ISSUES WITH THE OPERATIONAL ANALYSIS OF URBAN INTERSECTIONS

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ABSTRACT

Operational analyses of urban intersections are often undertaken during traffic impact studies to evaluate the impact of a development on traffic flow. Many improvements to the urban street network are often warranted on the basis of such analyses.

Various studies have, however, been undertaken in South Africa and Australia that show that the operational analysis of an intersection is a complex exercise which often produces invalid results. The studies have shown that a large number of factors need to be taken into account when modelling urban intersections, and that the models should be properly calibrated and validated. Unless these issues are properly addressed, the operational analysis of intersections serves little or no purpose.

In this paper, it is proposed that simpler approaches should be utilised for the purposes of traffic impact studies. One relative simple approach that can be considered is to evaluate intersections simply in terms of volume/capacity ratios. Improvements to the street network can then be warranted on the basis of such ratios. Maximum ratios for such purpose are proposed in the paper.

1. INTRODUCTION

The evaluation of the Level of Service (LOS) provided by a road traffic facility, is fundamental to the field of traffic engineering. The Level of Service is used to define a qualitative measure describing operational conditions of road traffic (TRB, 1985, 1994, 1997, 2000). This is of particular importance in urban street networks that are often subject to congestion resulting in excessive delays to traffic.

The capacity of an urban street network is mostly restricted by the intersections in the network. This is due to the need for separating conflicting traffic streams at such intersections. The evaluation of the Level of Service on an urban road network is therefore normally reduced to the analysis of traffic operations at intersections. Various traffic control types are used at intersections, of which the traffic signal is one of the most important. Traffic signals are often used at the more critical and heavily trafficked intersections. The objective of this paper is thus to critically evaluate the existing methods applied in the operational analysis of signalised intersections in an urban environment. Reference will be made to research conduced through the years in this specific field, and some important issues related to the operational analysis of signalised intersections will be highlighted.

2. DELAY AS A MEASURE OF EFFECTIVENESS

It is standard practice in the field of traffic engineering to use average delay as the Measure of Effectiveness (MOE) in the establishment of the Level of Service (LOS) of a signalised intersection. According to the Highway Capacity Manual (TRB, 1994, 1997,2000), delay is a measure of driver discomfort, frustration, fuel consumption and lost travel time.

Serious questions can, however, be asked regarding the suitability of delay in establishing Levels of Service.

The following are a number of criticisms that can be directed at the use of delay as a measure of effectiveness:

- The value of delay is probably not uniform and drivers would probably give different weights to different portions of delay experienced at an intersection. For example, it is likely that the time spent stopping is given more weight than the time actually stopped. This is particularly important in a co-ordinated traffic signal network where the reduction of stop-go cycles are probably more important than reducing stopped delay.
- Average delay is used in the establishment of the Level of Service and variation of delays is not taken into account. A large variation in delay can be as unacceptable as a large average delay. An example of an attempt to model the variance of delay, based on random arrival conditions, was undertaken by Fu and Hellinga (2000).
- The intersection may be operating under oversaturated conditions (demand/capacity ratio greater than 1.0) but the average delay could fall in an acceptable range, especially when a short analysis period of 15 minutes (for example) is considered. The 1997 Highway Capacity Manual explicitly states that "…nor does a Level of Service better than E automatically imply that unused capacity is available". It is thus possible to report that the level of service is acceptable while the intersection is in fact oversaturated.

The above discussion shows that the relationship between delay and Level of Service is complex. This, together with the difficulty in developing accurate models for delay, have resulted in the questioning of the advisability of using delay as a measure of effectiveness.

3. THE DEVELOPMENT OF DELAY MODELS

The first major studies of delays at signalised intersections were undertaken by Webster (1958) and Webster and Cobbe (1966). These studies provided the foundation for recent work in this field.

A number of refinements to the Webster models have been developed through the years, but all these models suffered from two major limitations:

- The models were only applicable to steady state, undersaturated conditions in which demand is less than the capacity at the intersections and queues are in equilibrium. Oversaturation occurs regularly in urban road networks worldwide, resulting in queues forming that are not in equilibrium.
- The models were also developed for isolated intersections at which traffic was assumed to arrive randomly, and not for situations with low or high levels of fluctuations in arrival patterns. It was also not possible to model the platooned arrivals observed in signalised networks.

The next major development was the TRANSYT model developed by Robertson in 1969. In this model, the network performance was evaluated by macroscopic simulation of traffic flow on the links of the street network. Flow patterns were traced from intersection to intersection and the effect there-of on queue length evaluated. In terms of the overflow delay component, the TRANSYT model, however, still assumed that traffic arrive randomly at an intersection. It is interesting to note that an earlier version of TRANSYT (Robertson, 1969) attempted to address the assumption regarding random arrivals, but not the later versions (Vincent, et al, 1980).

