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REPRINT

Implementation of the Australian roundabout analysis method in SIDRA

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NOTE:

This paper is related to the intersection analysis methodology used in the SIDRA INTERSECTION software. Since the publication of this paper, many related aspects of the traffic model have been further developed in later versions of SIDRA INTERSECTION. Though some aspects of this paper may be outdated, this reprint is provided as a record of important aspects of the SIDRA INTERSECTION software, and in order to promote software assessment and further research.

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Implementation of the Australian roundabout analysis method in SIDRA

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ABSTRACT: Progress towards the development of a comprehensive Australian method for the analysis of capacity and performance of roundabouts is reported. The new Australian roundabout design guide to be produced during 1991 will include a significant revision of the current capacity analysis method by the incorporation of the results of Australian research. This method allows for the effects of circulating flows, entry flows and roundabout geometry on gap acceptance parameters. The new Australian roundabout analysis method will be implemented in the SIDRA package which was originally developed for signalised intersection analysis. As a computerised method, SIDRA will provide some significant advantages in the implementation of the capacity and performance analysis method for roundabouts. These include an iterative method to calculate circulating flows with capacity limitation for oversaturated approaches, and techniques for estimating entry lane flows, shared lane capacities and time-dependent delays.

1 INTRODUCTION

The objective of this paper is to discuss the implementation of the Australian capacity and performance analysis method for roundabouts in the SIDRA package (Akcelik 1987, 1990; Akcelik and Besley 1990).

The background to the methodology to be adopted is explained briefly and the method is described in some detail. The reader is referred to Akcelik (1991) for a more comprehensive report which also includes the discussion of other unsignalised intersection cases.

Currently, the AUSTROADS (formerly NAASRA) roundabout design guide (NAASRA 1986) is under revision. A new guide as *Part 6 of the AUSTROADS Guide to Traffic Engineering Practice* will be produced during 1991. The new guide will include a significant revision of the current capacity analysis method by the incorporation of the results of research carried out by Troutbeck at the Australian Road Research Board, and more recently at the Queensland University of Technology under an ARRB contract (see the reference list: Troutbeck 1984 to 1991). The main reference for the purpose of this paper is Special Report No. 45 (Troutbeck 1989) which will be referred to as *SR 45*.

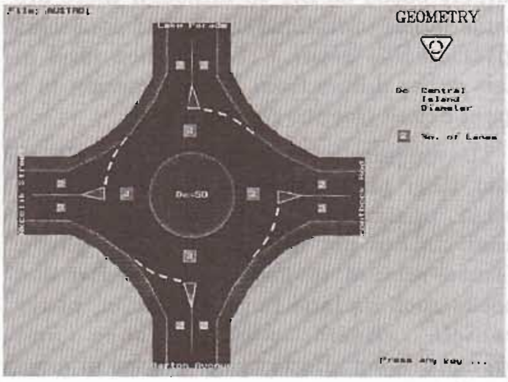

Fig. 1 – Roundabout geometry picture generated by RIDES (SIDRA)

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The new roundabout capacity analysis method can be seen as an extension of the *gap acceptance* techniques used in the current AUSTROADS method (hence different from the empirical approach used in the U.K.). The new method introduces the effects of circulating flows, entry lane flows and roundabout geometry on gap acceptance parameters (see *Table 1*).

AUSTROADS Guide to Traffic Engineering Practice prescribes the same method for the analysis of signalised intersections as used in SIDRA. Thus, SIDRA will be a comprehensive intersection analysis tool which complements the AUSTROADS intersection design guide.

A graphics-based input data editor, RIDES (Road Intersection Data Editing System), which will be part of the SIDRA package, has been designed for easy data specification for all types of intersection including roundabouts. The roundabout geometry picture drawn by RIDES is shown in *Fig. 1*.

As a computerised method, SIDRA will provide some significant advantages in the implementation of the capacity and performance analysis method for roundabouts and other unsignalised intersections. These are summarised in *Table 2*.

In SIDRA, the capacity estimation method adopted for roundabouts will be largely based on *SR 45* although there are some minor variations in various equations given in this paper. The notation used in this paper differs from *SR 45* notation to a great extent (see the *Notation* list).

The subjects covered extensively in *SR 45* and other publications on roundabout and other unsignalised intersection capacity analysis are not discussed in this paper. Issues related to major model extension needs such as lane flow estimation, heavy vehicle effects, shared lanes and oversaturation effects are discussed in more detail.

Table 1. ARRB roundabout research results

Gap acceptance parameters are related to roundabout geometry as well as circulation and entry flows as follows:

- Entry stream *minimum departure headway* (follow-on time) decreases (hence entry capacity increases) with
  - increasing circulation flow (*)
  - increasing diameter of the roundabout
  - decreasing number of circulation lanes
  - increasing number of entry (approach) lanes
- *Mean critical gap* is proportional to the minimum departure headway. The ratio of the critical gap to the minimum departure headway is in the range 1.1 to 2.1, and decreases (hence capacity increases) with increasing
  - circulation flow (*)
  - number of circulating lanes
  - average entry (approach) lane width
- For approach roads with more than one entry lane, the traffic in the lane with the largest flow rate is called *dominant stream* and the traffic streams in other lanes are called *subdominant streams*.
  - The minimum departure headway for a subdominant stream is greater than the minimum departure headway for the dominant stream.
  - The ratio of minimum departure headways for the subdominant and dominant streams increases as the ratio of dominant stream flow to subdominant stream flow increases (the mean ratio is 1.2).

  Thus, entry capacities depend on entry flows, requiring an iterative method to estimate lane flows and capacities.

(*) Hence driver behaviour patterns change with circulation flows (more vehicles can depart through an acceptable gap; and shorter critical gaps are accepted with increasing circulation flows). This confirms Kimber's (1989) observations.

