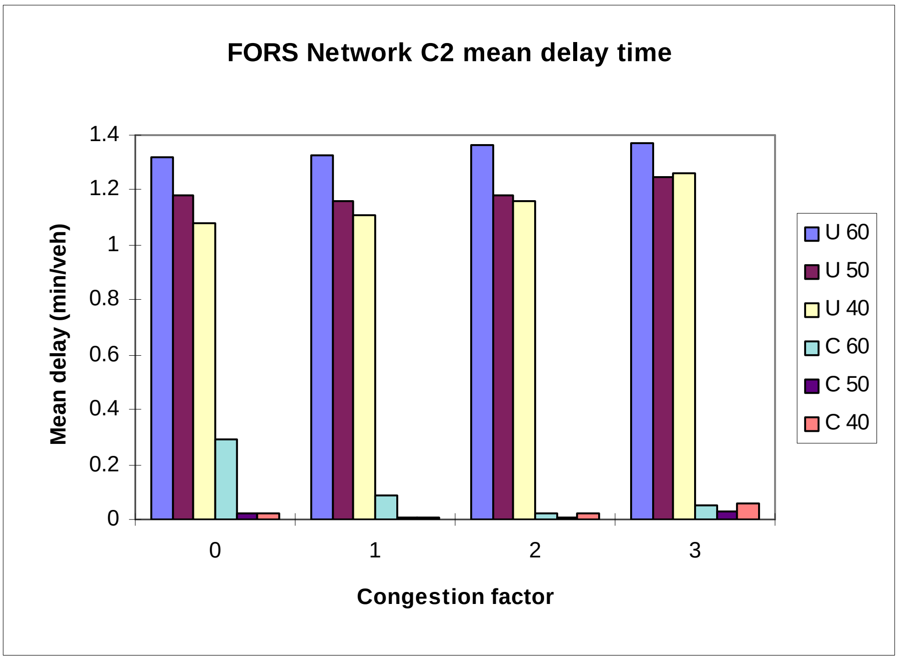


Figure 13.24
Mean vehicle delays in network C2



13. 2 Link-based traffic parameters for the network

More information on the relative performances of the six synthetic networks can be gauged by considering the link-based parameters of traffic flow, as introduced in Sections 11.2.2. and 11.2.3. Six parameters were identified, and subsequently used in examination of the synthetic networks:

1. unit travel time (u, min/km) on a link
2. congestion index (CI)
3. congestion index relative to free flow travel time at 60 km/h speed limit (CI60)
4. change in free flow travel time compared to that at 60 km/h speed limit ($\Delta t_0(60)$)
5. fuel consumption (mL/km)
6. emissions of carbon monoxide (g/km)

Tables 13.19-13.24 show the values of these parameters for the links in network C2, with the full set of parameter values for all networks listed in Appendix A. The data for network C2 provide a reasonable representation of the trends in the results for all of the networks.

Link travel times, congestion indices, fuel consumption and carbon monoxide emissions rise with increasing traffic congestion, as expected. Signal coordination brings benefits, with significant reductions in travel time, fuel and emissions when compared to isolated signal control.

Lower speed limits lead to longer travel times, as indicated from the considerations of the overall network traffic performance parameters described in the previous section. In addition, fuel consumption and emissions are also higher for the lower speed limits. Some care is needed in interpreting this result, as the running speed model of Section 8.2.3 (and equations (8.8)-(8.15)) used to estimate fuel and emissions relies on empirical data about acceleration noise and variations in speed collected on arterial roads with speed limits of 60 km/h or higher, and under these conditions driving at speeds below 40 km/h may represent different traffic flow situations than might occur at similar traffic speeds if the speed limit was 50 km/h or 40 km/h? Nevertheless, this result is in keeping with other studies (e.g. Watson (1995)). The difference between the fuel and emissions rates for different speed limits under the same congestion levels are small, but systematic. The congestion index, a non-parametric measure of system delay time, is the one parameter that improves with lower speed limits, again in keeping with the overall network performance results discussed earlier.

A means to indicate the overall performance of these variables in each network for each speed limit is of importance. One possible graphical display for multiple multi-dimensional data sets is provided by the star plot diagram from the field of ‘exploratory data analysis’. The construction and use of star plots was described by Chambers *et al* (1983) - see also Taylor, Young and Bonsall (1996, pp.325-327). This graphical technique has been applied in the analysis and interpretation of the link-based parameters from the modelling runs for the synthetic networks.

The star plot represents the values of the variables along a number of polar axes, one axis representing each variable. The performance measures are drawn along each axis (I) and separate star plots are drawn for each (j) data set. In this case, there are six variables and six data sets for each network (the three speed limits and the two signal control strategies).

Table 13.19
Mean link travel times (min/km) in network C2

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	1.386 1.547	1.549 1.401	1.556 1.400	1.425 1.580
50	isolated coordinated	1.709 1.556	1.709 1.556	1.709 1.560	1.576 1.723
40	isolated coordinated	1.942 1.790	1.942 1.791	1.949 1.801	1.881 1.977

Table 13.20
Mean link congestion index (CI) in network C2

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.131 0.003	0.132 0.012	0.137 0.001	0.157 0.009
50	isolated coordinated	0.104 0.002	0.105 0.003	0.105 0.005	0.114 0.016
40	isolated coordinated	0.008 0.000	0.082 0.000	0.009 0.001	0.103 0.003

Table 13.21
Mean link relative congestion index (CI60 in network C2

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.131 0.003	0.132 0.002	0.137 0.018	0.157 0.004
50	isolated coordinated	0.268 0.152	0.268 0.153	0.268 0.155	0.278 0.166
40	isolated coordinated	0.466 0.353	0.466 0.354	0.470 0.360	0.487 0.379

Table 13.22
Mean link change in free flow travel time (from 60 km/h) (min/km) in network C2

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	0.000 0.001	0.000 0.001	0.000 0.001	0.000 0.001
50	isolated coordinated	0.122 0.130	0.122 0.130	0.122 0.130	0.122 0.130
40	isolated coordinated	0.290 0.298	0.290 0.298	0.290 0.298	0.290 0.298

Table 13.23
Mean link fuel consumption (unleaded petrol, L/km) in network C2

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	14.33 13.93	28.89 27.76	43.68 41.86	58.18 55.61
50	isolated coordinated	14.97 14.31	30.12 28.76	45.55 43.55	60.56 57.89
40	isolated coordinated	15.88 15.26	31.90 30.68	48.26 46.47	64.33 61.78

Table 13.24
Mean link carbon monoxide emissions (kg/km) in network C2

Speed limit (km/h)	Signal control	Congestion factor			
		0	1	2	3
60	isolated coordinated	2.16 2.11	4.36 4.21	6.59 6.35	8.78 8.44
50	isolated coordinated	2.26 2.18	4.54 4.38	6.86 6.64	9.12 8.83
40	isolated coordinated	2.36 2.31	4.74 4.64	7.17 7.02	9.55 9.34

The length z_{ij} along axis i of star plot j represents the value of variable x_{ij} when compared over all of the stars, when

$$z_{ij} = \frac{(1-c)\left(x_{ij} - \min_i\{x_{ij}\}\right)}{\max_i\{x_{ij}\} - \min_i\{x_{ij}\}} + c$$

and the constant c is a scaling factor ($0 \le c < 1$).

The star plot then enables rapid visual comparisons to be made between the different networks, including the changes in specific variables between networks.

Figure 13.25 presents the six axes used for the star plots for the synthetic networks. The six axes radiate out from a central point, and the values of z_{ij} are plotted on each axis. For present purposes, the larger the value of z_{ij}, the worse the performance of the network for that parameter. In the star plot this means that the larger the area of the star, the poorer the performance of that network, Further the smaller the length of an individual axis in the star plot, the better the performance of that network for that variable plotted on that axis.

Figure 13.25: Representative star plot for comparisons, showing axis labels

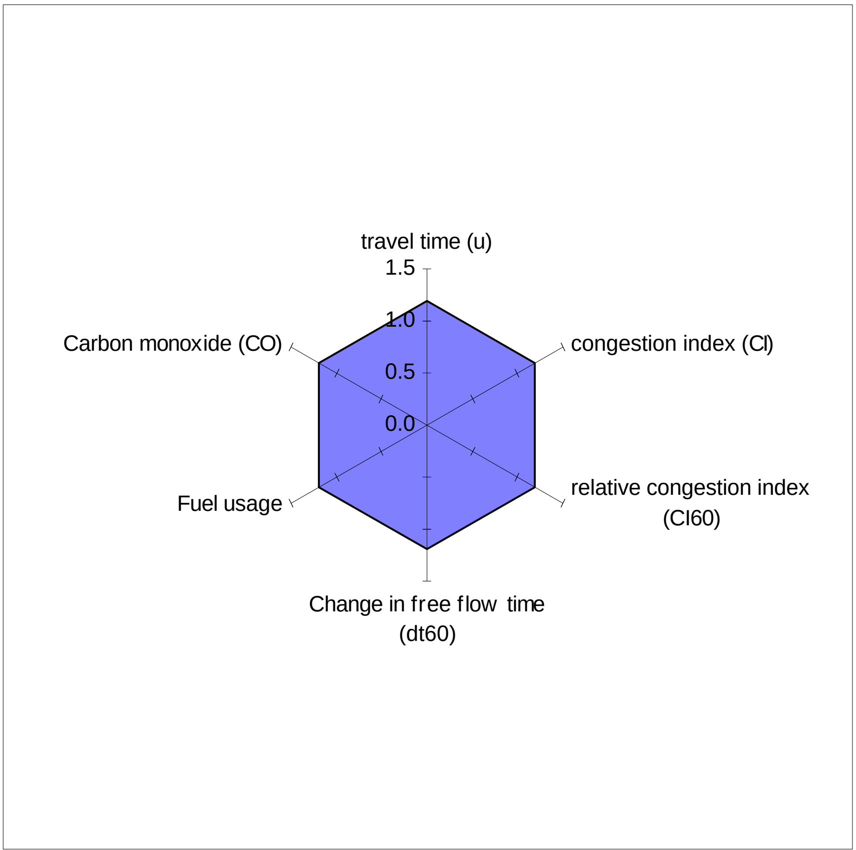


Figure 13.25 shows each of the six axes drawn to its maximum value of z_{ij}. It is intended only to provide a reference plot the star plots developed for the outputs from the different networks and model runs.

Star plots based on the model of Figure 13.25 were then drawn for each network, showing stars for each of the three speed limits (40, 50 and 60 km/h) and for isolated or coordinated signal control in each case. For each network separate star plots were constructed for each of the four congestion levels.

Thus Figures 13.26-13.29 show the star plots for network A1 with congestion factors 0, 1, 2 and 3 respectively. In each case the 60 km/h speed limit with coordinated signals gives the best performance for all of the traffic variables except the congestion index (CI), which is best for the 40 km/h limit with signal coordination. The 50 km/h speed limit with signal coordination shows a good all-round performance.

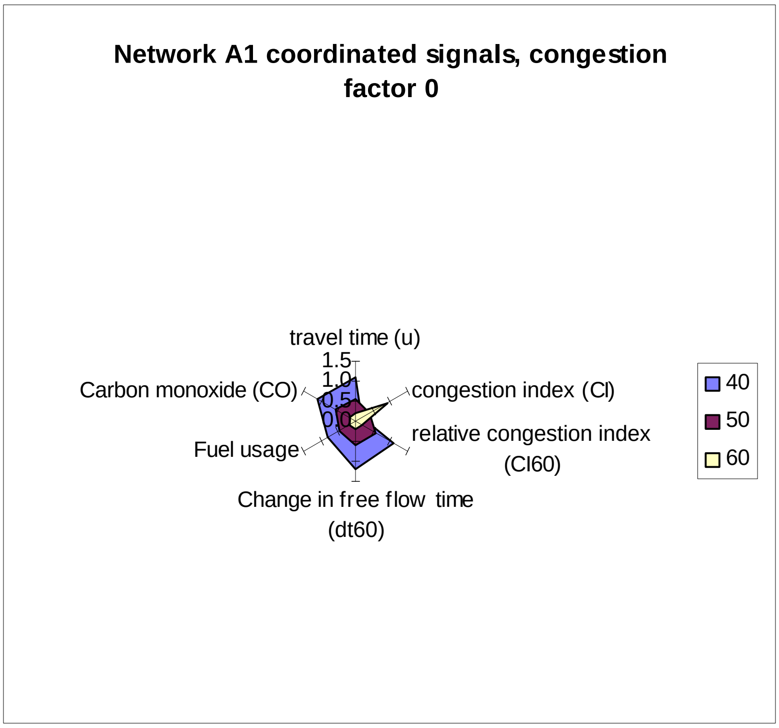
Figures 13.30-13.33 show the corresponding star plots for network A2. Similar results are found to those for network A1.

For network B1, these general trends are repeated - for congestion levels 0, 1 and 2 (see Figures 13.34-13.36). For congestion level 3 in network B1, capacity restrictions at congestion level 3 mean that the delay advantages of signal coordination are not being realised. Figure 13.37 shows this effect. The least delays are still experienced for the 40 km/h speed limit, but the best overall performances in the network under this congestion state are realised for the 60 km/h and 50 km/h speed limits, with isolated signal control.

The additional capacity available in network B2 overcomes the drop-off in performance of the coordinated signals - remember that the same travel demand patterns and intensities are applied in each of networks B1 and B2. Figures 13.38-13.41 show the plots for network B2.

Networks C1 (Figures 13.42-13.45) and C2 (Figures 13.46-13.49) show similar trends to networks B1 and B2. In network C2, the 50 km/h speed limit with signal coordination is always at least as efficient as the 60 km/h speed limit with isolated signal control. For network C1, this is also the case except for congestion level 3, where a capacity constraint similar to that for network B1 applies.

Figure 13.26: Star plots for network A1 (congestion factor 0)



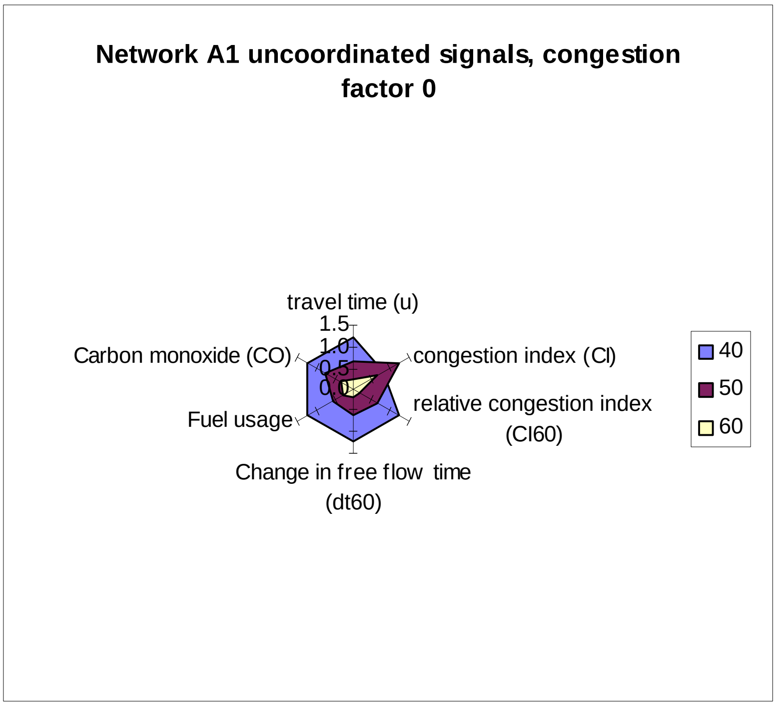
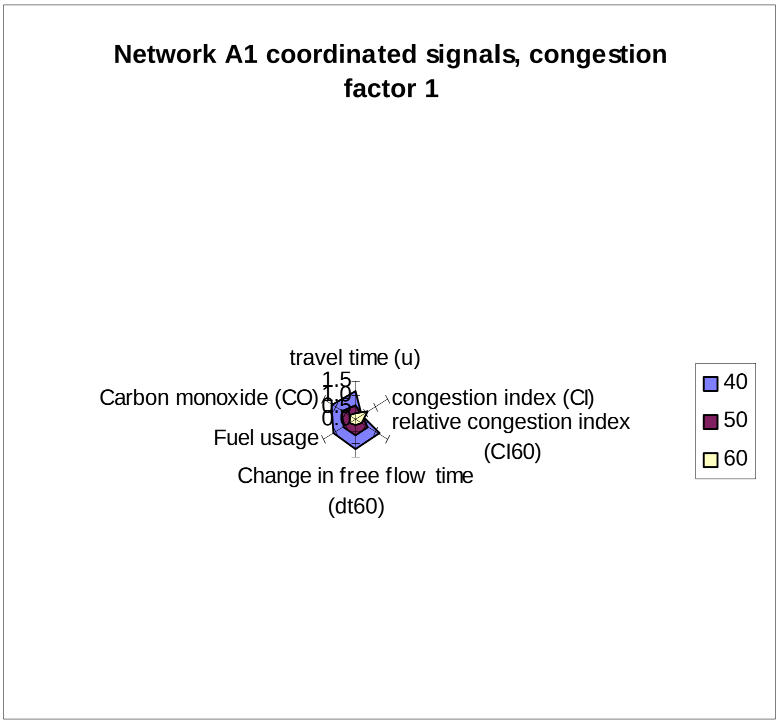