A third major milepost in the development of intersection delay models was the work undertaken by Kimber and Hollis in 1979 on time-depending queuing. A co-ordinate transformation technique was applied to obtain an approximate empirical solution to the intractable problem of oversaturated conditions in which traffic demand is higher than the capacity of an intersection. This addressed the first limitation in that it became possible to analyse conditions other than steady-state undersaturated. However, the problem of fluctuating arrivals in urban street networks was, however, still not addressed.

Very little research has been directed to the one major remaining problem - namely the assumption of random arrivals. This assumption may be appropriate at isolated intersections or intersections where traffic volumes are relatively low, such as in residential areas. Capacity analysis is, however, seldom undertaken at such intersections, simply because such intersections do not often pose a problem. The intersections that pose problems and require analysis, are those on the busier streets and arterials on which intersections are seldom isolated. On these streets, traffic is often heavily platooned, and traffic arrivals are not random.

The above problem was identified by Newell in 1990 and Van As in 1991a. Some work was also done by researchers such as Fambro and Rouphail (1997). Further work on the problem was undertaken by Pretorius in 2001 to demonstrate that the use of models currently available could seriously under- or overestimate traffic delays on urban street networks because of assumptions regarding arrival (and other) processes. This could have very serious consequences regarding various aspects such as optimal traffic control, traffic signal settings and traffic management. An alternative approach, based on relatively simple volume/capacity ratios is therefore proposed.

4. DELAY MODELS USED IN PRACTICE

A variety of delay models are available for use in practice. There are basically two types of models, namely models utilising Monte Carlo Simulation techniques (such as SIMTRA developed by Van As in 1991b) and analytical models. In practice, simulation models are seldom used and most models utilise analytical procedures for estimating delays.

The analytical models utilise queuing theory to estimate queue lengths and delay. Due to the complexity of traffic operations at intersections, various simplifying assumptions are required. It is simply not possible to develop a model that is precisely correct, and all analytical models are therefore approximations.

Some examples of analytical models that have been developed for the analyses of traffic signal controlled intersections over the past 40 years are shown in Table 1. Some of the models were modified by Pretorius (2001) to include the important I-factor, which will be defined and discussed later, in an attempt to account for non-random traffic arrivals. Some of these models have been incorporated into commercially available traffic engineering computer programs:

Where:

X = degree of saturation
I = coefficient of variance term
s = saturation flow
G = effective green time
C = cycle length
H(μ) = exponential decreasing function

The above models were evaluated by Pretorius (2001) who concluded that the most serious limitations of these models are due to the following two factors:

- Non-random traffic arrivals - Variation in traffic arrival patterns at intersections
- Time dependency - Unstable conditions during peak periods

These factors are discussed in more detail in the following sections.

Table 1. Steady-state overflow queuing models.

Reference	Average steady-state overflow queue	I-factor in original model
Webster (1958), modified by Hutchinson (1972)	$\frac{I \cdot X^2}{2 \cdot (1-X)}$	No
Miller (1963)	$\frac{2X-1}{2 \cdot (1-X)} I$	Yes
Newell (1965)	$\frac{I \cdot H(\mu) \cdot X}{2 \cdot (1-X)}$	Yes
Miller (1968)	$\frac{I \cdot e^{\frac{-4 \cdot \mu}{3 \cdot X}}}{2 \cdot (1-X)}$	No
Vincent, et al (1980), TRANSYT 8 Highway Capacity Manual (1985-2000) McNeil (1968)	$\frac{I \cdot X}{2 \cdot (1-X)}$	No
Akçelik (1980)	$\frac{1.5 \cdot I \cdot (X - 0.67 - \frac{s \cdot G}{600})}{1-X}$	No
SIDRA 5 Akçelik and Chung (1995)	$0.55 \cdot I \cdot X \frac{(X - X_0)}{1-X} \cdot \frac{G}{C} \cdot (sG)^{0.75}$ $X_0 = 0.40 \cdot (sG)^{0.2}$ $X_0 \leq 0.95$	No
$\mu = (1-X) \cdot \sqrt{\frac{s \cdot G}{I}}$		

5. NON-RANDOM TRAFFIC ARRIVALS

Most of the available delay models assume that traffic arrivals are random and follow the Poisson process. Although the random hypothesis agrees well with observations of free flowing light traffic, it is less effective when traffic becomes congested, or when traffic is metered by an upstream facility with a restricted capacity (Miller, 1963; Hutchinson, 1972; Newell, 1990; Van As, 1991a). The analytical tractability of the random assumption, however, has lead to wide adoption of the hypothesis. Models involving more complex assumptions are difficult to apply in practice, and could require significantly more data.