Table 2. Advantages to be introduced through implementing the roundabout analysis method in SIDRA

- Iterative method to calculate circulating flows at roundabouts: This will allow for *capacity constraint* for oversaturated approaches (particularly important for unbalanced flow cases).
- *Capacity constraint* for oversaturated opposing streams will also be applied at unsignalised intersections (this can occur especially when an opposing stream itself is opposed by other movements; iterative calculations are required for this reason).
- Different critical gap and minimum departure headways for different turns (left, through, right) from the same approach can be used.
- Entry lane flow estimation as a function of lane capacities: Iterative calculations required due to the dependence of the follow-on headway on dominant-subdominant flow ratios at roundabout entries will be carried out.
- Lane underutilisation for entry lanes: Unequal approach lane capacity utilisation will be allowed for.
- *Opposing flows*:
  - Number of lanes variable;
  - Unequal lane utilisation allowed;
  - Variable (flow dependent) or constant bunching allowed.
- Time-dependent delay formula (applies to oversaturated cases).
- Proportion of stops: congestion effects to be taken into account.
- Estimates of geometric delay, stopped delay, number of stops, fuel consumption, pollutant emissions, etc. based on detailed lane-by-lane modelling.
- Consistency of capacity and performance analysis methods for roundabouts, other unsignalised and signalised intersections.

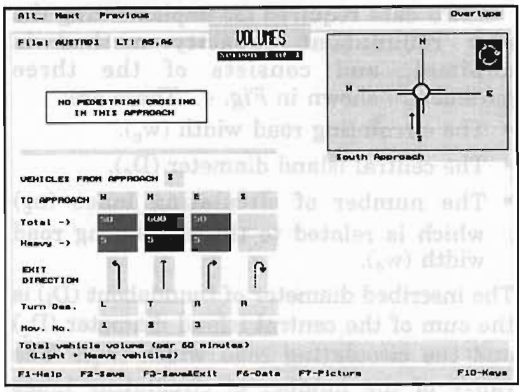

Fig. 2 – RIDES data screen for approach volumes

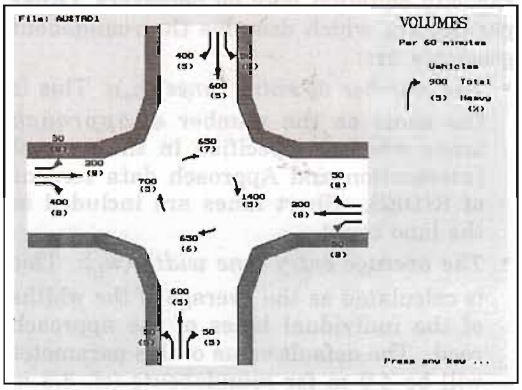

Fig. 3– Picture generated by RIDES for approach and circulating volumes

2 DATA FOR ROUNDABOUTS

The specification of data for roundabouts using RIDES input data editor in the SIDRA package is discussed in detail in Akcelik (1991). An important change introduced with RIDES which applies to all types of intersection is a new style of volume data specification. Instead of volumes by *Left, Through and Right* turns, an *Origin-Destination* style of volume specification (*From Approach – To Approach*) is used. RIDES volume data screen and a picture generated by RIDES for approach and circulating stream flows are shown in *Figs 2 and 3.*

Extra data required for implementing the new roundabout capacity method is minimal, and consists of the three parameters shown in *Fig. 4*. These are:

- The circulating road width (w_c).
- The central island diameter (D_c).
- The number of circulating lanes (n_c) which is related to the circulating road width (w_c).

The inscribed diameter of roundabout (D_i) is the sum of the central island diameter (D_c) and the circulating road width (w_c). The range of the number of circulating lanes used to derive equations given in *SR 45* was 1 – 3 lanes. For n_c greater than this value, different settings of parameters for the capacity equation may be necessary. Other parameters which describe the roundabout geometry are:

- *The number of entry lanes* (n_e): This is the same as the number of *approach* lanes which is specified in the general Intersection and Approach data screens of RIDES. Short lanes are included in the lane count.
- *The average entry lane width* (w_L): This is calculated as the average of the widths of the individual lanes of the approach road. The default value of this parameter will be 4.0 m for roundabouts (cf. 3.3 m for signalised intersections).

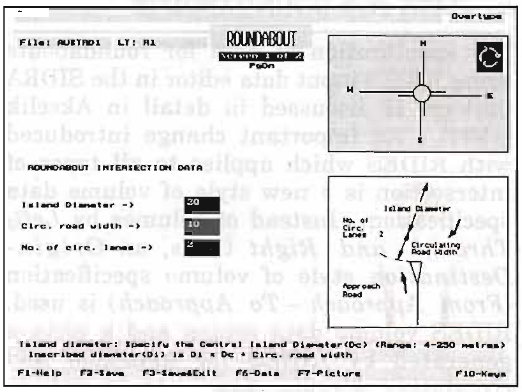

Fig. 4 – RIDES screen for roundabout data

3 CAPACITY MODEL FOR OPPOSED TRAFFIC STREAMS

In SIDRA, the capacity of an opposed traffic stream is estimated from a model based on gap acceptance theory. *Opposed* traffic streams can be:

(a) entry flows at roundabouts,

(b) minor (non-priority) flows at unsignalised intersections, or

(c) filter (permitted) right or left turns, including slip lane traffic and turns on red, at signalised intersections.

The term *traffic stream* is used in a very general sense to specify an entity which has a single set of gap acceptance parameters, and it may mean:

(a) an individual movement (left, through or right turn),

(b) an individual lane (all turns in the lane considered together),

(c) a lane group (several lanes combined), or

(d) an approach road (all lanes of the approach road treated as one lane group).

The application of the capacity and performance models depends on how the opposed and opposing traffic streams are defined, and the results may differ according to the definitions used.

SIDRA provides the flexibility to use any one of the traffic stream definitions given above. Generally, the preferred method is to treat opposed and opposing streams at individual lane and movement levels.

In the case of roundabouts, the opposing traffic stream (circulating flow) is treated as a single entity as specified in *SR 45*. This is different from the lane-by-lane method used for other unsignalised and signalised intersections.

3.1 Capacity model parameters

The parameters used in the gap-acceptance based capacity model are:

(a) Parameters relating to the *opposed (entering*, or *non-priority)* stream:

- Follow-on time (minimum departure headway)
- Mean critical gap

(b) Parameters relating to the *opposing (circulating,* or *priority)* stream:
- Intra-bunch headway (minimum headway)
- Proportion of free (unbunched) vehicles

3.1.1 Follow-on time and mean critical gap

Follow-on time (the minimum departure headway) and *mean critical gap* are the two main parameters used for estimating the capacity of an opposed traffic stream. The follow-on time (β) for the opposed traffic lane is the minimum departure headway which corresponds to the maximum opposed turn capacity, $Q_m = 3600/\beta$.

Usually, constant values of these parameters are used. *SR 45* improves on this by relating these parameters to the roundabout geometry as well as circulating and entry flows.