Figure 13.27: Star plots for network A1 (congestion factor 1)



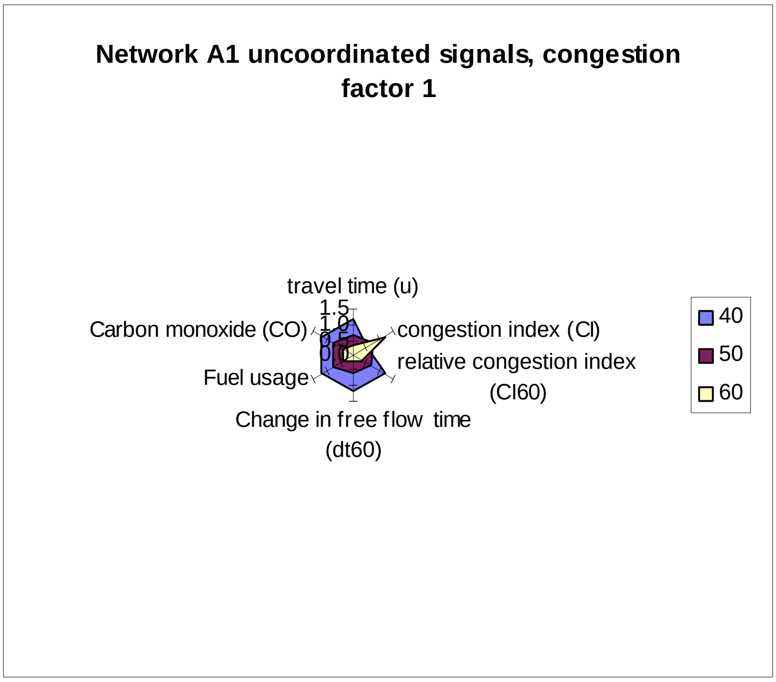
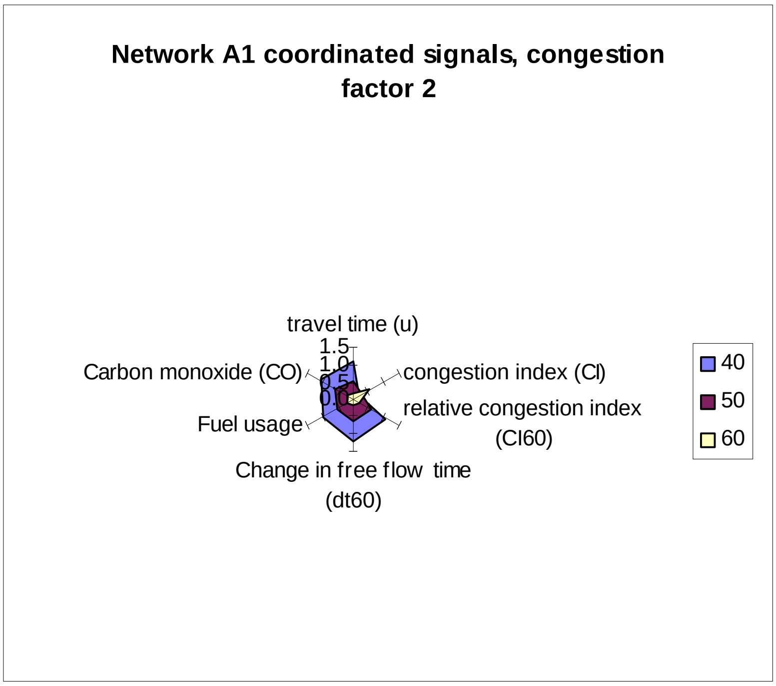


Figure 13.28: Star plots for network A1 (congestion factor 2)



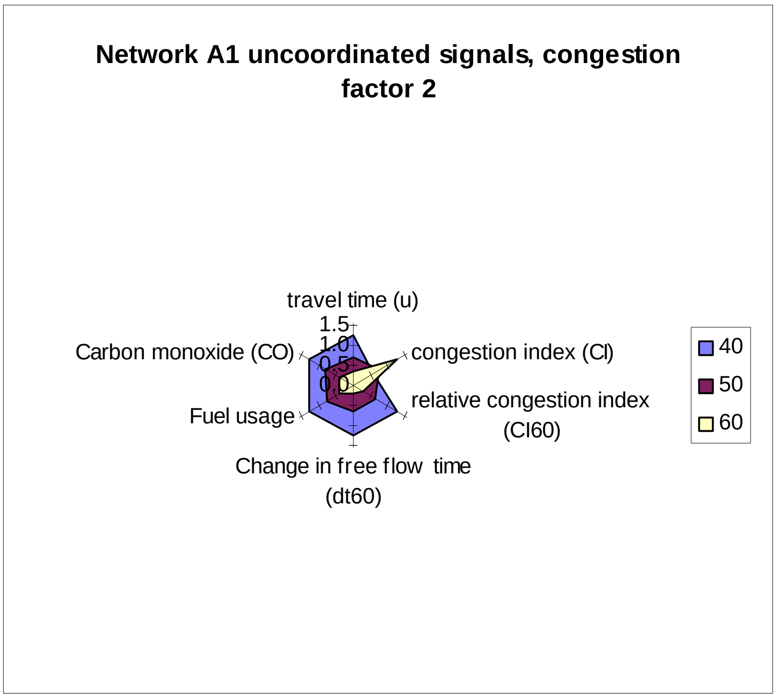
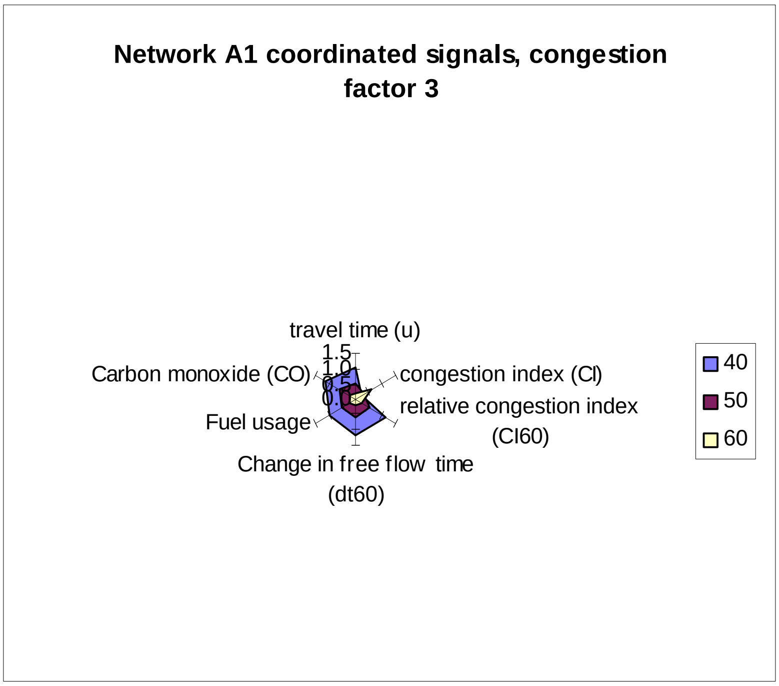


Figure 13.29: Star plots for network A1 (congestion factor 3)



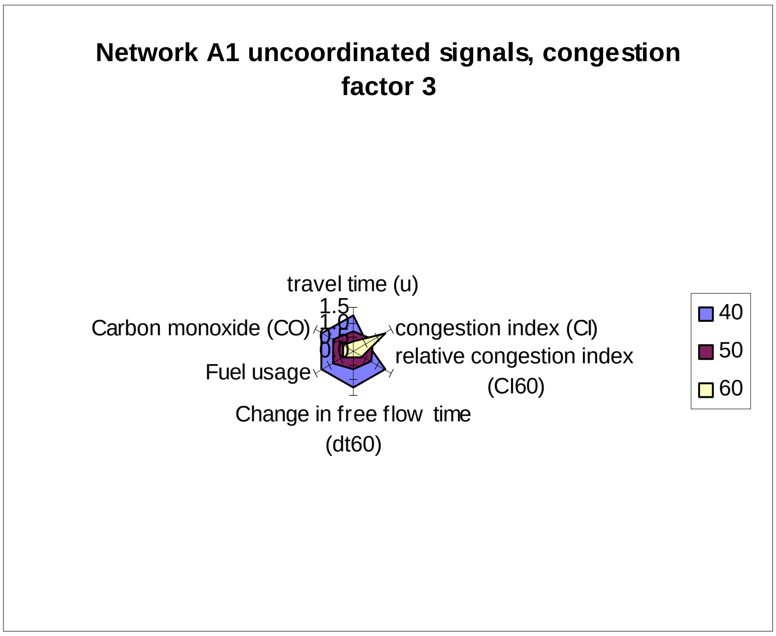
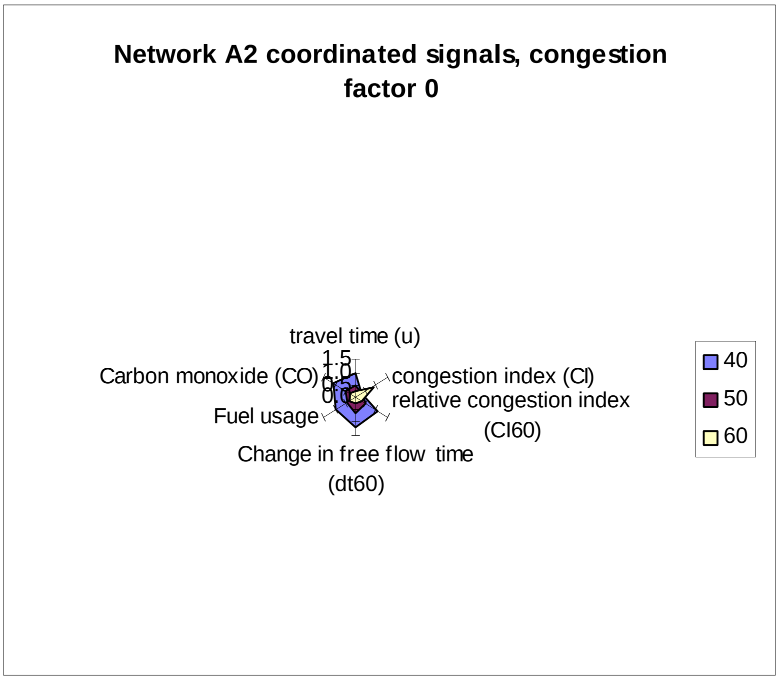


Figure 13.30: Star plots for network A2 (congestion factor 0)



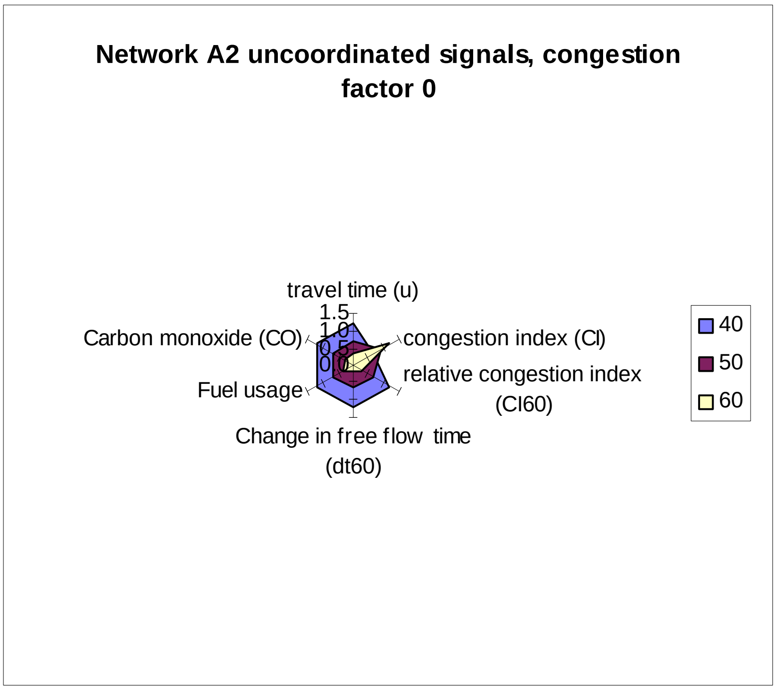
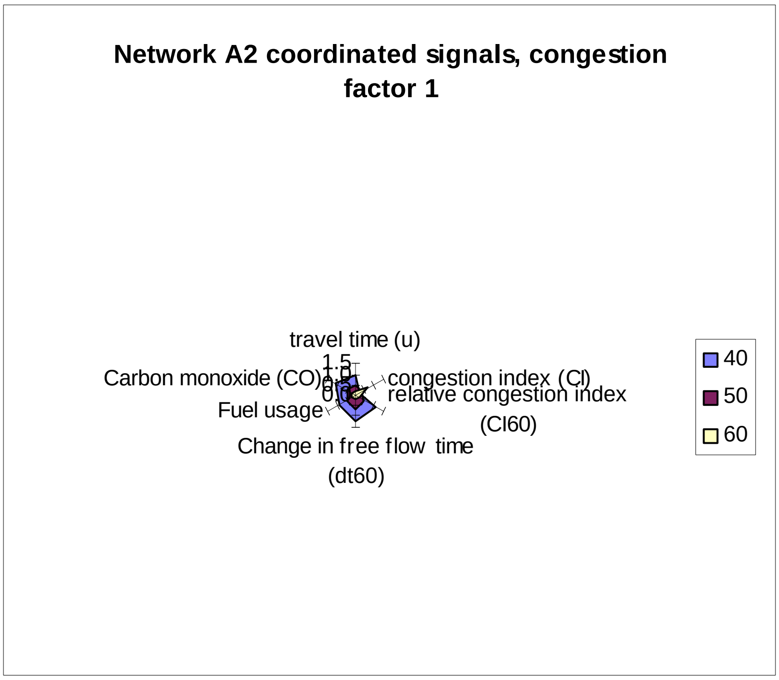


Figure 13.31: Star plots for network A2 (congestion factor 1)



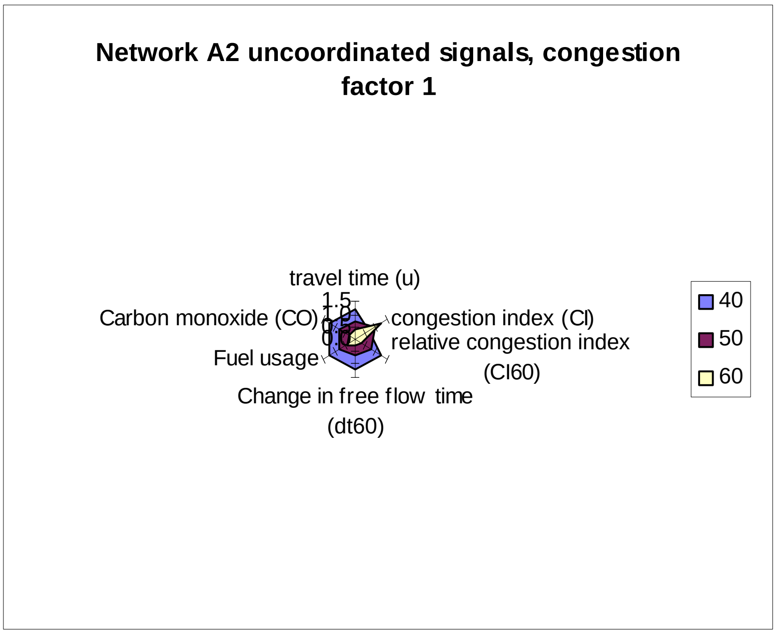
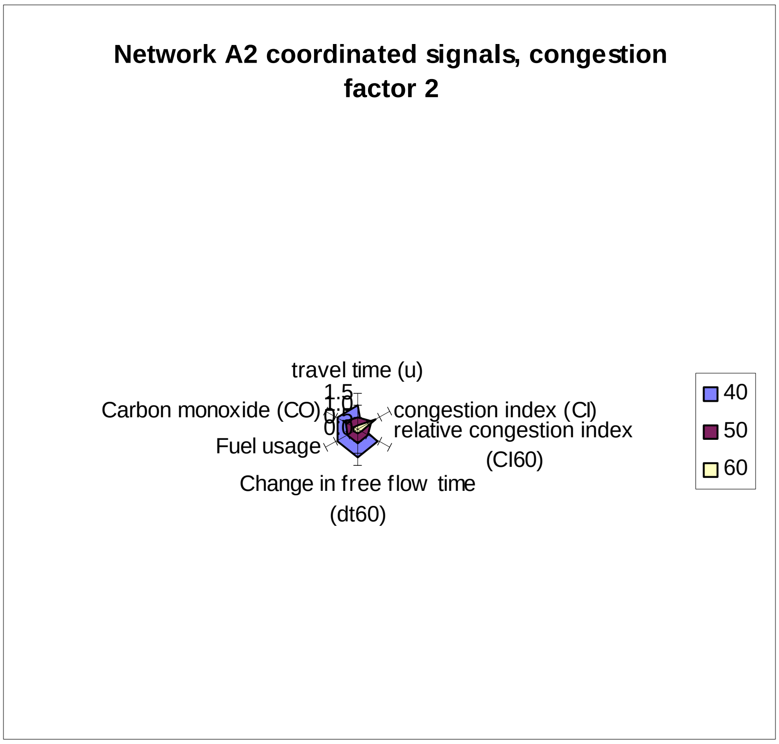