In urban areas, traffic flow is seldom random while fluctuations in traffic arrivals can vary significantly from intersection to intersection.

This variation is caused by factors such as the following:

- A lack of passing opportunities that results in vehicles forming platoons. The arrivals of platoons could be random, but successive arrivals within a platoon are no longer independent and the random hypothesis would then break down (Miller, 1961).
- Upstream traffic signals also result in vehicles forming platoons. However, not only are successive arrivals within a platoon no longer random, but the platoon arrivals themselves are cyclic and therefore also not random.
- Traffic signals also have a filtering effect on traffic, which reduce large fluctuations in traffic and therefore cause downstream arrivals to become more uniform. Cobbe indicated in 1964 that variations in traffic flow during the peak period would be reduced to the impact of the limiting capacity of upstream roads and junctions. Robertson (1969) found some evidence that the overflow delay at an intersection, which is located downstream of another intersection, is considerably smaller than the overflow delay at an isolated intersection, and therefore applied a reduction factor of 50% to the overflow delay formula in earlier versions of TRANSYT. This was highlighted in the 1990's by Newell (1990) and van As (1991a), amongst others.

Some of the impacts of non-random arrivals were greatly ignored by many delay studies. In discussing the generally accepted assumption of random arrivals, Newell in 1990 stated that: "*it is not clear from the published literature if this postulate evolved because some people believed it might be true or because they were desperate for any formulas which would serve as a proxy*." However, as can be seen in Table 1, the majority of delay models still ignore the effect of non-random arrivals, by the omission of the I-factor from the model.

Pretorius (2001) provided a comparison of the accuracy of the existing delay models, based on the application of an extensive macroscopic simulation model. The results are shown in Table 2, with an indication of the average and standard deviation of modelling errors for degrees of saturation of approximately 85%, 90% and 95%. It is important to note that the evaluation is based on steady state conditions with non-random arrival and non-constant service flow rates. Furthermore, the I-factor was omitted from all the models in Table 2, irrespective of whether the original model included the factor, such as the Miller (1963) and Newell (1965) models.

Table 2. Modelled error in some existing delay models (excl. I-factor).

Model	Average and standard deviation of modelling errors (% of cycle length) for degrees of saturation of :					
	±85%		±90%		±95%	
	Avg	Std Dev	Avg	Std Dev	Avg	Std Dev
Webster (1958)	3.0	8.4	3.2	11.3	0.8	17.9
Miller (1963)	2.7	8.4	3.1	11.2	0.7	18.0
Newell (1965)	-1.8	8.6	-2.1	11.1	-5.2	18.2
Miller (1968)	-1.6	8.6	-2.0	11.1	-5.5	18.1
TRANSYT 8 (1980); HCM	4.6	8.4	4.6	11.9	2.2	17.7
Akçelik (1980)	-1.2	8.8	-1.4	10.8	-5.9	18.4
SIDRA (1995)	-1.4	9.2	-1.9	10.6	-3.9	22.8

It is interesting to note the very high standard deviations, especially at the higher degrees of saturation which is generally considered to be the critical evaluation scenario. Most of the later models tend to underestimate the delay on average, with the earlier models apparently providing better results.

A further evaluation was conducted where provision was made for non-random effects, with the inclusion of the I-factor. The relevant models shown in Table 1 were modified by Pretorius (2001) to include the non-random factor, with the results shown in Table 3.

Table 3. Modelled error in some existing delay models (incl. I-factor).

Model	Average and standard deviation of modelling errors (% of cycle length) for degrees of saturation of :					
	±85%		±90%		±95%	
	Avg	Std Dev	Avg	Std Dev	Avg	Std Dev
Webster (1958)	4.9	2.4	5.4	2.5	6.4	2.5
Miller (1963)	4.6	2.1	5.2	2.3	6.3	2.4
Newell (1965)	-0.2	0.3	-0.2	0.5	0.1	0.9
Miller (1968)	0.0	0.7	-0.1	0.5	-0.1	1.6
TRANSYT 8 (1980); HCM	6.8	4.7	7.0	4.1	8.2	4.7
Akçelik (1980)	-0.3	3.0	-0.3	2.6	-2.3	4.3
SIDRA (1995)	-0.5	4.5	-0.6	7.5	-0.5	20.1

The evaluation indicated a significant improvement in results for steady-state conditions, especially with the Newell (1965) and Miller (1968) models. A significant reduction in the standard deviation of errors was obtained for most of the models. It should be noted that the Miller model already included the non-random arrival factor in the original version there-of.