An important feature of *SR 45* is to treat various lanes of multi-lane approach roads as *dominant* and *subdominant* lanes (streams) which are assigned different values of gap acceptance parameters. The traffic in the lane with the largest flow rate is called *dominant stream* and the traffic streams in other lanes are called *subdominant streams* (see *Section 4* for detailed discussion on dominant and subdominant lanes).

Follow-on time and mean critical gap for dominant and subdominant streams at a *roundabout entry* are estimated as follows.

Firstly, the *dominant stream follow-on time* (in seconds) is estimated from:

$$\begin{aligned}\beta_d &= 3.37 - 0.0208\, D_i + 0.889\text{x}10^{-4}\, D_i^2 \\ &\quad - 0.395\, n_e + 0.388\, n_c \\ &\quad - 3.94\text{x}10^{-4}\, q_c \qquad \textit{for } D_i \le 100 \\ &= 2.179 - 0.395\, n_e + 0.388\, n_c \\ &\quad - 3.94\text{x}10^{-4}\, q_c \qquad \textit{for } D_i > 100 \\ &\qquad \textit{subject to } \beta_d \ge 0.8 \text{ s} \end{aligned} \tag{3.1}$$

where D_i is the inscribed diameter (m), n_e is the number of entry lanes, n_c is the number of circulating lanes, and q_c is the circulating flow rate (veh/h).

Fig. 5 shows the dominant stream follow-on time as a function of the circulating flow as calculated from Eqn (3.1) for inscribed diameters of D_i = 30, 60 and 100 m (the number of entry and circulating lanes, $n_e = n_c = 2$). It is seen that the dominant stream follow-on headway decreases with increasing roundabout diameter and increasing circulating flow rate.

Secondly, the *subdominant stream follow-on time* (in seconds) is estimated from:

$$\beta_s = 2.149 + (0.5135\, \beta_d - 0.8735)\, r_{ds} \qquad \textit{subject to } \beta_s \ge \beta_d \tag{3.2}$$

where β_d is the dominant stream follow-on headway (sec), and r_{ds} is the ratio of entry lane flows (the ratio of dominant to subdominant stream flow rate):

$$r_{ds} = q_d / q_s \tag{3.2a}$$

With equal lane flows, $r_{ds} = 1.0$ ($q_d = q_s$):

$$\beta_s = 1.2755 + 0.5135\, \beta_d \qquad \textit{subject to } \beta_s \ge \beta_d \tag{3.2b}$$

For a given β_d, the subdominant stream follow-on time, β_s, increases (capacity decreases) with increasing r_{ds} values. Thus, the total approach capacity will decrease when the lane utilisation is more unbalanced.

Thirdly, the *mean critical gap* (in seconds) is calculated for each dominant and subdominant stream from:

$$\begin{aligned}\alpha &= (3.6135 - 0.339\, w_L - 0.2775\, n_c \\ &\quad - 3.137\text{x}10^{-4}\, q_c)\, \beta \\ &\qquad \textit{subject to } \alpha/\beta \ge 1.1\end{aligned} \tag{3.3}$$

where w_L is the average entry lane width (m), n_c is the number of circulating lanes, q_c is the circulating flow rate (veh/h), and β is the follow-on headway (sec).

The ratio α/β is the same for dominant and subdominant streams.

For a given follow-on time, the critical gap will be smaller (hence the capacity higher) for roundabouts with more circulating lanes. The critical gap will be reduced (hence the capacity will be higher) with increasing circulating flow levels.

For *single-lane* roundabout entries (n_e = 1), the dominant stream formula (*Eqn 3.1*) will be used for follow-on headway calculation. When $n_e > 1$, Eqn (3.1) or (3.2) will be applied according to the dominant or subdominant lane specification.

It is important to note that, even when the entry lanes have equal flows, the capacities of the subdominant lanes will be smaller than the capacities of the dominant lanes due to the larger follow-on headway ($\beta_s \geq \beta_d$) according to Eqn (3.2b) and the larger mean critical gap ($\alpha_s \geq \alpha_d$) according to Eqn (3.3). Thus, the designation of dominant and subdominant lanes is a critical aspect of the *SR 45* method (see *Section 4*).

3.1.2 Intra-bunch headway and proportion of free vehicles

Intra-bunch headway and *proportion of free vehicles* are the two other parameters used in the gap acceptance model for capacity estimation. Both are related to the opposing (circulating, or priority) stream.

The intra-bunch headway (Δ) is the minimum headway value within each bunch (equal for all bunches), and the proportion of free vehicles represents the unbunched vehicles with randomly distributed headways.

For roundabout capacity estimation, the circulating flow will be treated as a single stream, i.e. *not* lane by lane, but will be qualified as a *single-lane* or *multi-lane* circulating flow case according to the number of circulating lanes, i.e. n_c =1 and $n_c > 1$, respectively.

The *intra-bunch headway* is given by:

$$\Delta = 2 \text{ sec} \quad \textit{for single-lane circulating road}$$
$$= 1 \text{ sec} \quad \textit{for multi-lane circulating road} \qquad (3.4)$$

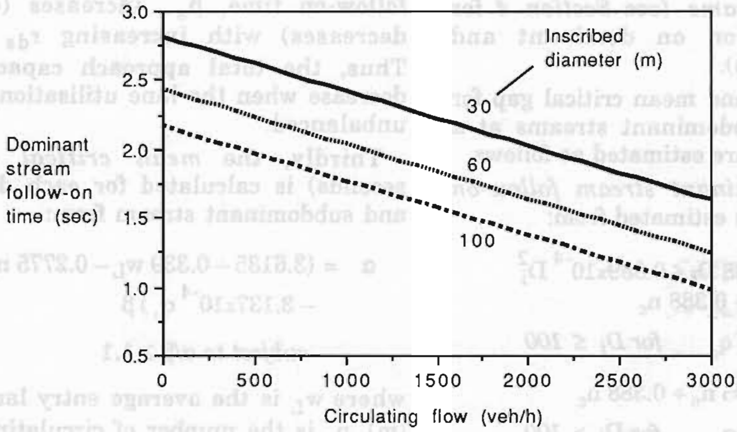

Fig. 5 – Dominant stream follow-on time, β_d as a function of the circulating flow, q_c (for various inscribed diameters, D_i (the number of entry and circulating lanes: $n_e = n_c = 2$)

The intra-bunch headway for a traffic lane corresponds to the lane saturation flow (uninterrupted stream capacity) through $\Delta = 3600 / s$, where s is the lane saturation flow (veh/h), e.g. $\Delta = 2$ sec corresponds to s = 1800 veh/h. The value of Δ is important in determining the entry capacity for very high opposing (circulating) flow values since capacity is zero at the point where the opposing flow equals $3600/\Delta$.