Figure 13.32: Star plots for network A2 (congestion factor 2)



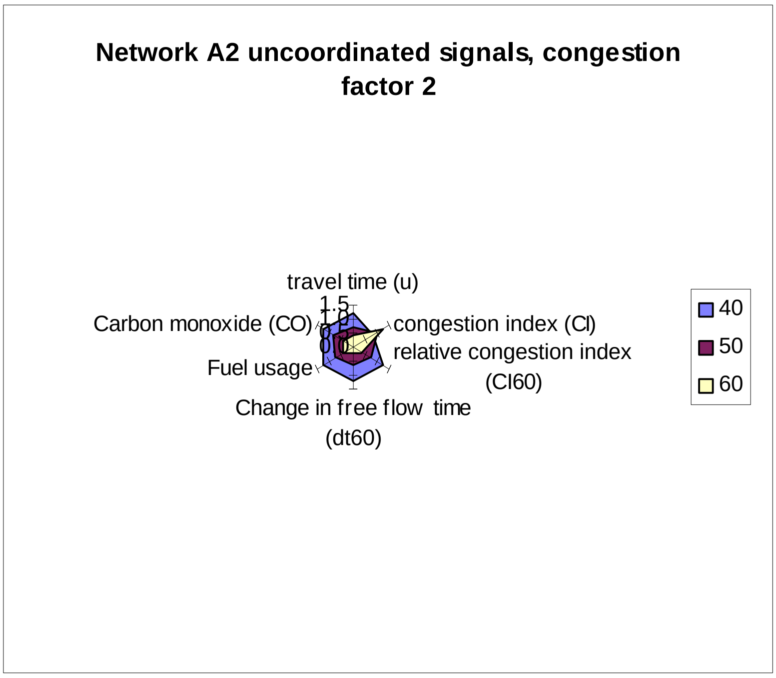
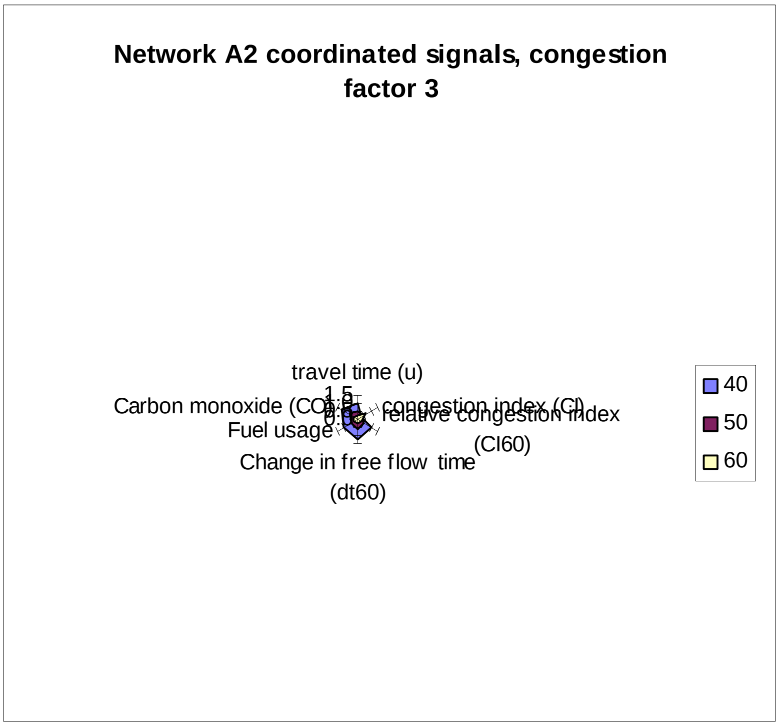


Figure 13.33: Star plots for network A2 (congestion factor 3)



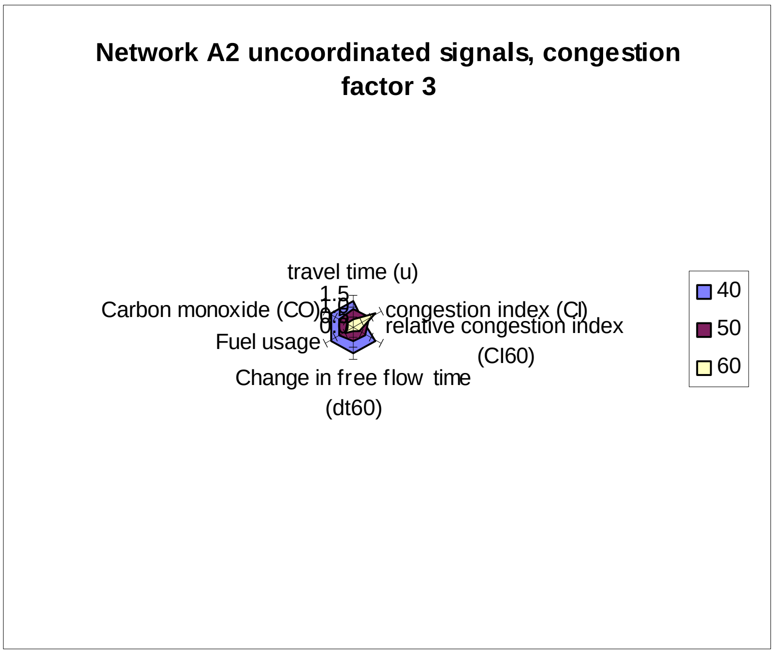
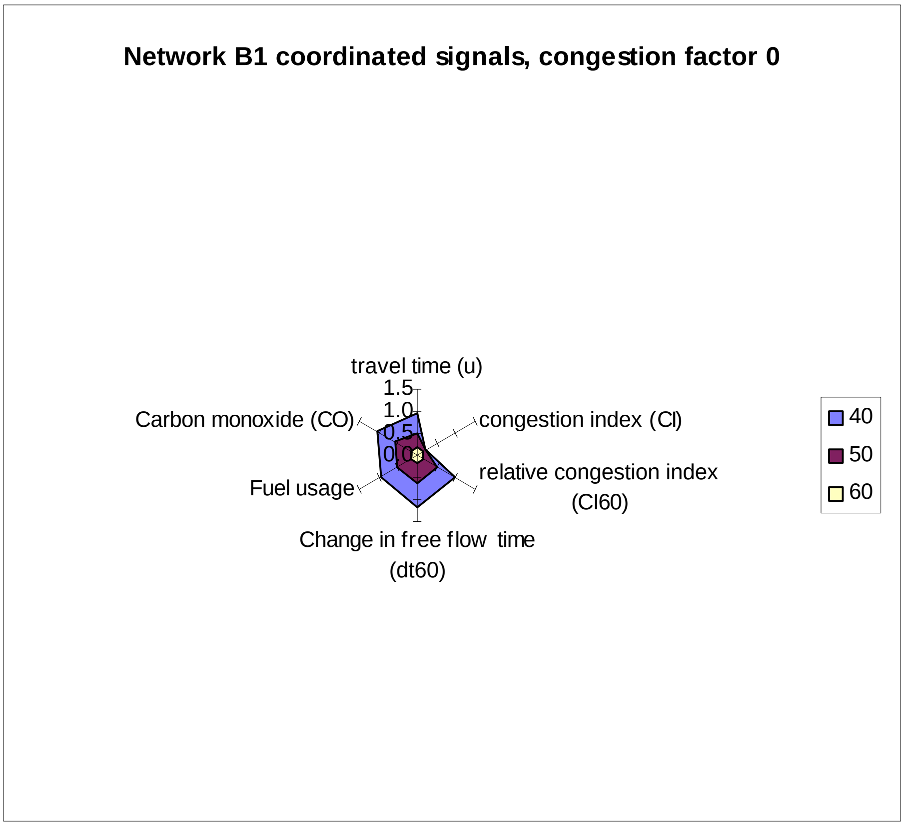


Figure 13.34: Star plots for network B1 (congestion factor 0)



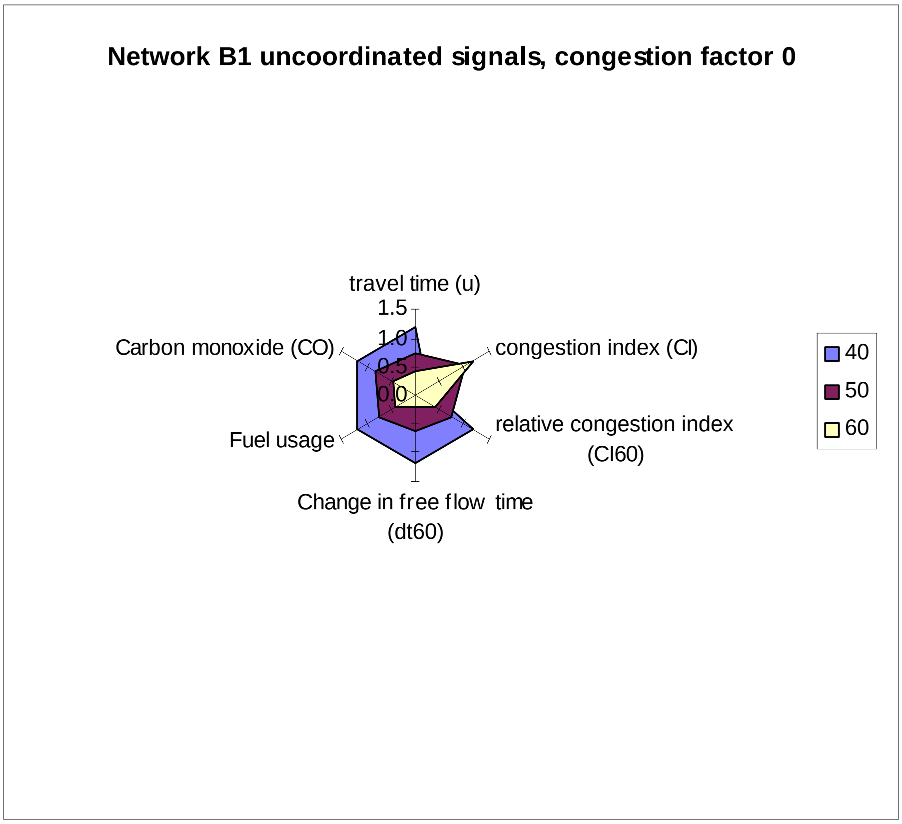
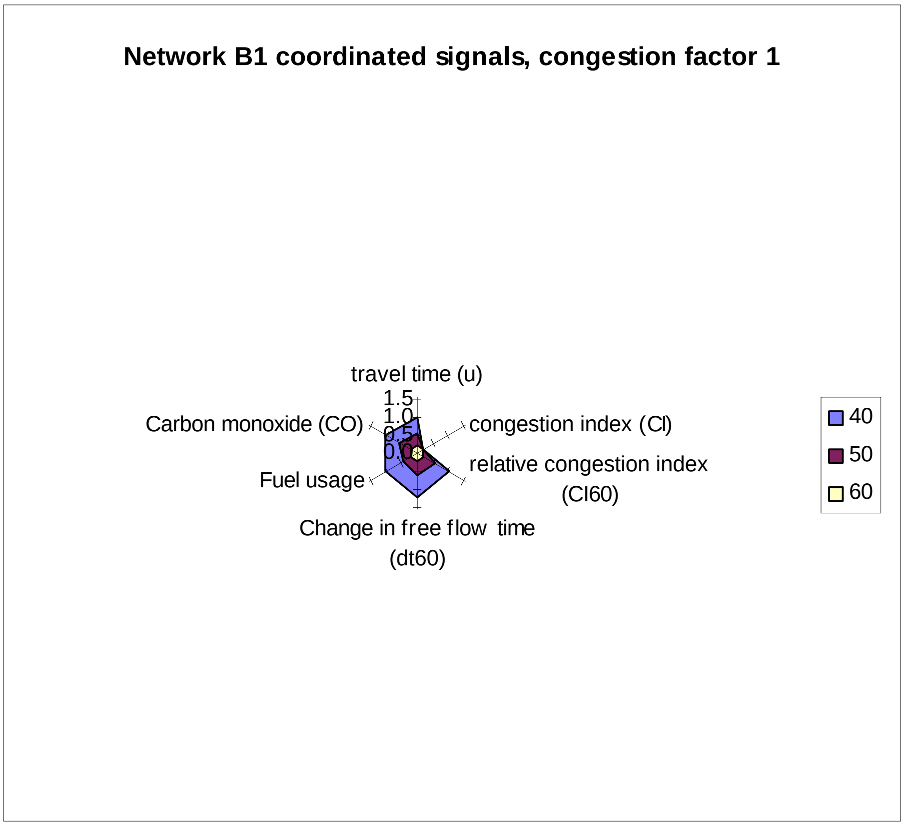


Figure 13.35: Star plots for network B1 (congestion factor 1)



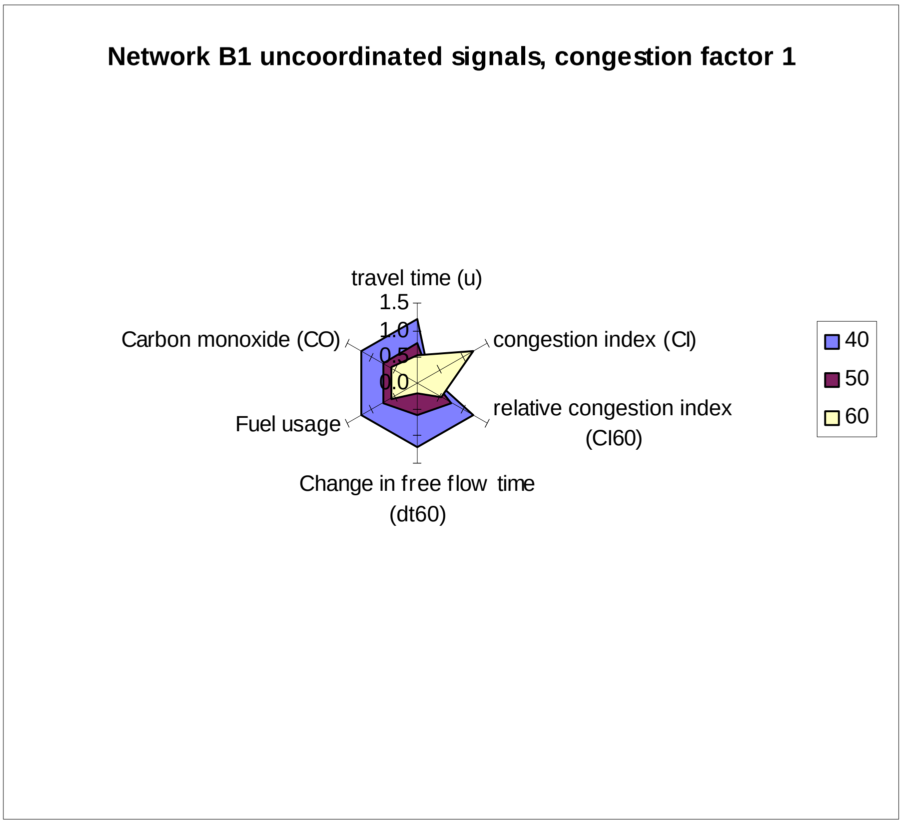
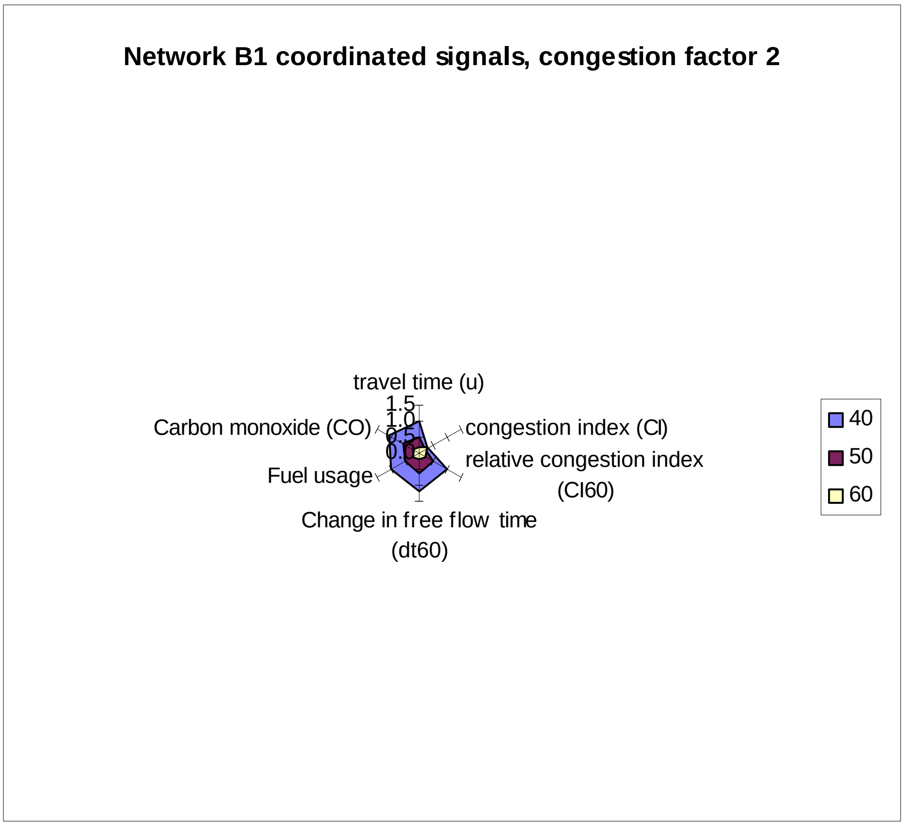