The results shown in Table 3 are based on the assumption that the estimation of non-random arrival effects could be determined accurately in practical applications. Unfortunately, this was shown to be a major stumbling block in the determination of delay (Pretorius, 2001). Nevertheless, it can be seen that the non-random arrival factor is very important in the estimation of delay and will thus be discussed in more detail in the following sections of the paper.

6. THE EXTENT OF NON-RANDOMNESS IN THE URBAN STREET NETWORK

The number of vehicles arriving fluctuates from cycle to cycle and may follow a variety of probability distributions. These distributions, such as the Binomial, Negative Binomial, Normal and other distributions present a mathematically involved way to analyse traffic operations at intersections. In an evaluation of alternative arrival distributions, Allsop (1972) noted that: "*The mathematical fascination of such analyses has, however, caused them to be pursued to a level of intricacy far greater than is likely to find practical application.*" However, a simplification could be introduced to accommodate non-random effects in arrival patterns by means of a coefficient of variance term of the following form:

$$I = \frac{\text{Variance of number of arrivals per cycle}}{\text{Average number of arrivals per cycle}}$$

This term was proposed by Cox in 1955 and later by Miller in 1963 and Newell in 1965. However, for some reason this critical element was largely neglected in practical delay models until Newell (1990), Van As (1991a), Fambro and Rouphail (1997) and Pretorius (2001) refreshed the need to address non-random arrivals.

It should be noted that the coefficient of variance term is equal to 1,0 when traffic arrivals are random according to the Poisson process. This was considered to be a reasonable assumption in delay models to date, but surveys by Pretorius (2001) indicated that the extent of non-randomness cannot be ignored.

The surveyed values indicated in Table 4 are some examples of the coefficient of variance term which deviates from unity (and randomness) under normal peak time conditions. Values significantly less than one was found where traffic arrivals tend to be more uniform, with values larger than one where a high degree of fluctuation occurs. The Highway Capacity Manual (TRB, 2000) currently makes provision only for a coefficient of variance term of one, or less than one. The very high values (greater than one) of I obtained during the study were not anticipated at the start of the study (the low values of smaller than one were expected).

Table 4. Surveyed coefficient of variance term.

Intersection	Location	Peak Hour	Observed coefficient of variance of arrivals
Charles/ Duncan	Pretoria	AM	1,74
Brooklyn / Lynnwood	Pretoria	PM	2,54
Coronation / Boomerang / Hale	Brisbane	AM Inbound to City	0,34
Coronation / Boomerang / Hale	Brisbane	AM Outbound from City	2,00
Alice / George	Brisbane	PM	2,89

7. THE EFFECT OF NON-RANDOM TRAFFIC ARRIVALS ON DELAY

The importance of taking the degree of fluctuations into account is illustrated in Figure 1. The delays given in the figure were established by means of the model developed by Pretorius (2001). The model was based on a modification to the Newell (1965) delay model, as proposed by Pretorius (2001) to improve accuracy. The figure shows the average delay experienced by vehicles at traffic signals as a function of degree of saturation and for different degrees of fluctuation in arrivals.

The figure shows three graphs for the following situations:

- I = 0.2 - Low degree of fluctuation, traffic nearly uniform
- I = 1.0 - Random arrivals
- I = 2.8 - High degree of fluctuation, significant variation in traffic arrivals

The figure shows that assumptions regarding fluctuations in arrivals could make one or more levels difference in the estimation of Level of Service. In some cases close to saturation conditions, the Level-of-Service would be A when the degree of fluctuation is low and at the boundary of D and E if the fluctuation is high. Under random arrival conditions, a Level-of-Service C could be estimated. It is therefore very important to take the degree of fluctuation in arrivals into account in delay models.

Although it is possible to take the degree of fluctuation into account in delay models, the collection of data to quantify such degree of fluctuation could be a difficult and expensive process. It is possible to develop some models for estimating this degree of fluctuation (TRB, 2000; Pretorius, 2001), but these models are relatively limited in scope and a significant research effort would be required to develop models in which all important factors are taken into account.

The Highway Capacity Manual model (TRB, 2000) is one of a few models that make some provision for variation in degree of fluctuation, but then only to a limited extent. The model will have to be expanded significantly (Pretorius, 2001) if this factor is to be taken into account adequately.