The *proportion of free vehicles* can be estimated from the following linear bunching model which has been obtained by a slight adjustment to Eqns (7 to 10) of *SR 45*:

$$\varphi = 0.75\,(1 - \Delta\, q_c) \qquad \textit{subject to } \varphi \geq 0 \tag{3.5}$$

where Δ is the intra-bunch headway for the opposing traffic stream (from Eqn 3.4) and q_c is the circulating flow rate (veh/sec).

This linear bunching model assumes that the proportion of free vehicles decreases linearly with increasing opposing flow rate. For very low flows ($q_c \approx 0$), $\varphi \approx 0.75$ and for very high flows ($q_0 \approx 1/\Delta$), $\varphi \approx 0$ (all vehicles bunched when flow rate approaches the saturation flow rate, and hence $\Delta\, q_c \approx 1.0$). Thus, even under very low flow conditions, circulating streams at roundabouts indicate some bunching, which can be explained in terms of the gap acceptance process.

SR 45 recommends the adjustment of the proportion of free vehicles according to the proximity of signalised intersections to the roundabout. When a fixed φ value which is considered to represent observed bunching effects is to be specified, it should be realised that circulating flows at a roundabout are filtered through a gap acceptance process. Therefore, the bunching of circulating flows should depend on the characteristics of all flows entering the roundabout (Eqn 3.5 which is based on empirical data is considered to reflect this process in a simple way).

3.2 Capacity of opposed streams

SIDRA uses a general gap-acceptance based capacity model for all types of intersections (Tanner 1967; Gipps 1982; Troutbeck 1984, 1986). For signalised and unsignalised intersections other than roundabouts, the general model is applied with the opposing stream flows treated lane by lane. In these cases, the same value of the intra-bunch (minimum) headway, Δ, is used for all opposing lanes. Constant follow-on and critical gap times (α, β) are used. The general model is applicable to the case of *several opposing streams* for signalised and unsignalised intersections other than roundabouts.

For roundabouts, the general capacity model is applied with the circulating (opposing) flow treated as a single stream, i.e. not lane by lane. The formula for the capacity of a roundabout entry is (as given in *SR 45*):

$$
\begin{aligned}
Q_e &= \frac{3600\, \varphi\, q_c\, e^{-\lambda(\alpha-\Delta)}}{1 - e^{-\lambda\beta}} && \textit{for } \lambda > 0 \textit{ and } q_c > 0 \\
&= 3600/\beta_0 && \textit{for } q_c = 0 \\
&= 0 && \textit{for } \lambda = 0
\end{aligned}
\tag{3.6}
$$

where parameter λ is given by

$$
\begin{aligned}
\lambda &= \frac{\varphi\, q_c}{1 - \Delta q_c} && \textit{for } q_c \leq 1/\Delta \\
&= 0 && \textit{for } q_c > 1/\Delta
\end{aligned}
\tag{3.7}
$$

and

Q_e = capacity of the roundabout entry (veh/h),

q_c = total flow rate for the circulating stream (*veh / sec*),

α = mean critical gap (Eqn 3.3),

β = follow-on time (Eqn 3.1 or 3.2),

β_0 = follow-on time for zero circulating flow (for $q_c = 0$ in Eqn 3.1 or 3.2),

Δ = intra-bunch headway (Eqn 3.4), and

φ = proportion of free vehicles (Eqn 3.5).

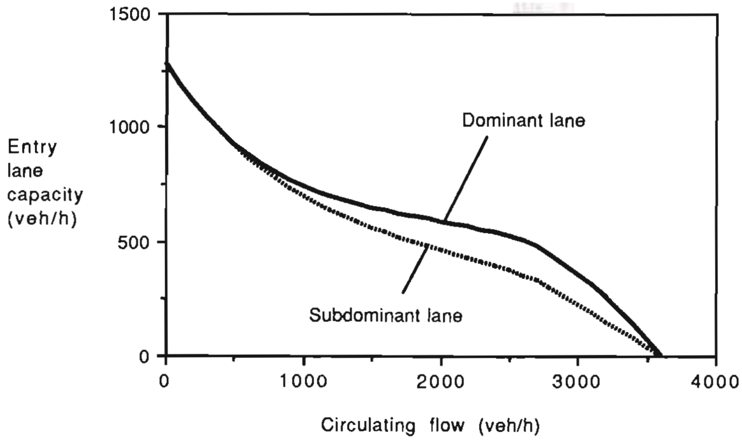

Fig. 6 – Entry lane capacity, Q_e as a function of the circulating flow (for the subdominant lane capacity, equal entry lane flows, $r_{ds} = 1.0$ assumed; $D_i = 30$ m, $n_e = n_c = 2$, $w_L = 3.3$ m)

Fig. 6 shows dominant and subdominant entry lane capacities (Q_d , Q_s) as a function of the circulating flow, q_c, for the case when $D_i = 30$ m, $n_e = n_c = 2$, $w_L = 3.3$ m. For the subdominant lane capacity, the ratio of entry lane flows, $r_{ds} = 1.0$ is assumed. For low circulating flows, $\beta_s = \beta_d$ results in equal capacities ($Q_d = Q_s$) for the dominant and subdominant lanes. For higher flows, subdominant lane capacities are increasingly smaller ($Q_d / Q_s > 1.0$). This would mean that the assumption $r_{ds} = 1.0$ is not correct since less flow would use the subdominant lane. Therefore, several iterations are required to find equilibrium values of r_{ds} and Q_s (see *Section 4* on lane utilisation).

3.3 Shared lane capacities

In accordance with *SR 45,* roundabout capacities will be calculated for each lane using the same capacity parameters for all turns from the lane. At other unsignalised intersections, a lane may be shared by

(a) two (or more) opposed movements with different opposed turn parameters, or

(b) an opposed turn and a priority (unopposed) movement; for example, under Victoria/New Zealand rule, left turns from the main road give way to right turns, and hence shared left and through lanes always contain mixed opposed and unopposed movements.