Figure 13.36: Star plots for network B1 (congestion factor 2)



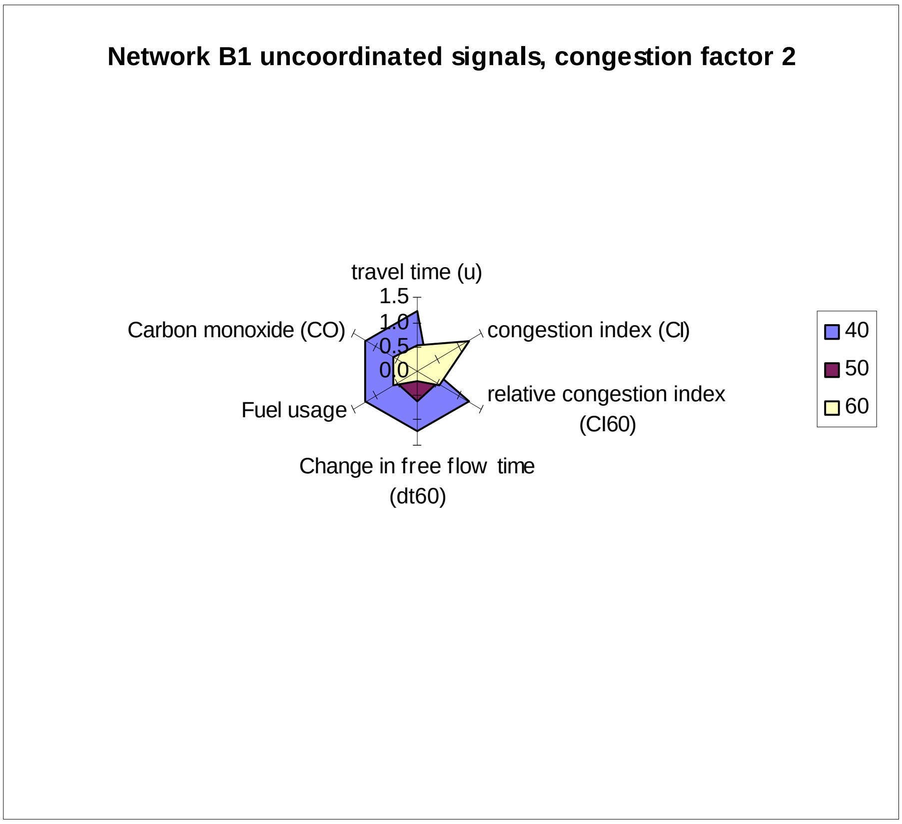
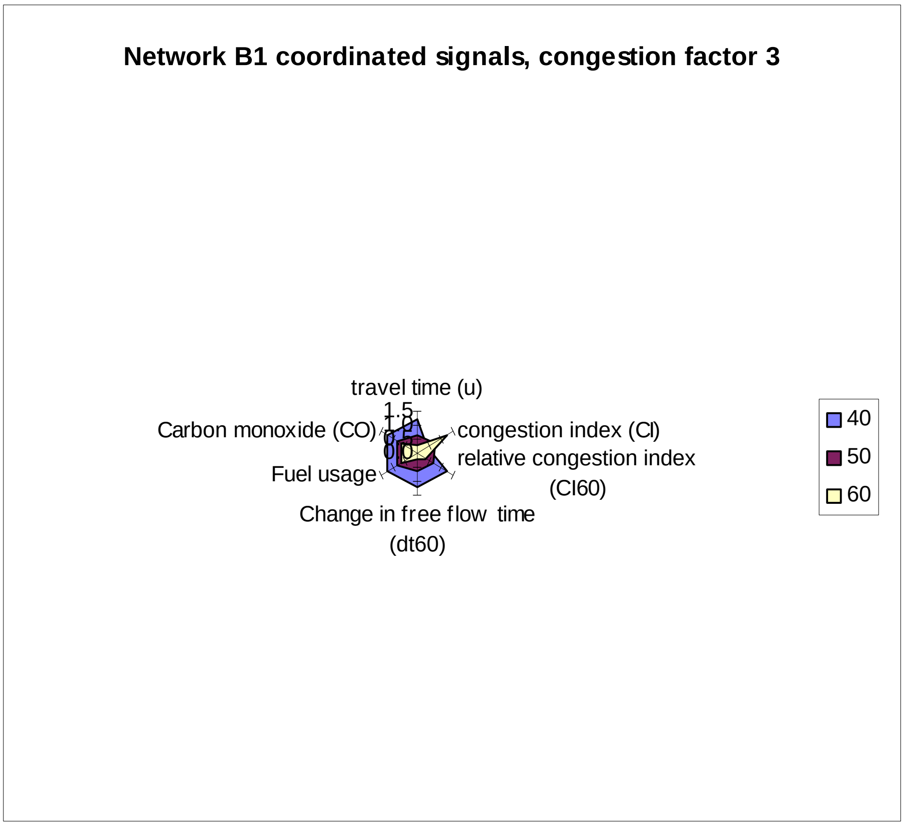


Figure 13.37: Star plots for network B1 (congestion factor 3)



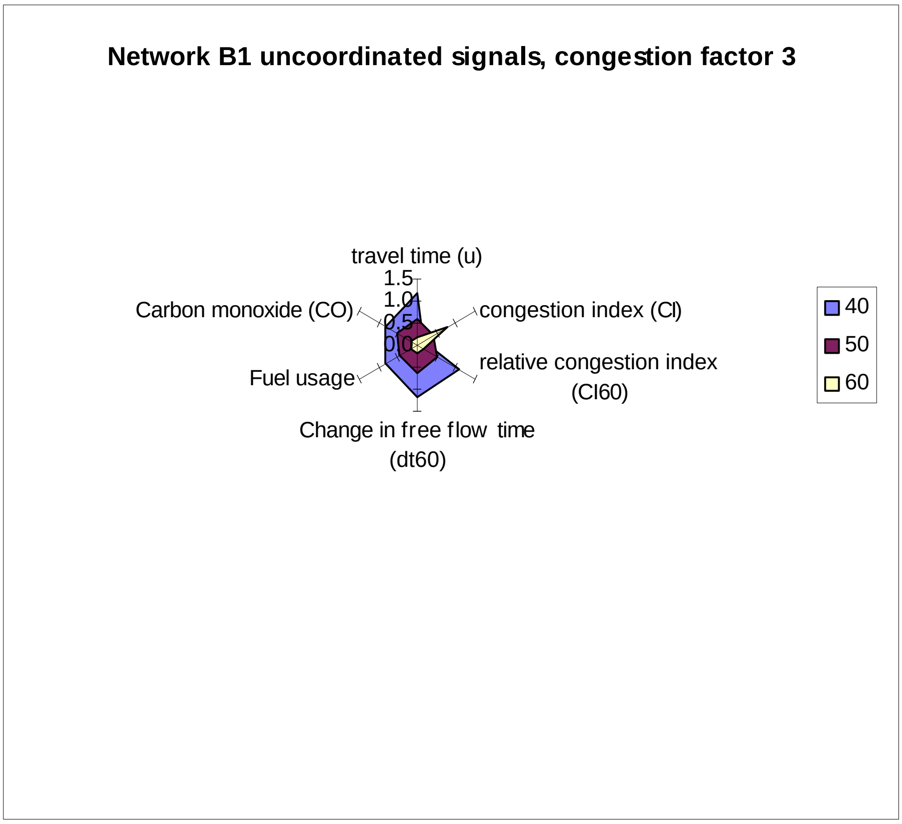
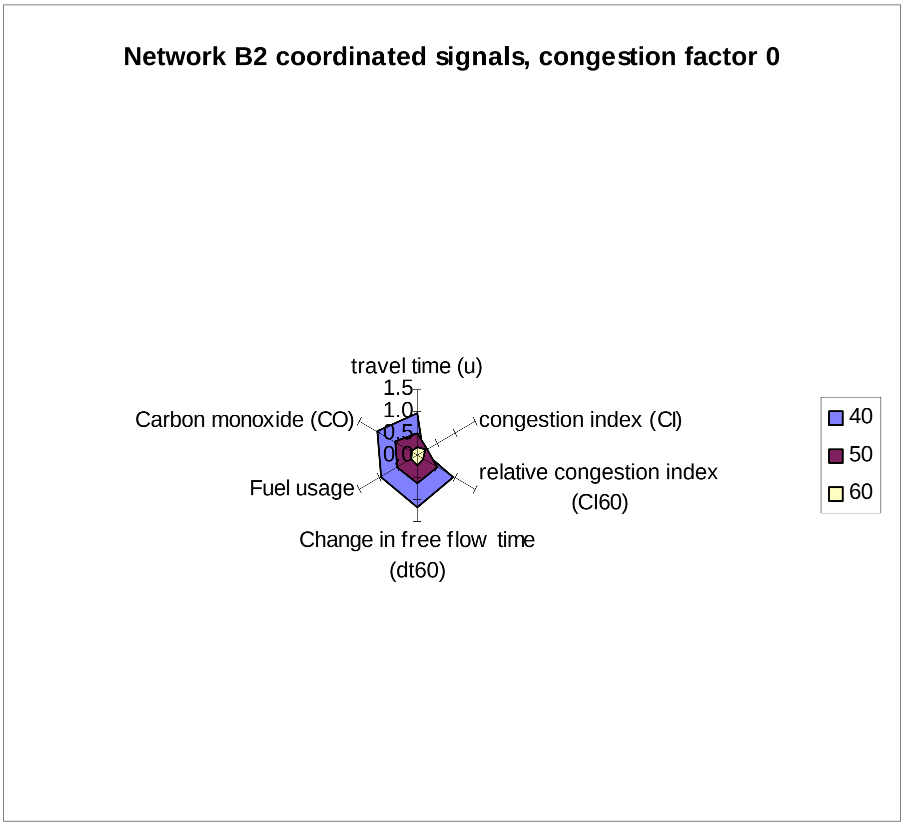


Figure 13.38: Star plots for network B2 (congestion factor 0)



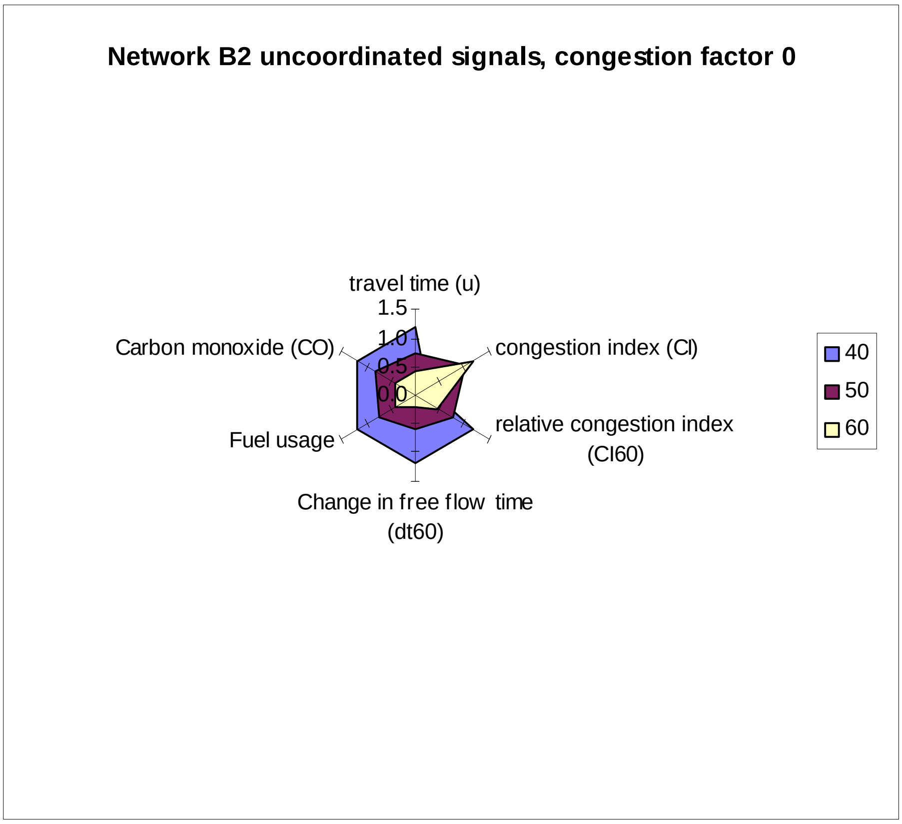
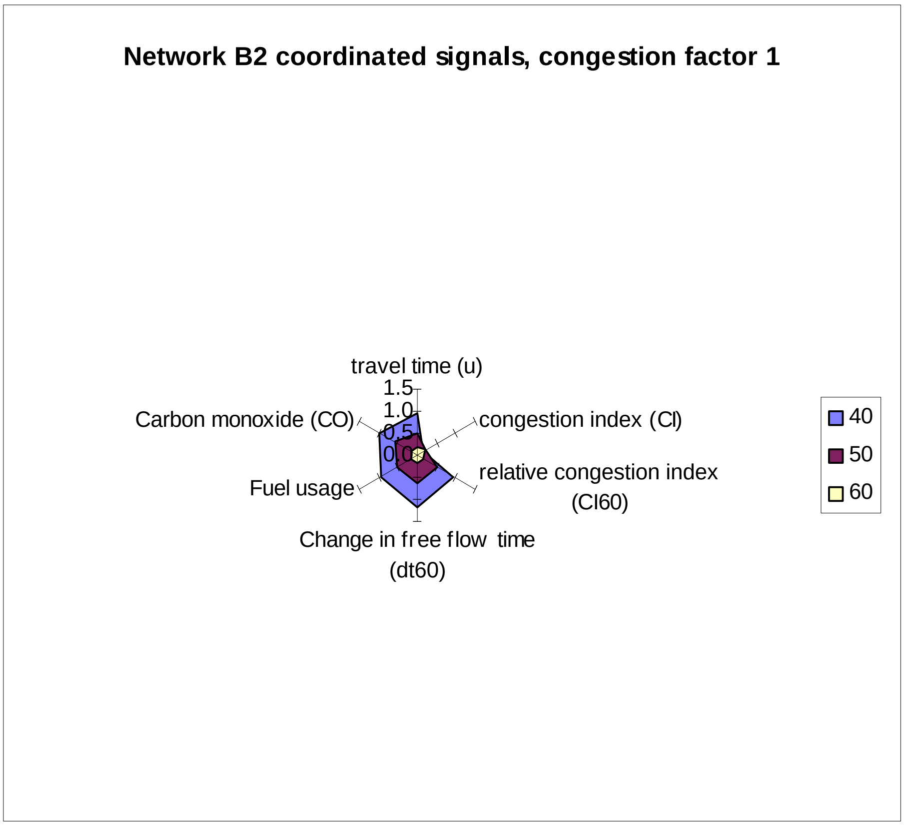


Figure 13.39: Star plots for network B2 (congestion factor 1)