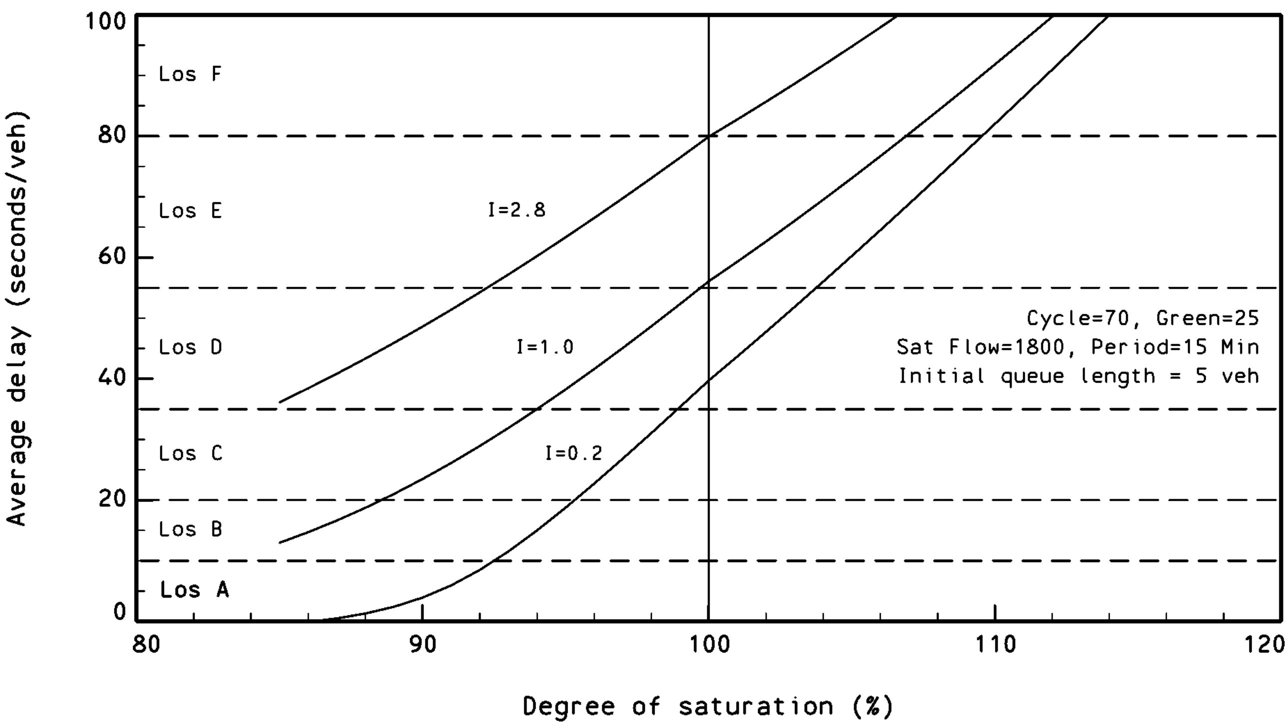

Figure 1. Sensitivity of delay at traffic signals to fluctuations in traffic arrivals.

8. TIME DEPENDENCY

Many of the earlier models have only been developed for steady state conditions in which it is assumed that queuing properties remain stationary in time. Equilibrium conditions exist in which the intersection is undersaturated and delays and queue lengths are constant when averaged out over a long period of time. These models are not concerned with the dynamics of queue formation during peak periods, and the time it takes for the queue to reach equilibrium is ignored. These models can only be used for the analysis of traffic signals that operates at low degrees of saturation and are not suitable for peak period analyses. The use of steady state models during peak periods can introduce significant errors and time dependent models are required for such periods.

Various attempts have been made by a number of researchers to develop detailed time dependent delay models. The studies have indicated that detailed time dependent solutions of even the simplest M/M/1 queuing models (single server queues with Poisson arrivals and service times) are generally very intractable. The solutions for more complex queuing models are even more intractable, making it nearly impossible to use in practical traffic engineering applications.

Newell (1971, 1982) considered the application of diffusion theory to produce approximations to complex queuing problems. The approach of diffusion theory in traffic engineering applications was largely ignored for a number of years until Troutbeck and Blogg (1998) and Blogg (1999) illustrated that the approach could be made more accessible for practical applications. The diffusion theory approach is considered to be less of an approximation than the co-ordinate transformation method that is generally used in delay models to date, including this study undertaken by Pretorius (2001). However, the application of the diffusion theory approach in practical traffic engineering problems is still less workable than the co-ordinate transformation method.

The co-ordinate transformation technique was developed originally at the Transportation Research Laboratory by researchers such as Whiting (reported by Kimber and Hollis, 1979), Catling (1977), Kimber, Marlow and Hollis (1977), Kimber and Hollis (1979), Mayne (1978), Robertson (1979) and Akcelik (1980,1981). It is an approximate empirical method in which a steady-state overflow equation is transformed to a transition function that has the steady-state overflow model as one asymptote when traffic flows are very low, and the deterministic overflow model as a second asymptote when the intersection is heavily oversatured. A typical example of the application of the co-ordinate transformation technique is given in Figure 2.