The general formula for calculating the capacity of a mixed stream (shared lane) is:

$$Q = \frac{q}{x} \tag{3.8}$$

where

$$x = \Sigma x_i = \Sigma \frac{q_i}{Q_i} \tag{3.8a}$$

Q = capacity of the mixed stream (veh/h),

q = total flow rate for the mixed stream (veh/h), $q = \Sigma q_i$,

x = degree of saturation of the mixed stream,

q_i = flow rate of *ith* stream in the shared lane,

Q_i = capacity of the *ith* stream in the shared lane, and

x_i = degree of saturation of the *ith* stream in the shared lane.

Eqn (3.8) determines the degree of saturation of the mixed flow (shared lane) as the sum of the degrees of saturation of component streams which are calculated as if each stream is in an exclusive lane. Note that this is different from adding the capacities of component streams (e.g. to calculate the capacity of a lane group, individual lane capacities are summed up).

For uninterrupted (unopposed) streams, the saturation flow is used as the capacity value.

This method calculates the mixed-stream (shared lane) capacity directly without the use of adjustment factors. The method is also applicable to different vehicle types (light and heavy vehicles) in a mixed stream.

3.4 Opposing flows

Two important points in determining the opposing (circulating, or priority) flows are:

(a) the effect of oversaturation of opposing streams, and

(b) the effect of traffic exiting from the same approach road where the opposed stream enters the intersection.

3.4.1 Effect of oversaturation of opposing streams

For any opposing stream lane which is oversaturated (arrival flow exceeds the capacity), the capacity rather than the arrival flow value (Q_i rather than q_i) should be used in the opposed stream capacity model. This *capacity constraint* method should be applied irrespective of the opposing stream being uninterrupted (priority) or opposed (non-priority).

Interactions at a roundabout or other unsignalised intersection are such that the oversaturation of any opposing stream requires the repetition of the capacity estimation process with capacity constraint on oversaturated streams. Thus, capacity calculations for all types of unsignalised intersections need to be iterative.

At roundabouts, capacity constraint on oversaturated approaches would affect the circulating flows, and hence the capacities significantly. This is important not only where flow levels are generally high, but also where there are unbalanced flows resulting in oversaturation of one or more approaches.

Iterative capacity estimation methods for roundabout and other unsignalised intersections can be developed easily making use of existing SIDRA algorithms for saturation flow estimation at signalised intersections.

The *capacity constraint method* needs to be applied to individual lanes of the opposing approaches since a lane can be oversaturated but another lane can be undersaturated due to unequal lane utilisation (see *Section 4*). Furthermore, individual movement (turn, or origin-destination flow at a roundabout) capacities from the lane need to be calculated so as to be used as opposing flow where relevant. The following formulae can be used for this purpose.

For calculating circulating flows at roundabouts, calculate an adjusted value of each *origin-destination flow component* in each lane from:

$$q_j' = \frac{q_j}{x_i} \qquad \text{if } x_i > 1.0$$
$$= q_j \qquad \text{if } x_i \le 1.0 \tag{3.9}$$

where

q_j = arrival flow rate (veh/h) for origin-destination component j in lane i (total lane flow, $q_i = \Sigma q_j$),

q_j' = flow rate adjusted for capacity constraint (veh/h) for origin-destination component j in lane i, and

x_i = degree of saturation of lane i.

For an oversaturated lane i ($x_i > 1.0$), the adjusted value of the total lane flow (q_i') equals lane capacity (Q_i):

$$q_i' = \Sigma q_j' = \Sigma \frac{q_j}{x_i} = \frac{q_i}{x_i} = Q_i \tag{3.10}$$

The total origin-destination flow for each approach road is then calculated as the sum of adjusted values of the relevant origin-destination flow components for all lanes of the approach road. These are the values to be used for calculating circulating flows.

For other unsignalised intersections, apply Eqn 3.9 to calculate adjusted origin-destination flows for each lane of each approach road. To find the opposing flow rate for a given *opposing movement*, add the adjusted component flows in all relevant lanes according to the *opposing flow code* (L, T, R) and the *turn designations* (L, T, R) given to individual origin-destination flows.

3.4.2 Effect of exiting traffic

For roundabouts, both NAASRA (1986) and *SR 45* specify that *exiting* vehicles are not included in circulating flows since field studies indicated that entering drivers were not influenced by exiting traffic (i.e. traffic whose destination number equals the origin number of the entry flow) . However, there may be some cases when the capacity estimation can be improved by including some effect of traffic using the adjacent exit. Allowance will be made in SIDRA for this purpose.

3.5 Heavy vehicle effects

Traditionally, the effect of heavy vehicles on the capacity of an opposed traffic stream has been predicted using a simple method which converts heavy vehicle flows in both the opposed and opposing streams to passenger car equivalents ("passenger car units"). This method may not accurately reflect the behaviour of gap acceptance *by* heavy vehicles and *in front of* heavy vehicles. Troutbeck (1991) investigated the sensitivity of capacity estimates to different *passenger car equivalents* and *gap acceptance factors* for heavy vehicles. He recommended the use of the passenger car equivalents until appropriate values for heavy vehicle gap acceptance parameters are available through field studies.

In SIDRA, both the passenger car equivalent and gap acceptance factor methods can be implemented easily.

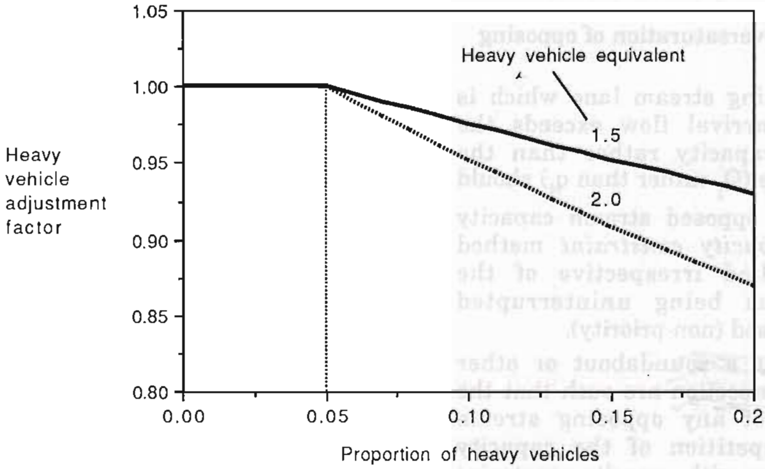

Fig. 7 – The heavy vehicle adjustment factor (f_{HV}) as a function of the proportion of heavy vehicles (p_{HV}) for heavy vehicle equivalents of e_{HV} = 1.5 and 2.0

When different gap acceptance parameters are used to estimate capacities for heavy and light vehicle flows, the mixed stream formula (Eqn 3.8) can be used for calculating the capacity of an entry lane with different light and heavy vehicle flows for each traffic movement (left, through, right) in the lane. This method is preferable to the use of passenger car equivalents, and can be adopted when data becomes available.