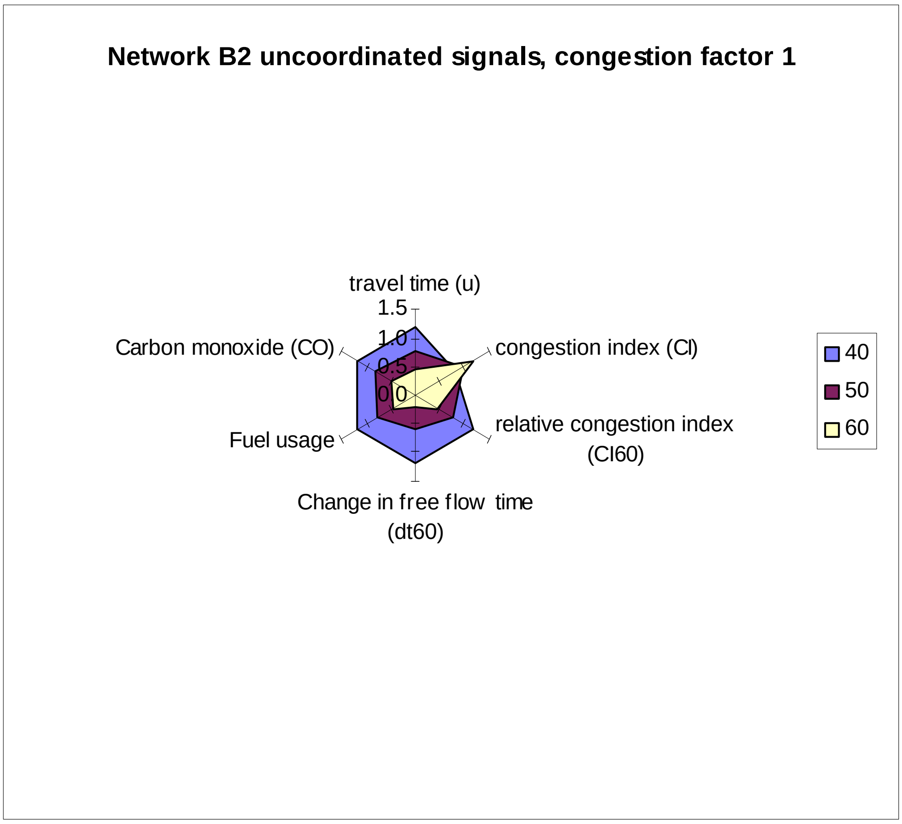
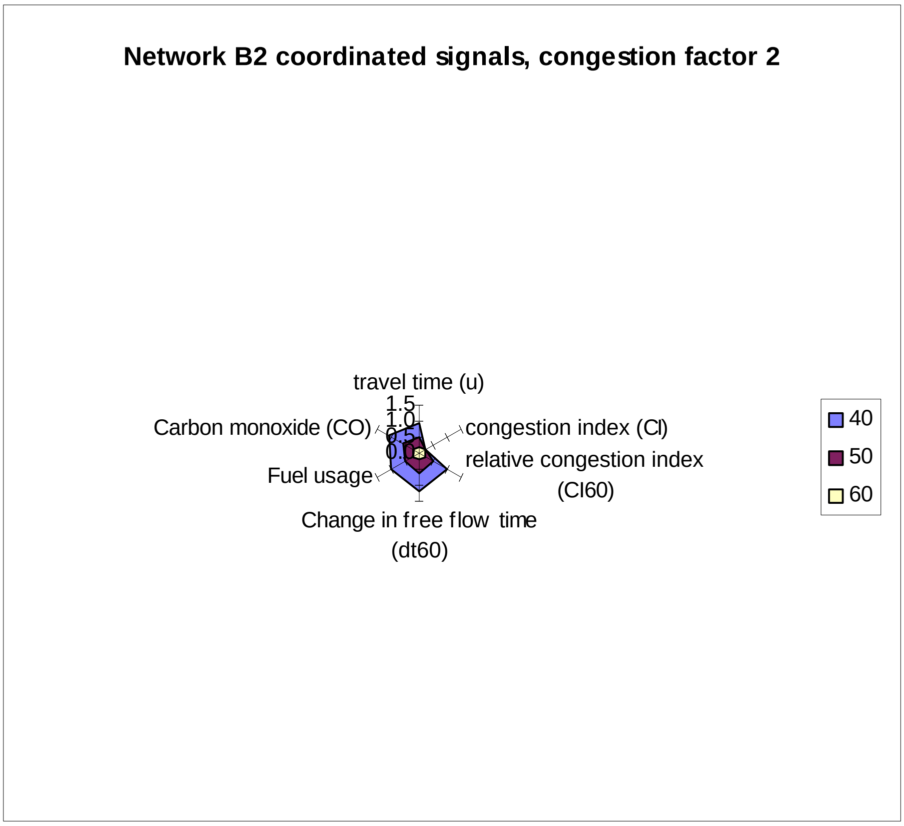


Figure 13.40: Star plots for network B2 (congestion factor 2)



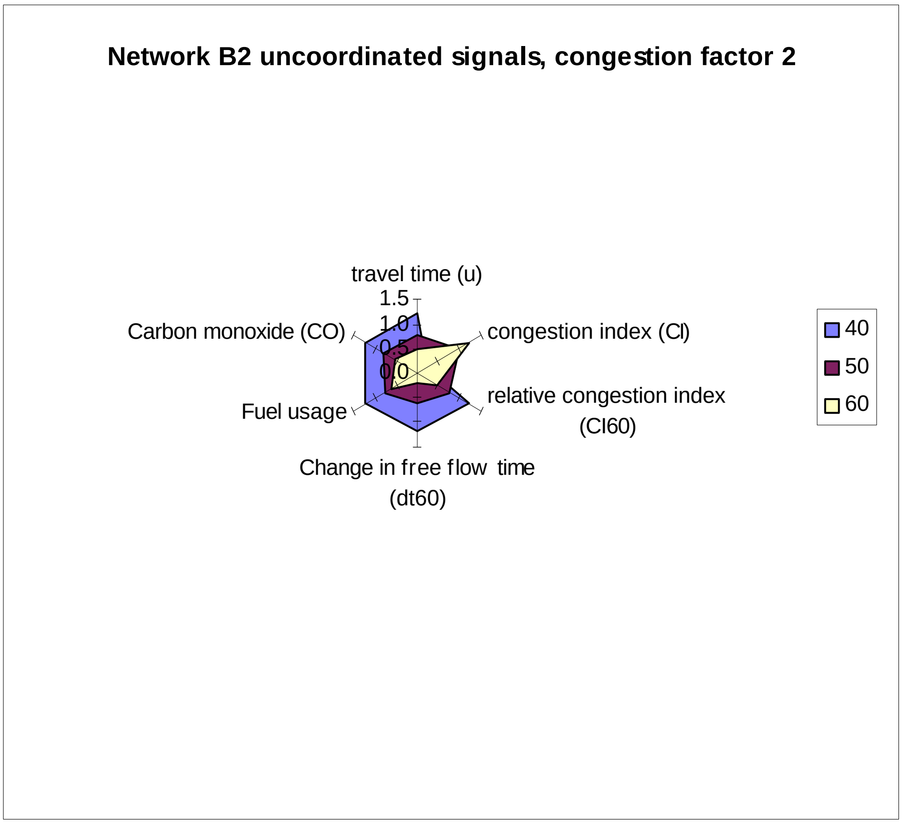
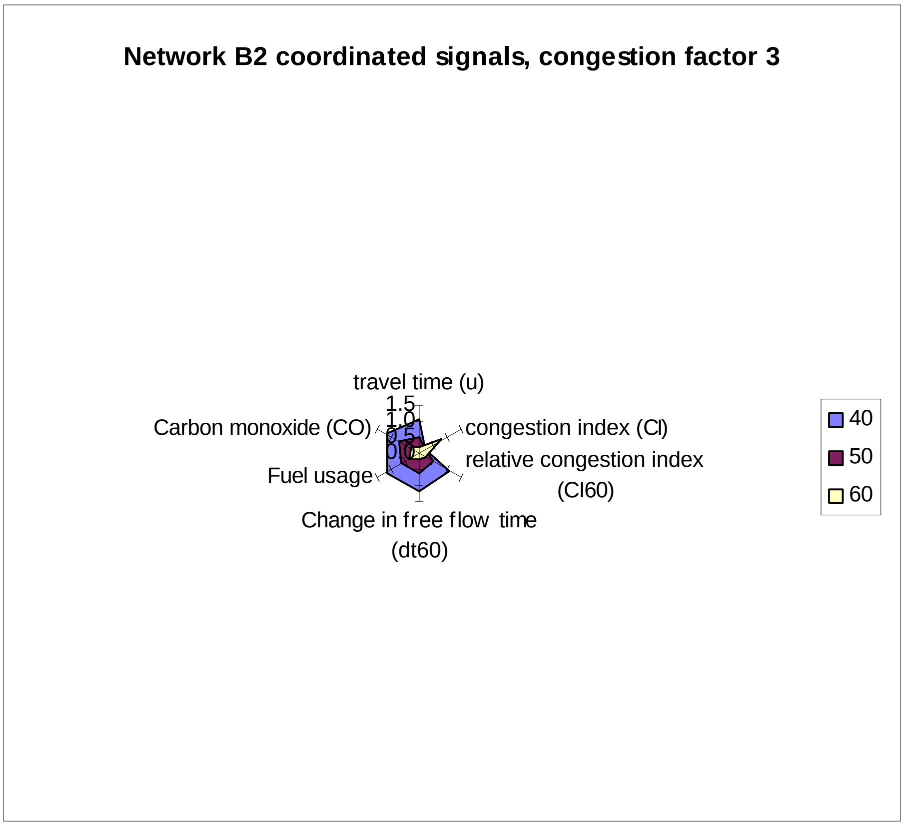


Figure 13.41: Star plots for network B2 (congestion factor 3)



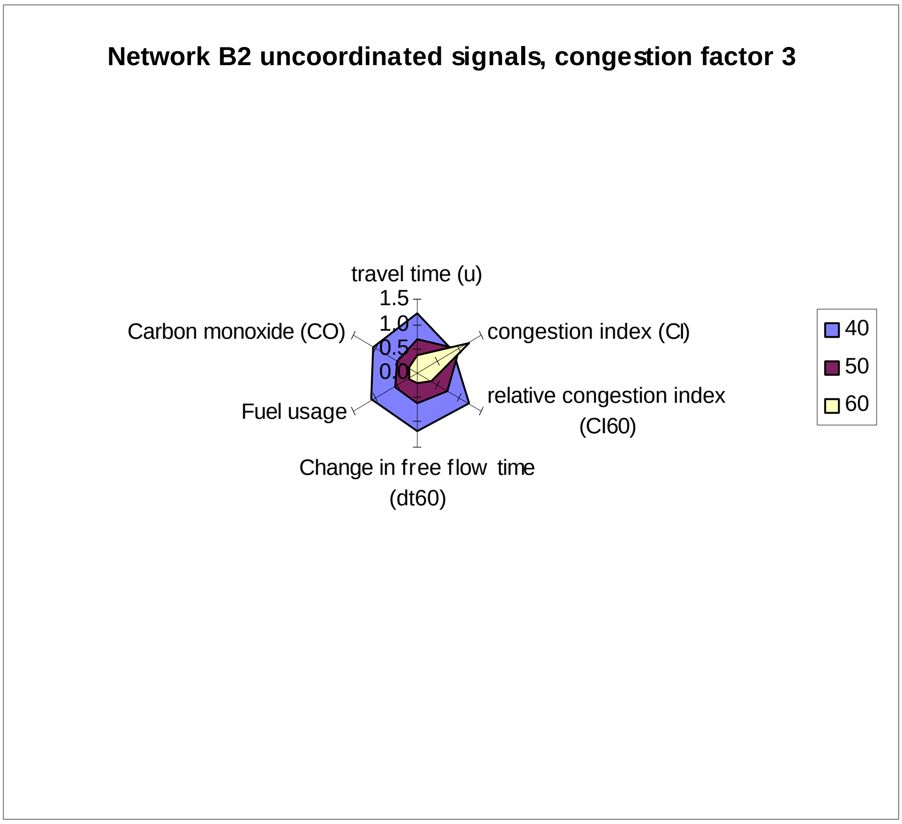
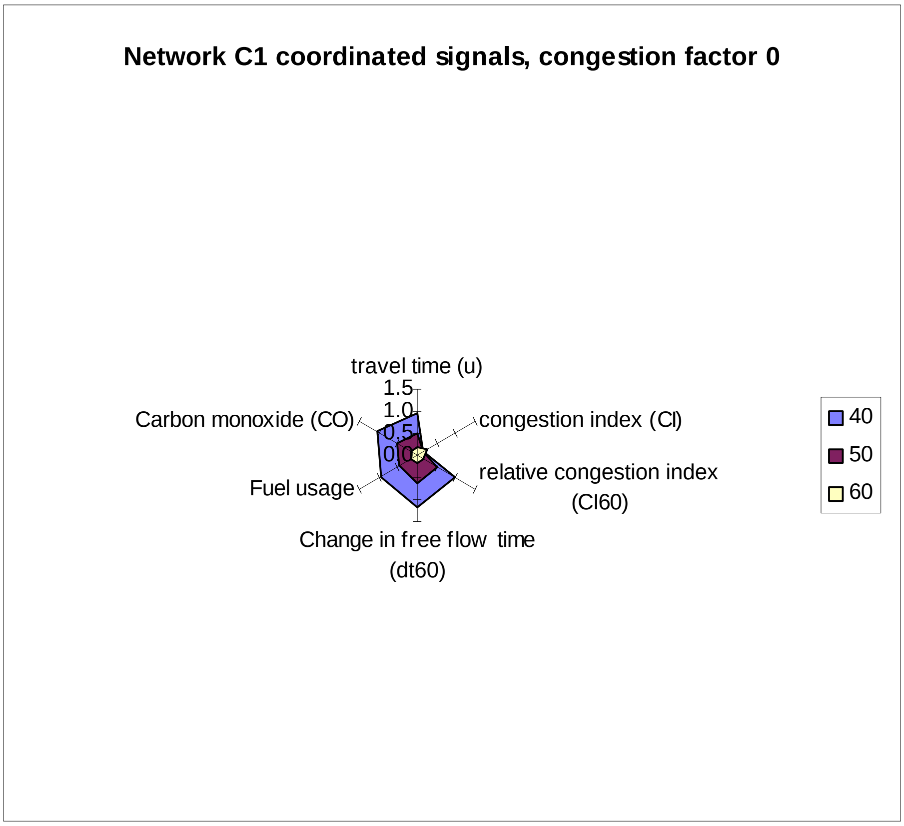


Figure 13.42: Star plots for network C1 (congestion factor 0)