Kimber and Hollis (1979) justified the use of the co-ordinate transformation technique on the basis that the more detailed models require lengthy computer calculations while the simpler models produce predictions that are close to those of the more detailed models.

Pretorius (2001) presented an example of the realistic application of a time dependent model to the complete peak period traffic flow in a number of different time periods. This concept is illustrated in Figure 3. The figure shows queue growth and decay over three time periods. The initial queue was assumed to be zero at a time before the onset of peak traffic flow conditions. The signal is undersaturated during time period 1 and the overflow queue tends to converge to an equilibrium value according to steady state theory. During time period 2, however, the signal is oversaturated, and the queue continues to grow rapidly. The signal is again undersaturated in time period 3, and the queue length shows a rapid decay to a low equilibrium queue length.

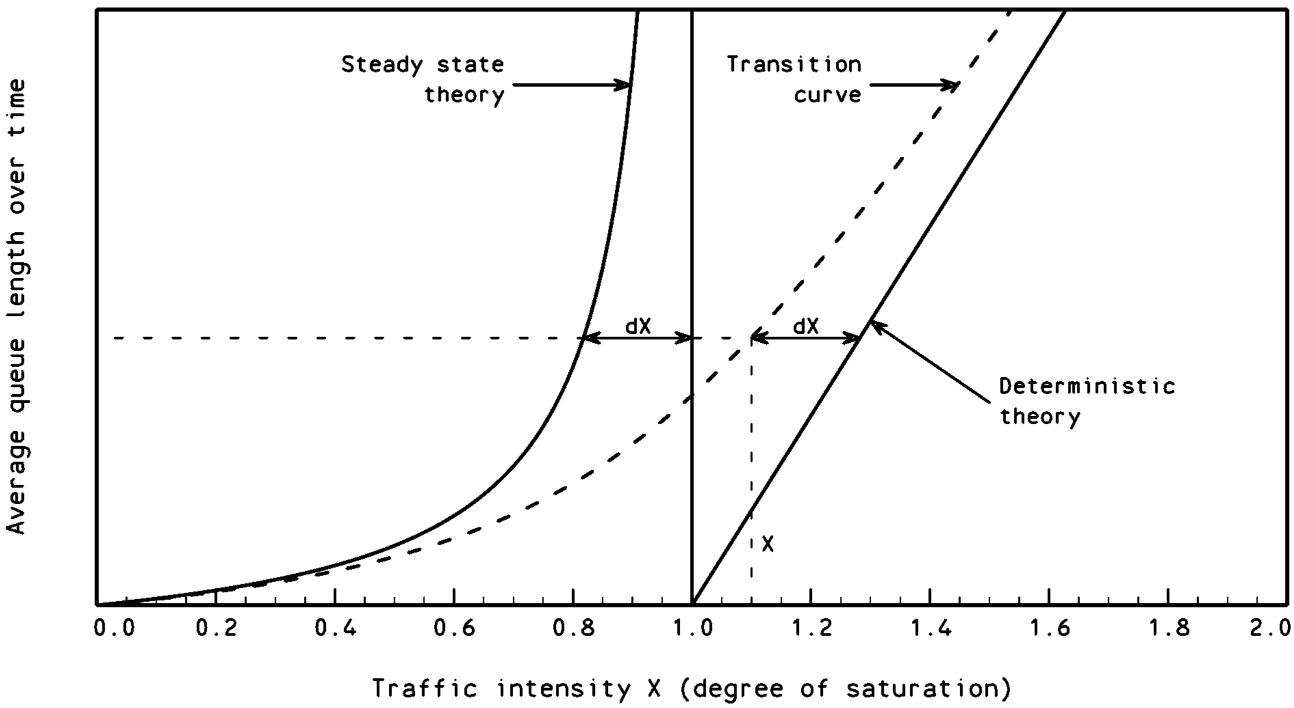

Figure 2. Example of the application of the co-ordinate transformation technique.

Most models currently in use make some provision for variation in queue length over time, but the majority of models ignore such variation over a number of different time intervals. In such models, it is generally assumed that the queue grows from a zero queue length at the start of the time interval being analysed. This assumption is probably one the main reasons why models often seriously underestimate delay at intersections. The Highway Capacity Manual model (TRB, 2000) is one of a few models that make adequately provision for variation of queue length over time. An alternative model was also developed by Pretorius (2001). Both these models, however, are complicated to use and require extensive calculations.