The implementation of the passenger car equivalent method in SIDRA for roundabout capacities is explained below. It should be noted that no adjustments are made for heavy vehicles in opposed traffic streams in the case of other unsignalised and signalised intersections. These areas also need to be addressed in future research.

For roundabouts, NAASRA (1986) recommends that flows in units of pcu/h are used instead of veh/h when heavy vehicle flows exceed 5 per cent with heavy vehicle equivalents of 2 for single unit trucks and 3 for articulated vehicles. As discussed in detail by Troutbeck (1991), the *SR 45* models were derived from data which included heavy vehicles in the general driver population. The data included *up to 5 per cent* heavy vehicles. The method given below reflects this fact by using *incremental* corrections for heavy vehicles flows above 5 per cent.

The incremental heavy vehicle adjustment factor is derived as follows. The heavy vehicle equivalent for heavy vehicle flows below 5 per cent ($p_{HV} \le 0.05$) is the same as the light vehicle (car, etc.) equivalent, $e_{HV} = e_{LV} = 1.0$, and for heavy vehicle flows above 5 per cent, the heavy vehicle equivalent is $e_{HV} > 1.0$. Applying the traffic composition factor formula from Akcelik (1990, p. 6) with $e_{LV} = 1.0$, the following incremental heavy vehicle adjustment factor is obtained for the conditions when $p_{HV} > 0.05$:

$$f_{HV} = \frac{1.0}{1.0 + (e_{HV} - 1.0)\,(p_{HV} - 0.05)} \qquad \textit{for } p_{HV} > 0.05$$

$$= 1.0 \qquad \textit{for } p_{HV} \le 0.05 \qquad (3.11)$$

Fig. 7 shows the heavy vehicle adjustment factor (f_{HV}) from Eqn (3.11) as a function of the proportion of heavy vehicles (p_{HV}) for heavy vehicle equivalents of $e_{HV} = 2.0$ and 1.5.

The circulating flow rate should be adjusted for heavy vehicle effects using:

$$q_{ca} = q_c / f_{HV} \qquad (3.12)$$

where q_{ca} is the adjusted circulating flow rate in pcu/h, q_c is the circulating flow rate in veh/h and f_{HV} is the heavy vehicle adjustment factor calculated from Eqn (3.11) using p_{HV} as the proportion of heavy vehicles in the circulating stream.

The adjusted value of the circulating flow should be used in calculating the dominant lane follow-on headway (Eqn 3.1), the mean critical gap (Eqn 3.3), proportion of free vehicles (Eqn 3.5), and entry lane capacity (Eqn 3.6). Note that, since $f_{HV} < 1.0$ for $p_{HV} > 0.05$, an increased circulating flow value will be found ($q_{ca} > q_c$) which will result in a reduced entry lane capacity.

Secondly, the entry lane capacity (Q_e) should be calculated using the adjusted circulating flow from Eqn (3.12). This includes the effect of heavy vehicles in the circulating stream only.

To adjust the entry lane capacity for heavy vehicles in the entry lane, the adjustment factor f_{HV} calculated from Eqn (3.11) with p_{HV} as the proportion of heavy vehicles in the entry lane should be used:

$$Q_{ea} = f_{HV}\ Q_e \qquad (3.13)$$

where Q_{ea} is the adjusted entry lane capacity, and Q_e is the capacity calculated as explained above. Note that both Q_{ea} and Q_e are in units of veh/h.

The factor f_{HV} is used to decrease the entry lane capacity to keep it in units of veh/h, rather than increasing the entry lane flow through a formula similar to Eqn 3.12. The use of flows and capacities in real vehicle units is essential for correct estimation of delays and queue lengths.

4 LANE UTILISATION

An important aspect of the *SR 45* method for estimating the capacity and performance of roundabouts is the designation of entry (approach) lanes as *dominant* and *subdominant* lanes (streams). The traffic in the lane with the largest flow rate is called dominant stream and the traffic streams in other lanes are called subdominant streams.

As discussed in Section 3, the capacity of a subdominant lane is less than the capacity of a dominant lane (except when the follow-on headways are found to be equal, especially in the case of low circulating flow rates and low ratios of entry lane flows). Since the lane capacities and lane flows are interdependent, an iterative method is required to apply the *SR 45* capacity estimation method.

Troutbeck (1991) discussed the issue of lane utilisation at roundabout approaches in some detail providing additional data and and recommending a feasible iterative method for lane flow and capacity estimation. In this section, this issue is further discussed and an adaptation of the general method of lane flow estimation which is used in SIDRA to the roundabout case is described.

The lane flow estimation method used in SIDRA, which is applicable to all types of intersection approaches, allows for equal and unequal lane utilisation (Akcelik 1981, 1990). The method defines a *lane utilisation ratio,* ρ_i for the *ith* entry lane as:

$$\rho_i = x_i / x_c \tag{4.1}$$

where x_i is the degree of saturation of the *ith* lane and x_c is the degree of saturation of the *critical lane* (x_c is the largest degree of saturation for any approach lane). The degree of saturation for the *ith* lane is defined as $x_i = q_i / Q_i$ where q_i and Q_i are the arrival (demand) flow and capacity.

For the lanes which have $x_i = x_c$, the utilisation ratio is $\rho_i = 1.0$ (100 % utilisation). In the case of equal lane utilisation, all entry (approach) lanes have $\rho_i = 1.0$. An underutilised lane has $\rho_i < 1.0$ since $x_i < x_c$. Thus, the lane utilisation ratio must be in the range $0 < \rho_i < 1.00$. In SIDRA, lane utilisation ratios are specified by the user, or are determined by the program in the case of de facto exclusive lanes.