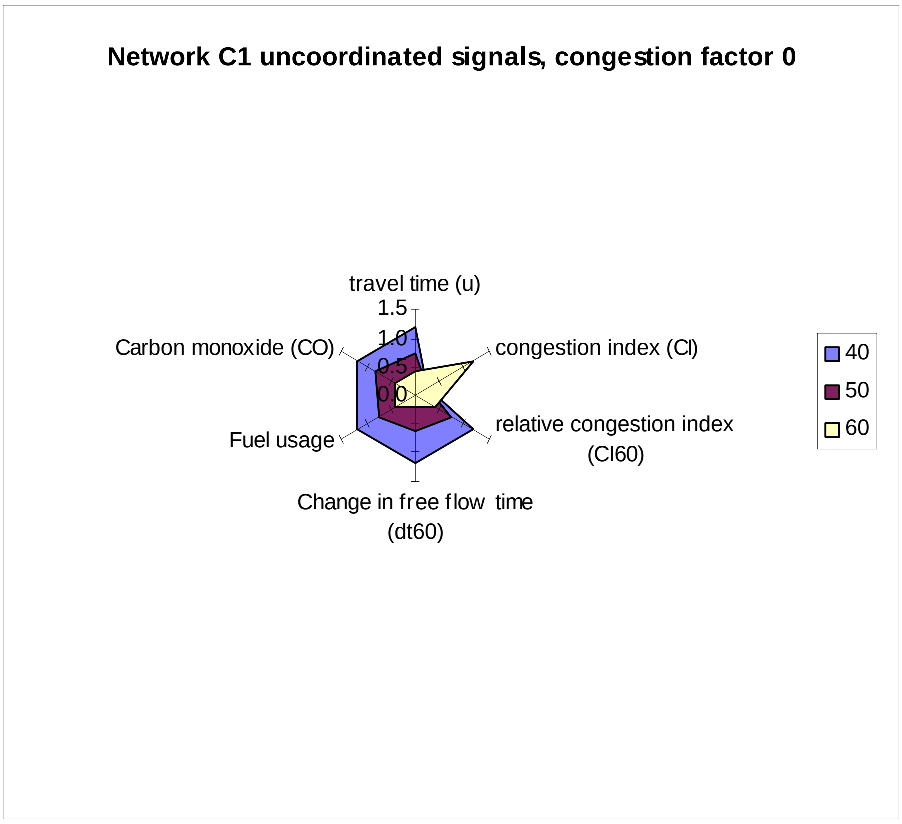
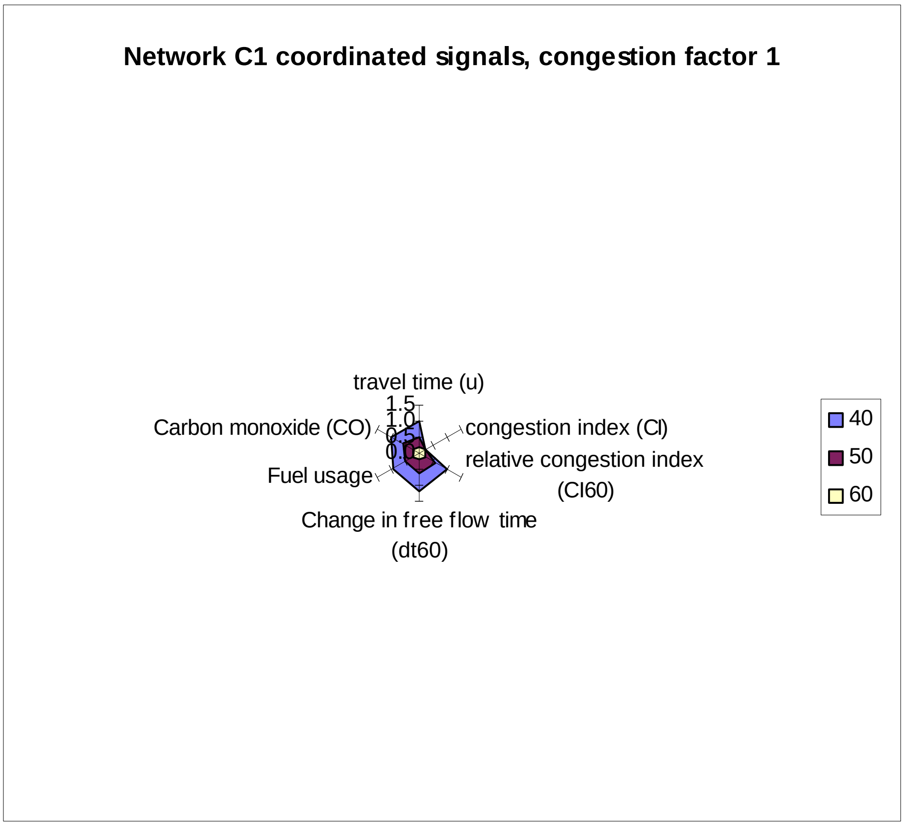


Figure 13.43: Star plots for network C1 (congestion factor 1)



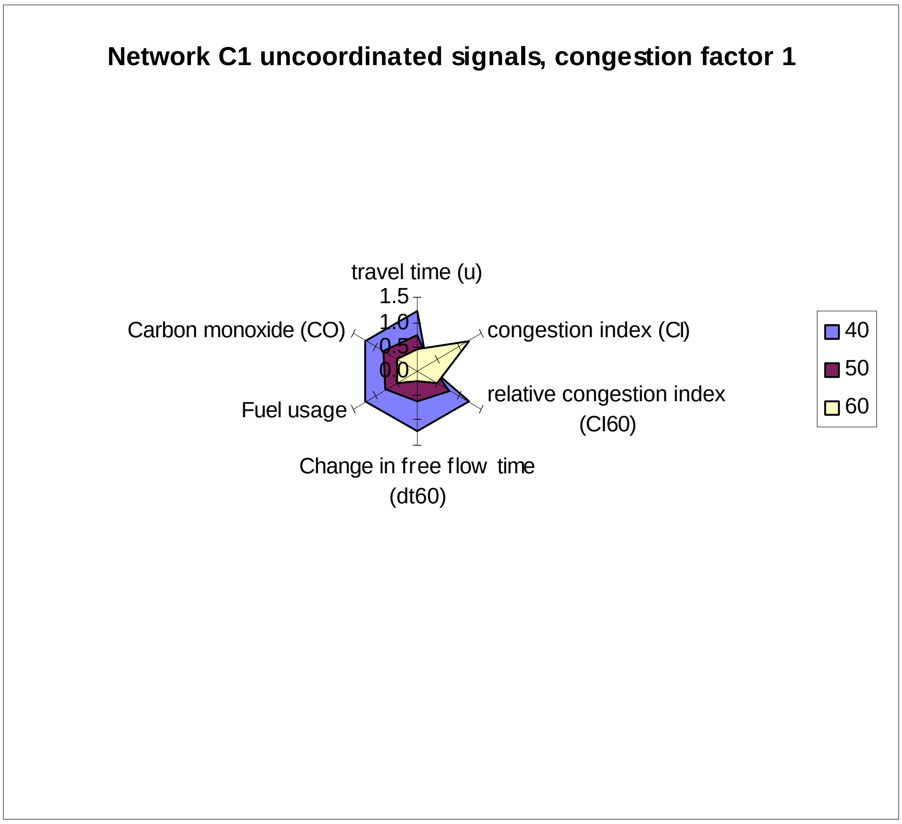
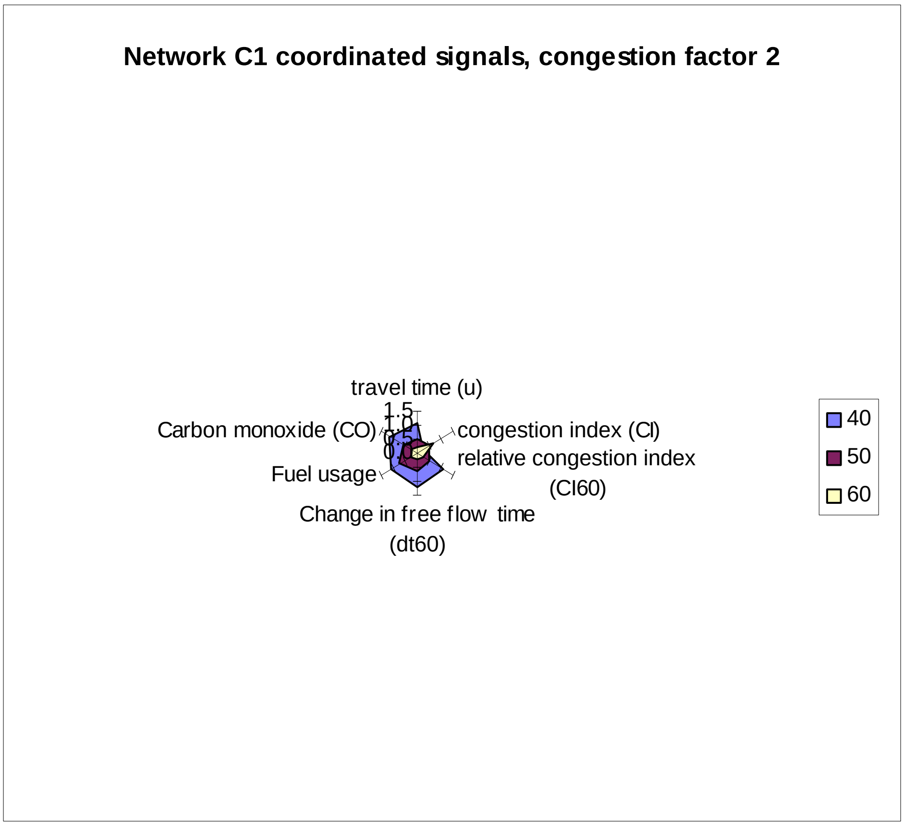


Figure 13.44: Star plots for network C1 (congestion factor 2)



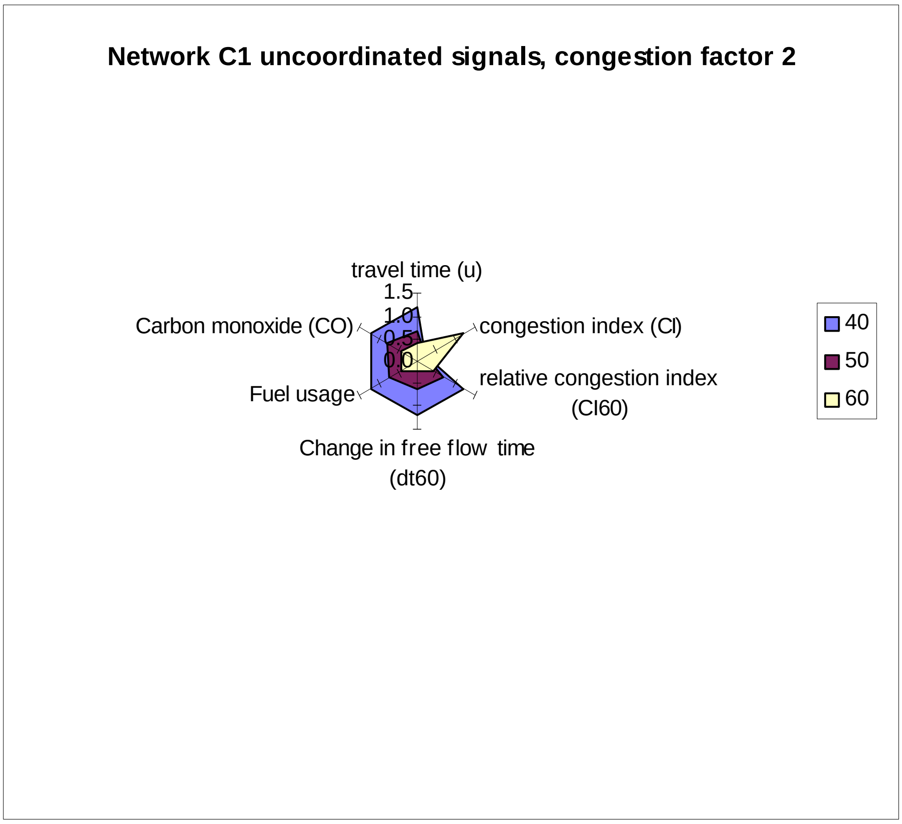
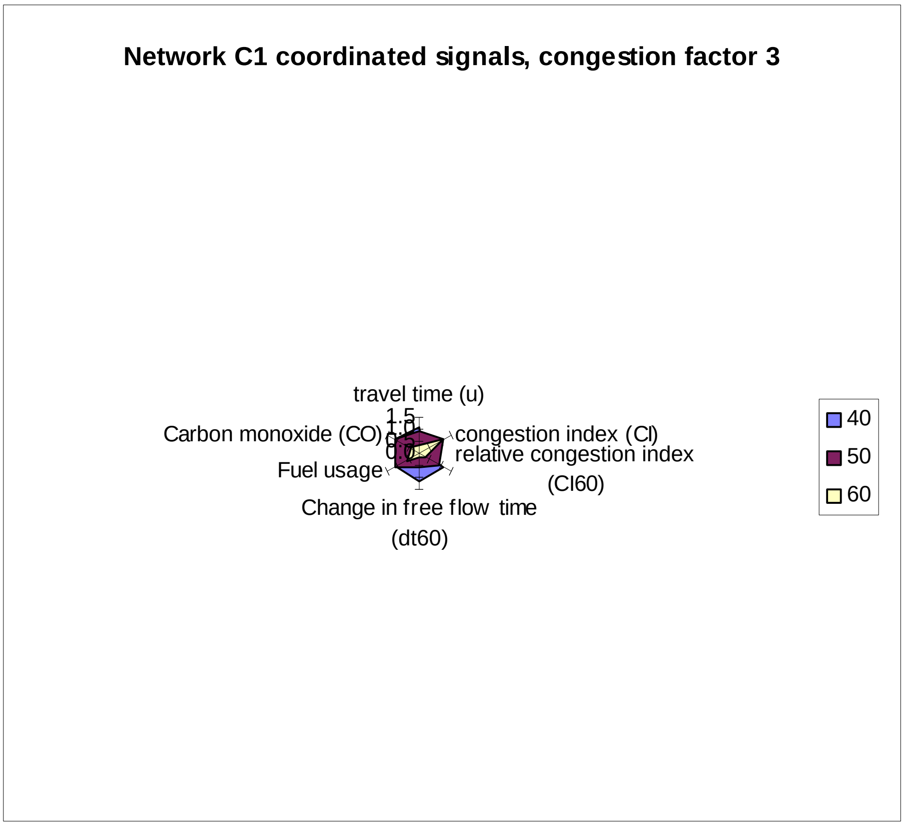


Figure 13.45: Star plots for network C1 (congestion factor 3)



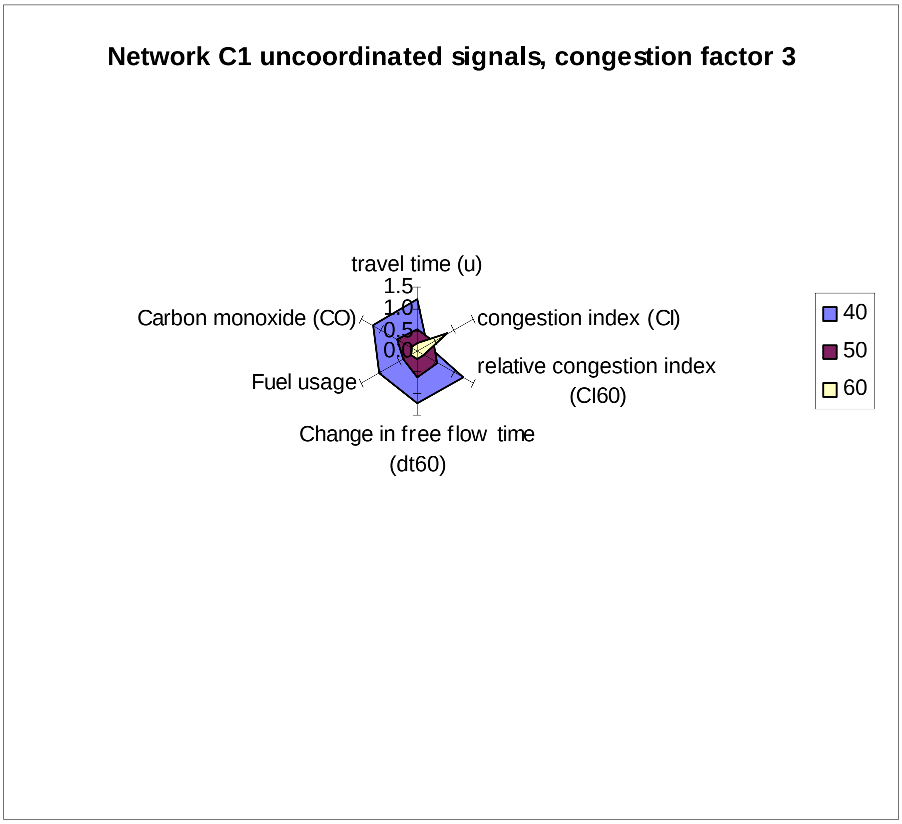
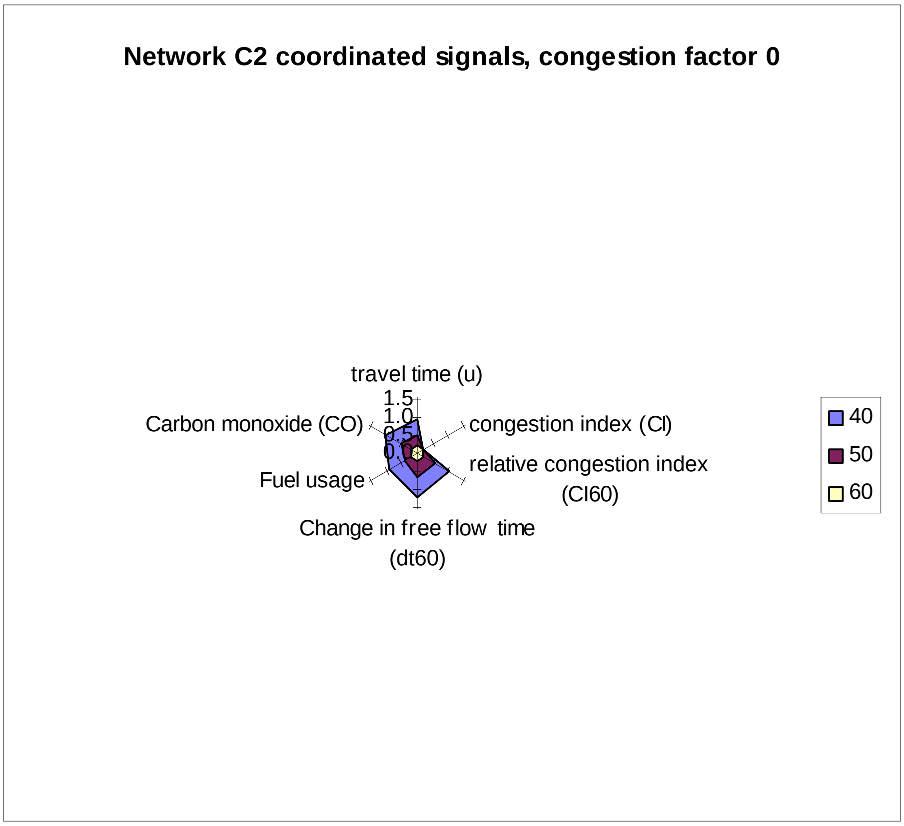