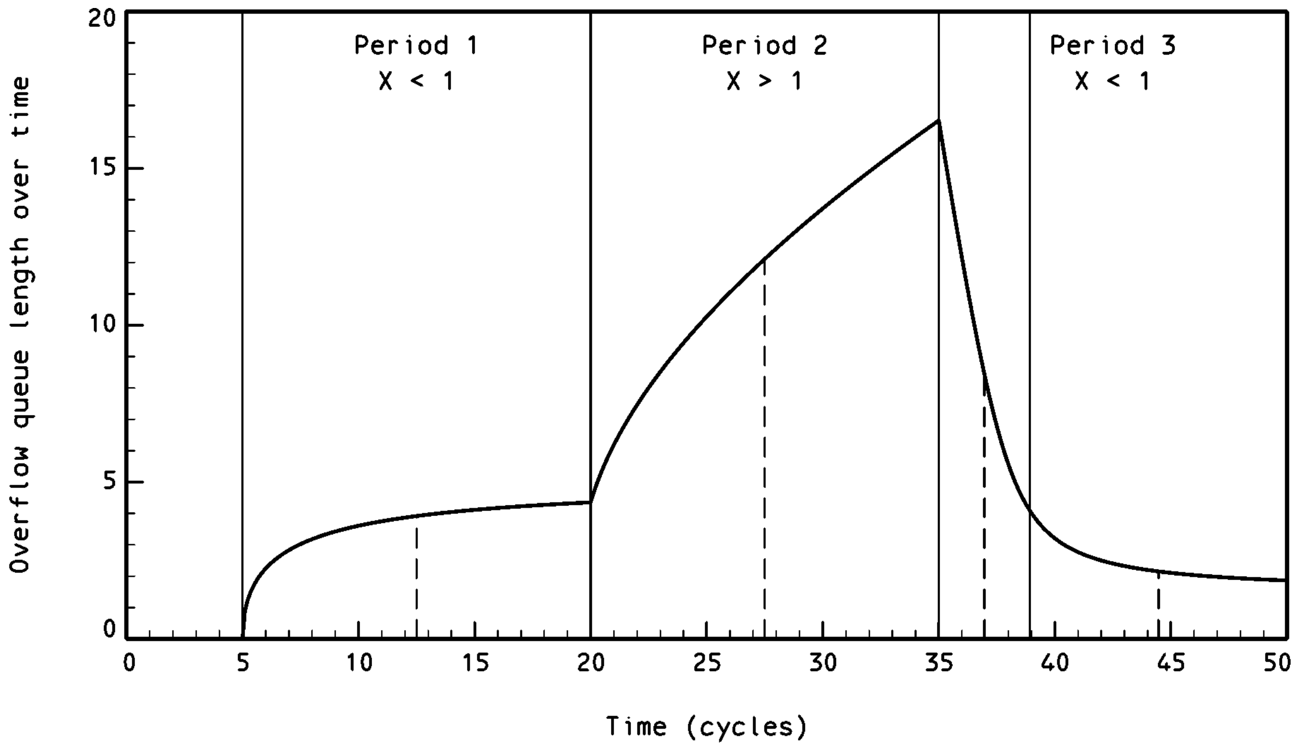

Figure 3. Example of a time dependent queuing model.

9. THE VOLUME/CAPACITY RATIO AS A NORM

The various available delay models were extensively evaluated by Pretorius (2001) using Monte Carlo simulation models. The main conclusion of the study is that some of the generally applied delay models have serious limitations that need to be addressed. Some of the models, such as those developed by Newell (1965) and Miller (1968) performed better than others, including models that were developed later. All the models, however, were found to have serious shortcomings.

It could be possible to develop a delay model that provides an acceptable level of accuracy, but this would be a complex task, relying on a number of parameters that are difficult to estimate. The question can therefore be asked whether the effort of developing such a model would be worthwhile while a simpler measure of effectiveness such the volume/capacity model may be available that could be acceptable for practical applications.

The estimation of the volume/capacity ratio is significantly simpler than estimating delay at an intersection. Although it is realised that this ratio has some limitations, these limitations are probably minor, compared to the difficulty in estimating delay.

Pretorius (2001) has undertaken a very large number of computer simulation studies at traffic signal controlled intersections for a very wide range of conditions. The results of all these studies are summarised on one graph shown in Figure 4, in which delay is plotted against degree of saturation. The figure shows that for degrees of saturation of less than about 90%, delays would be less than about 70% of the cycle length. For a cycle length of 100 seconds, this would correspond with an average delay of 70 seconds per vehicle. This delay is slightly below the 80 seconds delay per vehicle that is the maximum for a Level of Service E. This level of service has become the de-facto norm during peak hours in many cities throughout the world including South Africa. Funds are no longer freely available for providing higher Levels of Service. It may therefore be adequate to select a simple norm such as 80% or 90% degree of saturation for the evaluation of performance at traffic signals.