The general method to calculate lane flows (q_i) which yield a specified set of lane utilisation ratios (ρ_i) using known lane capacities (Q_i) and the total entry (approach) flow (q_e) is as follows:

(i) Calculate an approach degree of saturation which equals the critical lane degree of saturation:

$$x_c = \frac{q_e}{\Sigma \rho_i Q_i} \tag{4.2}$$

where $q_e = \Sigma q_i$, and the summations are for lanes i = 1 to n_e (n_e = number of entry lanes).

(ii) Calculate the flow for the *ith* lane from:

$$q_i = x_c \rho_i Q_i \tag{4.3}$$

In SIDRA, the above method is applied to each set of exclusive turn lanes in the approach separately. In the case of shared lanes, the lane flows from Eqn (4.3) are compared with turning flows (Left, Through and Right) to establish any cases of de facto exclusive lanes. Any excess flows from short lanes are allocated to adjacent lanes before lane flow calculations are made.

The SIDRA method of lane flow estimation can be applied to the *SR 45* method for roundabouts directly. It is important to note that the dominant and subdominant lanes do not necessarily coincide with the critical and non-critical (underutilised) lanes since their definition is based purely on relative flow rates. Thus, for dominant and subdominant lanes, $x_d = q_d / Q_d$ and $x_s = q_s / Q_s$. If the dominant lane is critical ($x_d > x_s$), then the utilisation ratios are $\rho_d = 1.0$ and $\rho_s = x_s / x_d < 1.0$. It is possible for the the subdominant lane to be critical ($x_s > x_d$), in which case the utilisation ratios are $\rho_s = 1.0$ and $\rho_d = x_d / x_s < 1.0$.

When the same ρ_s value is used for all subdominant lanes, the critical (approach) degree of saturation from Eqn (4.2) becomes:

$$x_c = \frac{q_e}{\rho_d\, Q_d + (n_e - 1)\, \rho_s\, Q_s} \tag{4.4}$$

where n_e is the total number of entry lanes (this is based on the use of only one dominant lane as real-life data indicates).

From Eqn (4.3), the dominant and subdominant lane flows are given as:

$$q_d = x_c\, \rho_d\, Q_d \tag{4.5a}$$

$$q_s = x_c\, \rho_s\, Q_s \tag{4.5b}$$

The corresponding ratio of entry lane flows is:

$$r_{ds} = \frac{q_d}{q_s} = \frac{\rho_d\, Q_d}{\rho_s\, Q_s} \tag{4.6}$$

For an *equal degree of saturation* solution, $x_s = x_d$ and $\rho_s = \rho_d = 1.0$. The following equations can be used in this case:

$$x_c = \frac{q_e}{Q_d + (n_e - 1)\, Q_s} \tag{4.7a}$$

$$q_d = x_c\, Q_d \qquad q_s = x_c\, Q_s \tag{4.7b}$$

$$r_{ds} = \frac{q_d}{q_s} = \frac{Q_d}{Q_s} \tag{4.7c}$$

The value of r_{ds} from Eqn (4.6) or (4.7a to c) is likely to differ from the initial value of r_{ds} used to determine the subdominant lane capacity. Therefore, several iterations are required to find lane flows, capacities, r_{ds} and ρ_s values that match.

To reflect the current SIDRA algorithms, the following iterative method for lane flow and capacity calculations can be described for roundabouts:

Step 1. Calculate the dominant and subdominant lane capacities assuming $r_{ds} = 1.0$ (i.e. equal lane flows). SIDRA equates lane capacities to basic saturation flows for initial estimates of lane flows. Thus, this could lead to unequal lane flows when basic saturation flows differ. In the case of low circulating flow rates, equal lane flows could result in equal lane capacities and therefore the results would be final at this stage. Otherwise, carry out iterations as described in *Steps 2* and *3*.

Step 2. Using the capacities calculated in the previous step, calculate lane flows and the corresponding r_{ds} value using Eqns (4.4) to (4.7c).

Step 3. Calculate new subdominant lane follow-on headway and critical gap values and use these for a new capacity, $Q_{s(j)}$ (dominant lane capacities do not change). Compare this with the corresponding value calculated in the previous step, $Q_{s(j-1)}$. Stop iterations if the difference does not exceed 10 veh/h:

$\Delta Q_s =$
$abs\,[Q_{s(j)} - Q_{s(j-1)}] \le 10$ veh/h;

Otherwise continue from *Step 2* (in SIDRA, the maximum number of iterations is currently three). For the final iteration, calculate the new lane flows (needed for performance calculations).

An important requirement in implementing the *SR 45* method is the choice of dominant and subdominant lane utilisation ratios (ρ_d , ρ_s). Generally, an equal degree of saturation solution is acceptable ($\rho_d = \rho_s = 1.0$).

For SIDRA, the lane with the heaviest turning traffic volume should be designated as the dominant lane. Each set of exclusive lanes may be treated separately in terms of designating dominant and subdominant lanes. Lane utilisation ratio and dominant/ subdominant lane specifications by the user are subject to program override in the cases of de facto exclusive lanes. For example, a lane with a heavy right-turn volume which has been specified as a subdominant lane can be changed to a dominant lane if the program finds that all other flows (with a total less than the right-turn flow) are assigned to other lanes.

5 THE DELAY MODEL FOR OPPOSED TRAFFIC

Formulation of a steady-state delay model for roundabouts and other unsignalised intersections has been discussed extensively by Troutbeck (1986, 1988a,b, 1989, 1990b). This steady-state delay model can be expressed as follows (d_{st} in seconds):

$$d_{st} = d_m (1 + \gamma) + \frac{3600\, k\, x}{Q_e (1 - x)} \qquad (5.1)$$

where k is a delay parameter given by

$$k \;=\; d_m (\gamma + \varepsilon)\, Q_e / 3600 \qquad (5.2)$$

x = degree of saturation of the entry lane,

Q_e = entry lane capacity (veh/h),

γ, ε = parameters which depend on gap acceptance characteristics (these may be used for calibrating the delay model); for most practical purposes $\gamma = 0$ and $\varepsilon = 1.0$ are acceptable, and therefore:

$$k \;=\; d_m\, Q_e / 3600 \qquad (5.2a)$$

d_m = minimum delay experienced by the entering (opposed) traffic (in seconds):

$$d_m = \frac{e^{\lambda(\alpha - \Delta)}}{\varphi\, q_c} - \alpha - \frac{1}{\lambda} + \frac{\lambda\Delta^2 - 2\Delta + 2\Delta\varphi}{2\,(\lambda\Delta + \varphi)} \qquad (5.3)$$

(applicable for $q_c < 1/\Delta$)

where the gap acceptance parameters α, β, Δ, φ and λ are as defined in *Section 3* and the circulating flow, q_c is in veh/sec.