Figure 13.46: Star plots for network C2 (congestion factor 0)



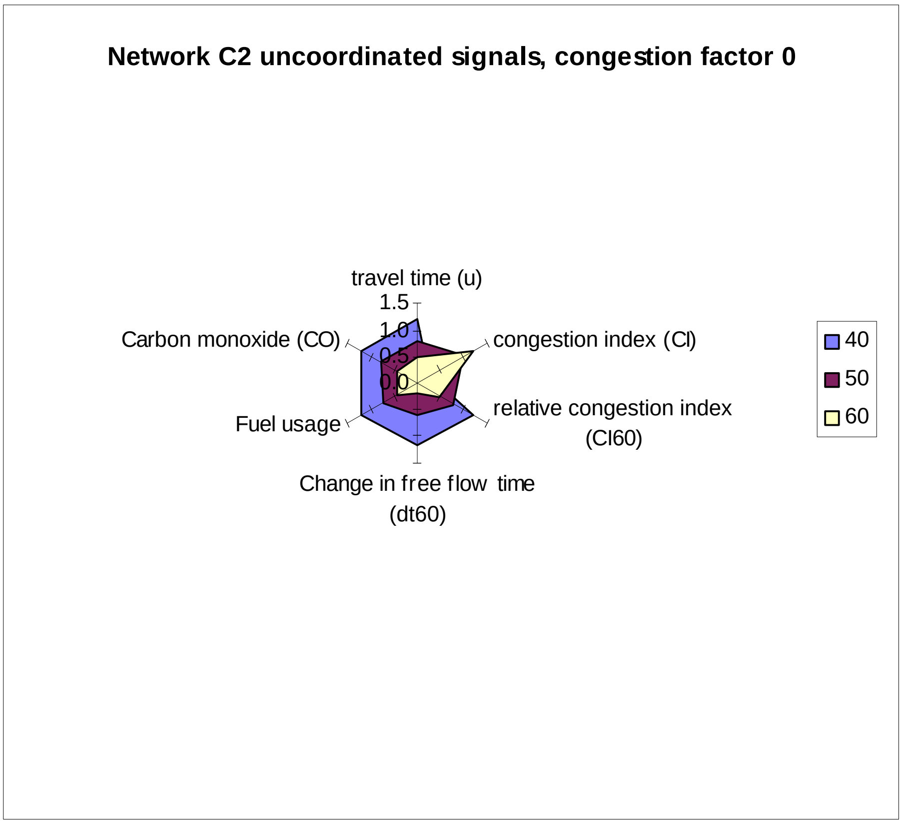
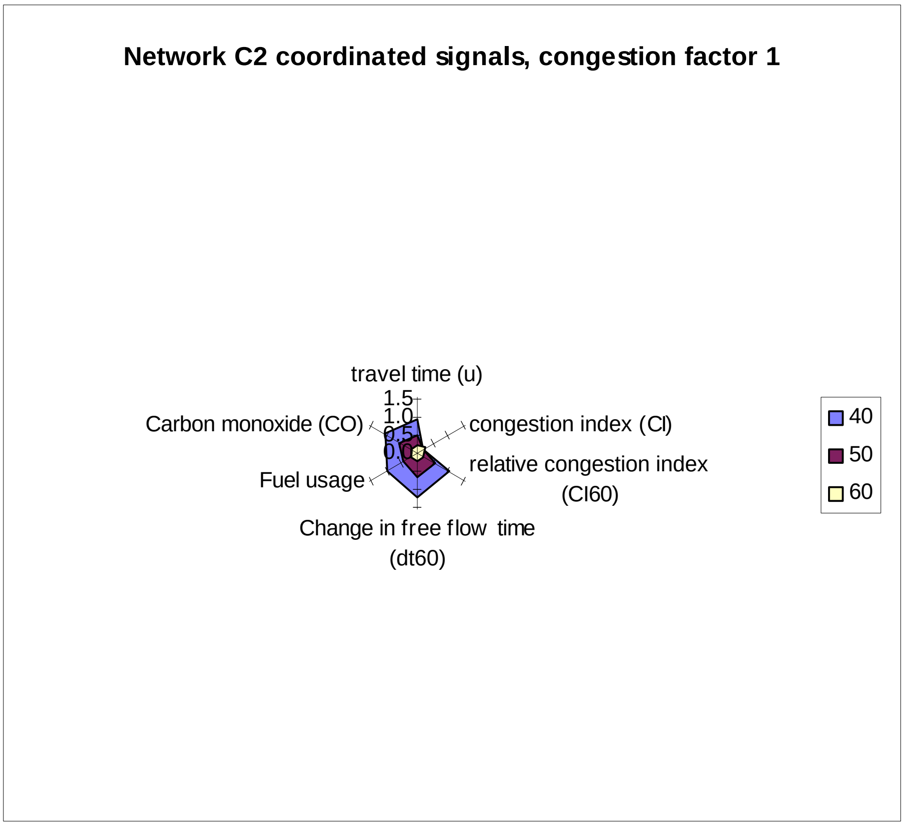


Figure 13.47: Star plots for network C2 (congestion factor 1)



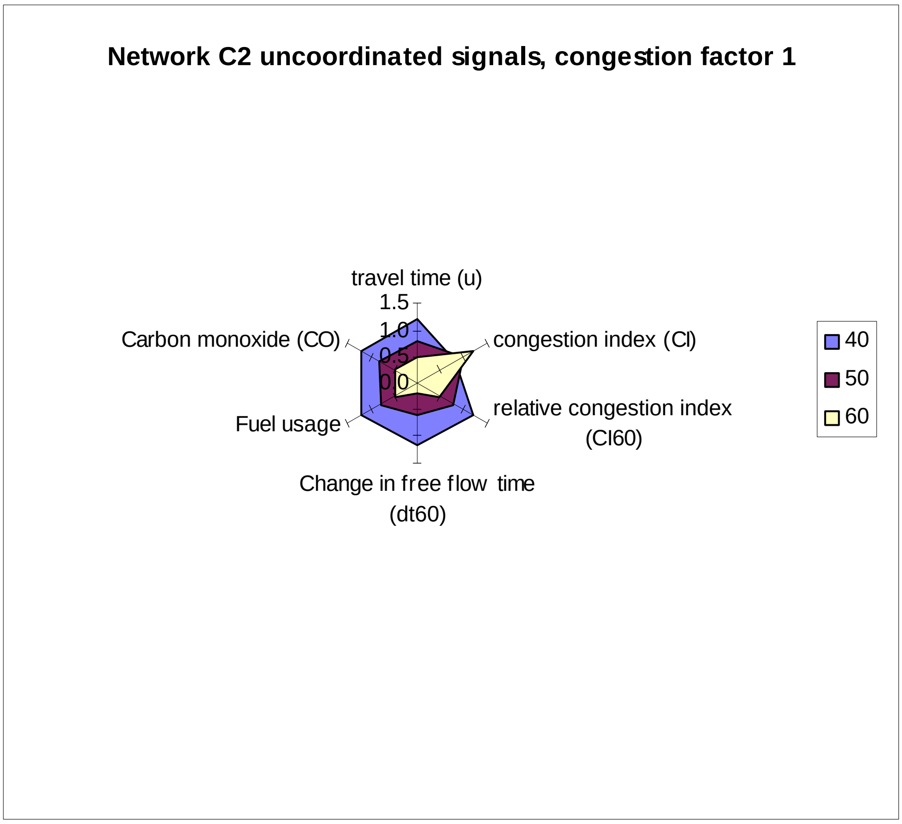
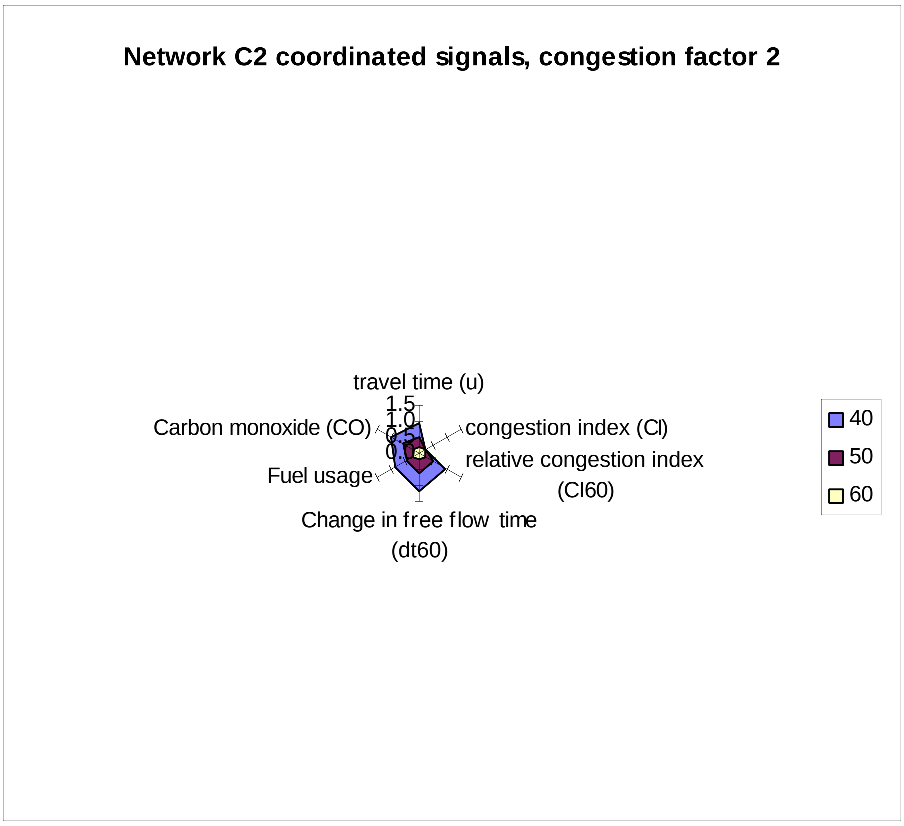


Figure 13.48: Star plots for network C2 (congestion factor 2)



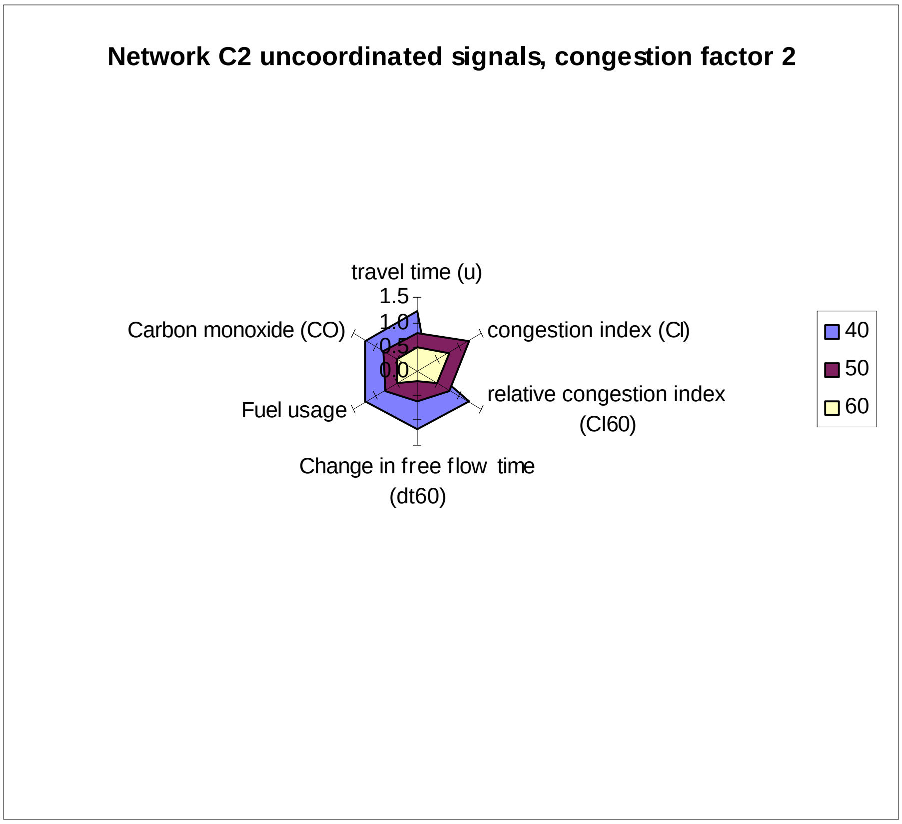
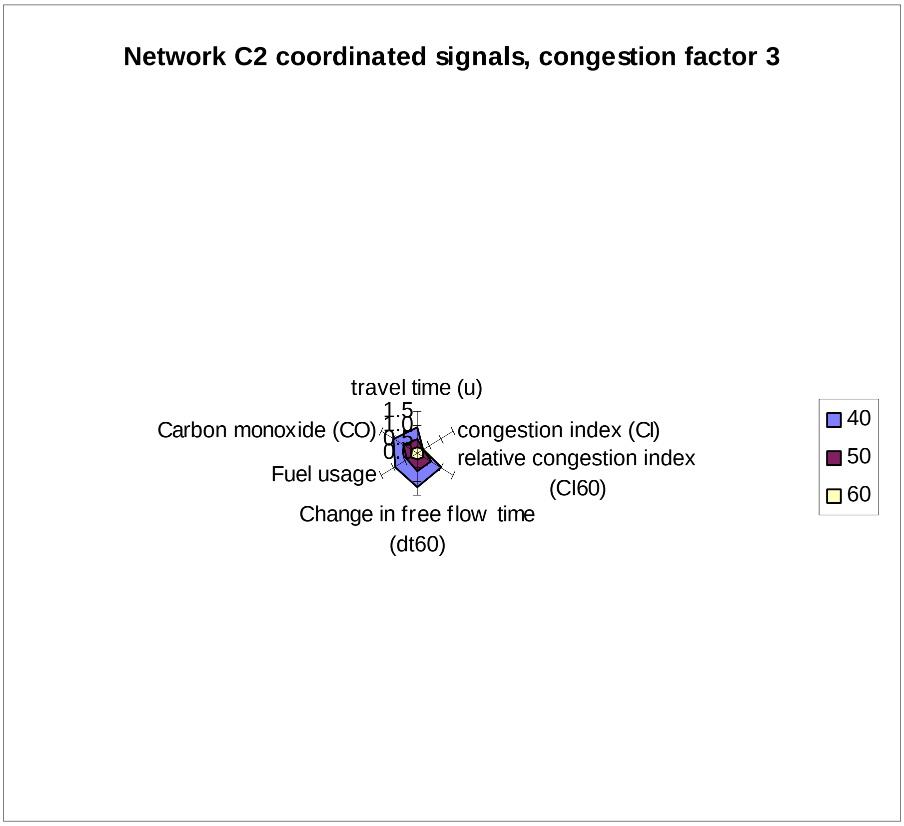
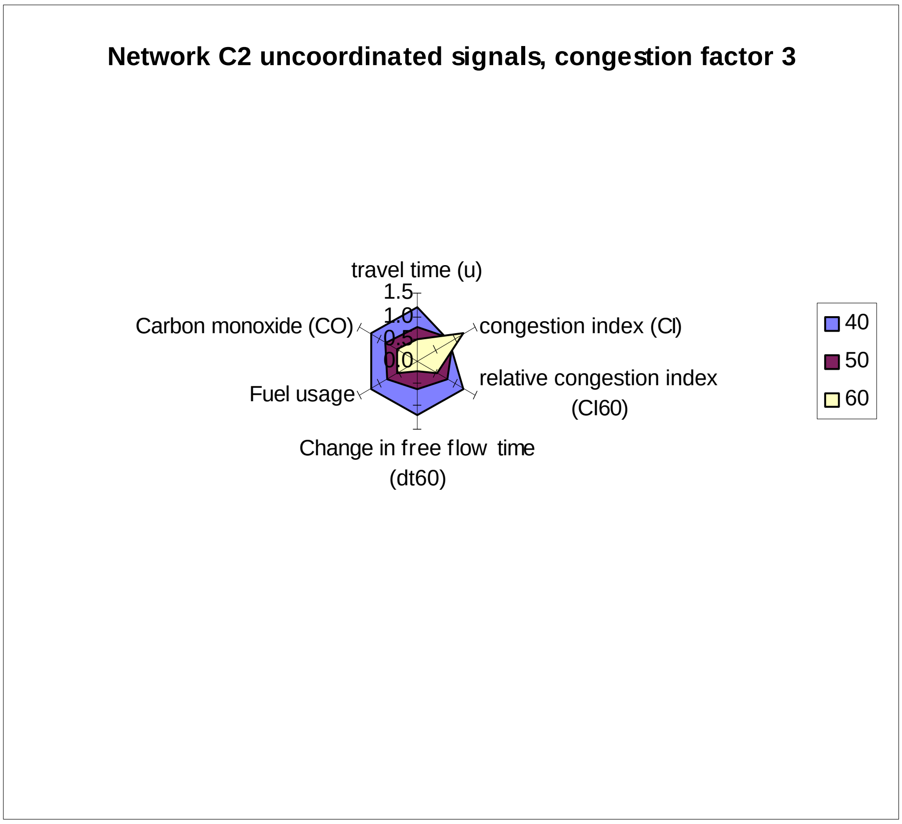


Figure 13.49: Star plots for network C2 (congestion factor 3)





13.3 Summary of results

This chapter has described the results of the modelling study in terms of overall traffic performance of the test networks, and then in terms of link-based performance. Comparisons were drawn between traffic performance under three different speed limits (40, 50 and 60 km/h) for networks of differing traffic design standards, with isolated and coordinated traffic signals control, under different traffic congestion levels.