The new Draft Volume 3: Traffic Signal Design of the South African Road Traffic Signs Manual (NDOT, 2001) recommends a degree of saturation of 85% for "isolated" traffic signals and 90% for traffic signals in networks. The larger degree of saturation in networks is allowed because traffic fluctuations tend to be lower than at isolated intersections.

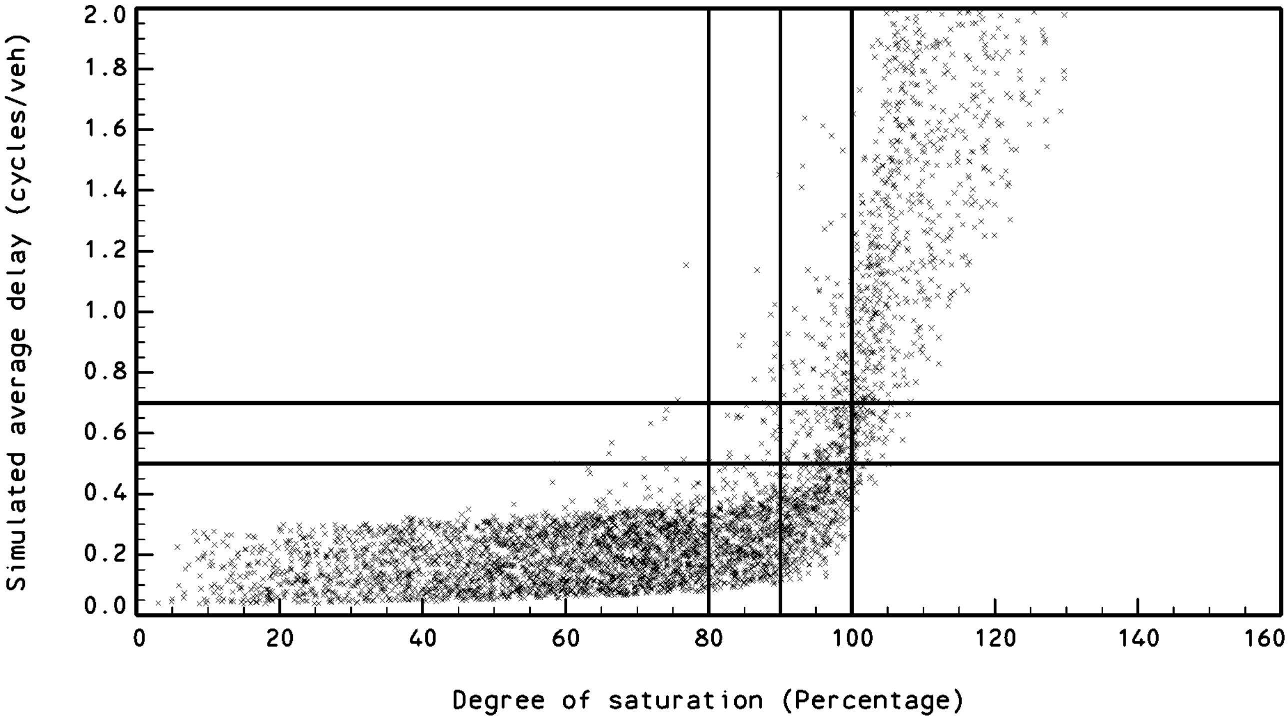

Figure 4. Average vehicular delay and degree of saturation for traffic signals.

10. CONCLUSIONS AND RECOMMENDATIONS

This paper has demonstrated that current delay models for intersections have serious limitations and that the accuracy of the models can be suspect.

This is primarily because of the following factors:

- A significant variation in traffic arrival patterns was found at signalised intersections, the estimation of which is either omitted, or not accurately included in the existing delay models. A significant research effort would be required to develop models to predict the extent of variation in arrival patterns.
- The application of time dependant analysis principles during peak periods is not sufficient in most delay models. Most of the models currently in use make some provision for variation in queue length over time, but the majority of models ignore such variation over a number of different time intervals. In such models, it is generally assumed that the queue grows from a zero queue length at the start of the time interval being analysed. This incorrect assumption is probably one the main reasons why models often seriously underestimate delay at intersections.

It is noted that is could be possible to develop more accurate delay models, but these models would be highly complex and would require a significant research effort to calibrate. The volume/capacity ratio is significantly simpler and a more robust norm which, although it has its own limitations, may be more appropriate for some practical applications until such time as research has been conducted to address the problems associated with the estimation of delay.

It is recommended that road authorities should consider the adoption of the volume/capacity ratio as the prime norm for the evaluation of traffic operations at signalised intersections. Delay can still be retained as a secondary norm in situations where the volume/capacity ratio may not be appropriate, but some additional research is required to identify such situations.

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ISSUES WITH THE OPERATIONAL ANALYSIS OF URBAN INTERSECTIONS

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