The overall results from the study were:

- journey speeds in the test networks were considerably less than the set speed limits
- differences in overall travel speeds and journey times were much less than the differences in the speed limits themselves
- signal coordination offered significant advantages for delays and traffic progression, except in some cases at the upper congestion level
- there were indications that coordinated signal operation combined with a 50 km/h speed limit could yield traffic operation conditions at least equal to those for the 60 km/h speed limit with isolated traffic signal control

On the basis of the link-based traffic parameters, the following additional results emerged:

- delays measured relative to the free flow travel time as determined using the specific speed limit were least for the 40 km/h speed limit
- all other link-based measures of traffic efficiency - travel time, changes in free flow travel times, fuel usage and pollutant emissions - tended to suggest that operations under a 60 km/h speed limit with coordinated signal control gave the best results. Operations under a 50 km/h speed limit with signal coordination also gave good results, often comparable with those for the 60 km/h speed limited with isolated signal control.
- the results for fuel usage and emissions under lower speed limits need to be considered with the rider that the empirical version of the ‘running speed’ model used to estimate fuel consumption and carbon monoxide emission rates is based on data collected under a 60 km/h speed limit regime, and thus may not completely reflect the situation when free flow speeds are actually at levels set by the lower speed limits. This is an area for further research.

A number of other areas for further work were identified, and these are discussed further in the final chapter.

14. CONCLUSIONS AND RECOMMENDATIONS

On the basis of the modelling studies described in this document and the results obtained from the model runs, the following conclusions may be drawn:

1. computer-based modelling of road traffic networks is a powerful tool for investigating the likely impacts of different traffic management and control regimes, such as alterative speed limits, that may be difficult to test in the real world. The effects of different levels on traffic congestion may also be examined, which is again and advantage as the ability to observe a range of congestion conditions in a given real network may be limited The limitation on modelling studies is the need to ensure that they conform to circumstances that will be encountered in real networks, and
2. modelling studies also allow the means to relate measured traffic performance in one network under a given set of operating conditions to those in another network under different conditions.

An analysis of overall network performance from the modelling studies suggested that:

3. journey speeds in the test networks were considerably less than the set speed limits;
4. differences in overall travel speeds and journey times were much less than the differences in the speed limits themselves
5. signal coordination offered significant advantages for delays and traffic progression, except in some cases at the upper congestion level
6. coordinated signal operation combined with a 50 km/h speed limit could, under some conditions, yield traffic operation conditions at least equal to those for the 60 km/h speed limit with isolated traffic signal control

Considerations of traffic performance at the link level in the test networks further suggested that:

7. delays measured relative to the free flow travel time as determined using the specific speed limit were least for the 40 km/h speed limit;
8. other link-based measures of traffic efficiency - travel time, changes in free flow travel times, fuel usage and pollutant emissions - tended to suggest that operations under a 60 km/h speed limit with coordinated signal control gave the best results. Operations under a 50 km/h speed limit with signal coordination also gave good results, often comparable with those for 60 km/h and isolated signal control, and
9. the modelled results for fuel usage and emissions under lower speed limits need to be considered with the rider that the empirical version of the ‘running speed’ model used to estimate fuel consumption and carbon monoxide emission rates is based on data collected under a 60 km/h speed limit regime, and thus may not completely reflect the situation when free flow speeds are actually at levels set by the lower speed limits. This is an area for further research.

Further conclusions to be drawn from the study include:

10. strategies to increase public support for lower speed limits could, as well as indicating the road safety advantages of lower limits (in terms of stopping distances and severity of pedestrian accidents, etc) include discussion of overall travel time differences being much less than those implied by differences in speed limits and of the improves progression or smoothness of flow implied for the lower limits, for which system delays (i.e. delays relative to free flow travel times) are significantly reduced for the lower speed limits, and

11. modelling of traffic networks should be extended to larger networks, including real world networks of arterial roads and local streets (possibly with LATM schemes) and other traffic calming measures, for which actual origin-destination patterns and intensities are known. Traffic models should be applied to these areas to test the effects of different speed limits, traffic signal control strategies and traffic calming strategies.

On the basis of these conclusions, a number of recommendations can be made:

1. Computer-based models be considered as important tools for the evaluation of traffic network performance and the investigation of traffic management and control strategies, including lower speed limits and traffic signal coordination.

2. efforts to improve traffic signal coordination strategies, for instance to make them more 'intelligent' and responsive to if not anticipative of changes in traffic demand, should be intensified, as improved signal coordination may remove any losses in traffic efficiency (e.g. in door-to-door travel times, fuel consumption and pollutant emissions) that may accompany the introduction of reduced speed limits.

3. Further research and development is needed to examine the likely fuel consumption and emissions performance of traffic streams operating under lower speed limits.

4. Strategies for increasing public acceptance of and support for lower speed limits could use a combination of the safety benefits, an explanation of the actual differences in door-to-door travel times under different speed limits, and the likelihood of smoother, less-stressful driving possible due to reduced delays (measured as a proportion of stopped time for a journey).

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Appendix A Link-based traffic performance parameters for the synthtic networks

Net	CF	Sig ctl	V	u	CI	CI60	dt0(60)	fuel	CO
A1	0.0	coord	40	1.906	0.086	0.554	0.340	17.38	2.575
			50	1.579	0.094	0.287	0.140	16.16	2.440
			60	1.363	0.113	0.113	0.000	15.38	2.329
A1	0.0	isol	40	1.974	0.110	0.594	0.340	17.80	2.605
			50	1.648	0.126	0.327	0.140	16.58	2.477
			60	1.388	0.104	0.104	0.000	15.71	2.364
A1	1.0	coord	40	1.906	0.087	0.555	0.340	34.78	5.154
			50	1.581	0.096	0.289	0.140	32.34	4.882
			60	1.365	0.116	0.116	0.000	30.78	4.662
A1	1.0	isol	40	1.980	0.114	0.600	0.340	35.71	5.221
			50	1.650	0.129	0.331	0.140	33.25	4.965
			60	1.453	0.167	0.167	0.000	31.69	4.765
A1	2.0	coord	40	1.911	0.090	0.558	0.340	52.20	7.736
			50	1.582	0.098	0.291	0.140	48.55	7.330
			60	1.367	0.119	0.119	0.000	46.21	7.000
A1	2.0	isol	40	1.987	0.117	0.604	0.340	53.72	7.845
			50	1.656	0.132	0.335	0.140	50.05	7.467
			60	1.462	0.173	0.173	0.000	47.70	7.168
A1	3.0	coord	40	1.954	0.115	0.586	0.340	70.19	10.390
			50	1.625	0.126	0.322	0.140	65.30	9.863
			60	1.416	0.154	0.154	0.000	62.01	9.404
A1	3.0	isol	40	2.051	0.151	0.641	0.340	72.28	10.526
			50	1.721	0.170	0.375	0.140	67.36	10.029
			60	1.518	0.214	0.214	0.000	64.23	9.641
A2	0.0	coord	40	1.938	0.076	0.526	0.339	17.42	2.578
			50	1.628	0.093	0.279	0.140	16.24	2.449
			60	1.446	0.141	0.141	0.000	15.71	2.384
A2	0.0	isol	40	2.076	0.127	0.602	0.339	18.07	2.620
			50	1.765	0.155	0.354	0.140	16.89	2.501
			60	1.554	0.187	0.187	0.000	16.10	2.403
A2	1.0	coord	40	1.936	0.075	0.520	0.340	34.86	5.160
			50	1.627	0.092	0.278	0.140	32.49	4.902
			60	1.433	0.130	0.130	0.000	31.24	4.745
A2	1.0	isol	40	2.078	0.127	0.603	0.340	36.21	5.249
			50	1.768	0.156	0.356	0.140	33.83	5.010
			60	1.556	0.188	0.188	0.000	32.26	4.815
A2	2.0	coord	40	1.941	0.008	0.528	0.340	52.33	7.746
			50	1.628	0.093	0.279	0.140	48.78	7.358
			60	1.423	0.119	0.119	0.000	46.63	7.075
A2	2.0	isol	40	2.082	0.129	0.605	0.339	54.40	7.880
			50	1.771	0.158	0.358	0.140	50.82	7.523
			60	1.559	0.190	0.190	0.000	48.47	7.231
A2	3.0	coord	40	1.988	0.105	0.560	0.339	70.59	10.431
			50	1.673	0.122	0.312	0.140	65.82	9.932
			60	1.468	0.152	0.152	0.000	62.63	9.510
A2	3.0	isol	40	2.125	0.151	0.629	0.339	72.99	10.560
			50	1.875	0.179	0.380	0.140	68.21	10.088
			60	1.605	0.222	0.222	0.000	65.07	9.706
B1	0.0	coord	40	1.739	0.002	0.398	0.308	14.84	2.255
			50	1.475	0.002	0.165	0.128	13.92	2.129

60	1.291	0.000	0.000	0.000	13.34	2.022

Net	CF	Sig ctl	V	u	CI	CI60	dt0(60)	fuel	CO
B1	0.0	isol	40	1.869	0.007	0.506	0.308	15.29	2.303
			50	1.607	0.093	0.275	0.128	14.39	2.190
			60	1.427	0.117	0.117	0.000	13.79	2.096
B1	1.0	coord	40	1.737	0.000	0.396	0.308	29.82	4.533
			50	1.473	0.000	0.163	0.128	27.97	4.279
			60	1.289	0.000	0.000	0.000	26.81	4.063
B1	1.0	isol	40	1.875	0.008	0.512	0.308	30.80	4.682
			50	1.613	0.010	0.282	0.128	29.01	4.414
			60	1.482	0.163	0.163	0.000	28.20	4.282
B1	2.0	coord	40	1.761	0.017	0.419	0.308	45.41	6.892
			50	1.488	0.001	0.178	0.128	42.53	6.507
			60	1.307	0.020	0.020	0.000	40.83	6.197
B1	2.0	isol	40	1.891	0.009	0.526	0.308	46.66	7.001
			50	1.462	0.010	0.150	0.128	42.35	6.431
			60	1.505	0.184	0.184	0.000	42.86	6.508
B1	3.0	coord	40	1.957	0.146	0.605	0.308	66.45	9.844
			50	1.699	0.186	0.385	0.128	63.09	9.420
			60	1.532	0.244	0.244	0.000	61.56	9.199
B1	3.0	isol	40	1.940	0.119	0.569	0.308	62.97	9.417
			50	1.672	0.145	0.337	0.128	59.42	9.004
			60	1.501	0.189	0.189	0.000	57.12	8.663
B2	0.0	coord	40	1.758	0.000	0.401	0.315	14.95	2.266
			50	1.504	0.011	0.178	0.135	14.07	2.147
			60	1.327	0.002	0.002	0.001	13.51	2.046
B2	0.0	isol	40	1.901	0.008	0.517	0.308	15.40	2.311
			50	1.641	0.107	0.290	0.128	14.51	2.201
			60	1.463	0.134	0.134	0.000	13.92	2.108
B2	1.0	coord	40	1.753	0.000	0.397	0.315	29.99	4.548
			50	1.494	0.003	0.169	0.135	28.17	4.300
			60	1.315	0.001	0.012	0.001	17.06	4.094
B2	1.0	isol	40	1.906	0.087	0.523	0.308	31.00	4.649
			50	1.647	0.112	0.296	0.128	29.23	4.434
			60	1.473	0.145	0.145	0.000	28.07	4.252
B2	2.0	coord	40	1.773	0.013	0.414	0.315	45.51	6.896
			50	1.501	0.001	0.176	0.135	42.66	6.511
			60	1.314	0.001	0.001	0.001	40.85	6.183
B2	2.0	isol	40	1.920	0.010	0.536	0.308	46.87	7.025
			50	1.660	0.123	0.309	0.128	44.21	6.707
			60	1.485	0.157	0.157	0.000	45.24	6.448
B2	3.0	coord	40	1.872	0.008	0.505	0.315	63.87	9.596
			50	1.583	0.007	0.254	0.135	60.02	9.098
			60	1.425	0.116	0.120	0.001	58.01	8.735
B2	3.0	isol	40	1.955	0.118	0.564	0.308	62.91	9.404
			50	1.685	0.142	0.332	0.128	59.31	8.892
			60	1.509	0.181	0.181	0.000	57.00	8.636
C1	0.0	coord	40	1.766	0.000	0.352	0.290	15.20	2.300
			50	1.533	0.000	0.147	0.121	14.22	2.172
			60	1.374	0.010	0.010	0.000	13.67	2.075
C1	0.0	isol	40	1.883	0.006	0.443	0.290	15.71	2.348
			50	1.649	0.008	0.240	0.121	14.77	2.237
			60	1.487	0.102	0.102	0.000	14.11	2.139

Net	CF	Sig ctl	V	u	CI	CI60	dt0(60)	fuel	CO
C1	1.0	coord	40	1.780	0.001	0.361	0.290	30.66	4.635
			50	1.544	0.002	0.158	0.121	28.72	4.383
			60	1.381	0.002	0.002	0.000	27.57	4.185
C1	1.0	isol	40	1.890	0.007	0.447	0.290	31.68	4.727
			50	1.659	0.009	0.247	0.121	29.83	4.511
			60	1.504	0.116	0.116	0.000	28.59	4.328
C1	2.0	coord	40	1.847	0.004	0.410	0.290	47.56	7.127
			50	1.598	0.050	0.198	0.121	45.56	6.757
			60	1.450	0.073	0.073	0.000	42.94	6.488
C1	2.0	isol	40	1.910	0.008	0.463	0.290	48.19	7.175
			50	1.672	0.010	0.260	0.121	45.48	6.866
			60	1.544	0.153	0.153	0.000	43.96	6.630
C1	3.0	coord	40	1.946	0.104	0.495	0.290	67.08	9.836
			50	1.879	0.250	0.441	0.121	67.90	9.793
			60	1.643	0.244	0.244	0.000	63.80	9.266
C1	3.0	isol	40	1.990	0.117	0.517	0.290	66.81	9.815
			50	1.728	0.134	0.305	0.121	62.30	9.340
			60	1.597	0.189	0.189	0.000	60.94	9.134
C2	0.0	coord	40	1.790	0.000	0.353	0.298	15.26	2.306
			50	1.555	0.002	0.152	0.130	14.31	2.182
			60	1.386	0.002	0.003	0.001	13.93	2.110
C2	0.0	isol	40	1.941	0.008	0.466	0.290	15.88	2.360
			50	1.709	0.104	0.268	0.122	14.97	2.255
			60	1.547	0.131	0.131	0.000	14.33	2.161
C2	1.0	coord	40	1.791	0.000	0.354	0.298	30.68	4.638
			50	1.556	0.003	0.153	0.130	28.76	4.388
			60	1.401	0.012	0.002	0.001	27.76	4.211
C2	1.0	isol	40	1.942	0.082	0.466	0.290	31.90	4.741
			50	1.709	0.105	0.268	0.122	30.11	4.537
			60	1.549	0.132	0.132	0.000	28.89	4.356
C2	2.0	coord	40	1.801	0.001	0.359	0.298	46.47	7.024
			50	1.560	0.005	0.155	0.130	43.55	6.644
			60	1.400	0.001	0.018	0.001	41.86	6.349
C2	2.0	isol	40	1.949	0.009	0.470	0.290	48.53	7.174
			50	1.709	0.268	0.268	0.122	45.55	6.863
			60	1.556	0.137	0.137	0.000	43.68	6.586
C2	3.0	coord	40	1.831	0.003	0.379	0.298	61.78	9.336
			50	1.576	0.016	0.166	0.130	57.88	8.832
			60	1.425	0.009	0.004	0.001	55.61	8.437
C2	3.0	isol	40	1.977	0.103	0.487	0.290	64.33	9.553
			50	1.723	0.114	0.278	0.122	60.56	9.122
			60	1.580	0.157	0.157	0.000	58.18	8.776