The time-dependent delay formula which corresponds to the steady-state delay given by Eqn (5.1) can be expressed as:

$$d = d_m (1 + \gamma) + 900\, T \left[(x - 1) + \sqrt{(x - 1)^2 + \frac{8k\, x}{Q_e\, T}} \right] \qquad (5.4)$$

T = duration of the flow period in hours,

x = degree of saturation in the specified flow period, and

Q_e is in veh/h, d_m and k are as given by Eqns (5.2 and (5.3).

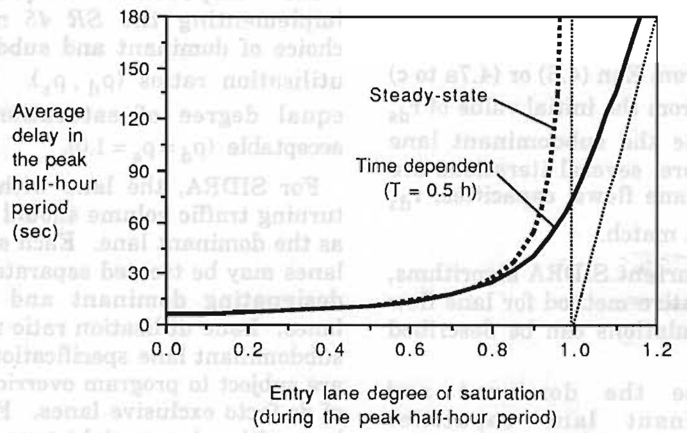

Fig. 8 – Steady-state and time-dependent delays as a function of the entry lane degree of saturation for the example in Fig. 5.1 (circulating flow, q_c = 900 veh / h, entry lane capacity, Q_e = 606 veh / h, k = 0.906)

In both the steady-state and the time-dependent formulations, the second term accounts for the queueing delays due to the presence of a queue in the entry lane.

The delay formulation presented here is consistent with the general delay formulation used in SIDRA (Akcelik 1990) with parameters $m = 8k$, $x_o = 0$ and $n = 0$.

The delays predicted by the steady-state formula and the time-dependent formula for a single-lane roundabout case where $D_i = 30$ m, $n_e = n_c = 1$, $w_L = 4.0$ m (therefore $\Delta = 2$ sec) are shown in Fig. 8. The time-dependent delays are based on a peak flow period of 0.5 hours. In this period, the circulating flow is 900 veh/h and the corresponding capacity is 606 veh/h. Therefore, the delay parameter is $k = 0.906$. Entry lane flow rate is varied to calculate the degrees of saturation and the corresponding delays.

An alternative delay formulation which is based on traditional queueing theory assuming random arrivals and random service rates is widely used. The steady-state and time-dependent delay expressions given here (Eqns 5.1 and 5.4) are also applicable to this formulation. For this purpose, a different minimum delay expression is used:

$$d_m = 3600 / Q_e \tag{5.5}$$

Therefore, the delay parameter (from Eqn 5.2a) is constant:

$$k = 1.0 \tag{5.6}$$

6 VARIABLE-DEMAND CONSIDERATIONS

In the derivation of the time-dependent delay formula given in *Section 5.1* (Eqn 5.4), a zero initial queue and a constant demand rate throughout the flow period T is assumed. When there is significant peaking during the flow period, this formula may underestimate delay for high degrees of saturation. This point has been emphasised in papers by Burrow (1989) and Brilon and Wu (1990) in relation to the signalised intersection delay models.

However, recent work reported by Akcelik and Rouphail (1991) and Rouphail and Akcelik (1991) indicates that a single-period analysis with a constant demand rate is adequate provided that it is applied to a peak flow period which is determined with due attention to the peaking profile in the total flow period.

Akcelik and Rouphail (1991) described a simple variable-demand model which uses a generalisation of the well-known Peak Hour Factor (PHF) parameter (defined as the ratio of the average flow rate in the total flow period to the average flow rate in the peak flow period). This model divides the total flow period (e.g. T = 1 h) to peak and non-peak periods (e.g. $T_p = 0.5$ h and $T - T_p = 0.5$ h) with constant flow rates, i.e. a step demand function is used.

Using this model, the general delay formula (Eqn 5.4) can be applied to peak and non-peak periods as well as the post-peak oversaturation period when the peak period degree of saturation is greater than 1.0. The delay estimate for the total period can than be calculated as a flow-weighted average of delays for the peak, non-peak and post-peak oversaturation periods. However, it was found that the average delay in the peak flow period (e.g. $T_p = 0.5$ h, or in the case of U.S. Highway Capacity Manual, $T_p = 0.25$ h) was a good approximation to the average delay calculated by such a complex process. This simplifies the delay estimation process considerably.

In the example given in Fig. 9, the roundabout characteristics are as in Fig 8 where flow conditions are considered to represent the peak half hour period ($T_p = 0.5$ h). A Peak Hour Factor of PHF = 0.889 is applied to both the entering and circulating flows (various parameters were derived assuming a triangular demand pattern). The circulating flow, capacity, minimum delay and delay parameter values for the peak, non-peak and total flow periods are given in Table 3.

Fig. 9 shows three different delay estimates:

(i) Average delay (d_p) for the peak flow period calculated from Eqn (5.4) using the degree of saturation (x_p) for the *peak* flow period of duration $T = T_p$ = 0.5 h. This delay is identical to the time-dependent delay given in Fig. 8. However, the degrees of saturation shown in Figs 8 and 9 are different.

(ii) Average delay (d_a) for the total flow period calculated as a flow-weighted average of delays in the peak, non-peak and post-peak oversaturation periods. The method for calculating this delay is described in Akcelik and Rouphail (1991).

(iii) Average delay (d_a') for the total flow period (T = 1 h) calculated from Eqn (5.4) using the average degree of saturation (x_a) for the total flow period .

It is seen that d_a' underestimates delays for the total period (compared to d_a) for high degrees of saturation particularly because it does not account for oversaturation during the peak flow period. It is also seen that d_p is larger than d_a. Considering extra transitional delays between peak, non-peak and post-peak periods, d_p is probably a good representative of the average delay in the total flow period.

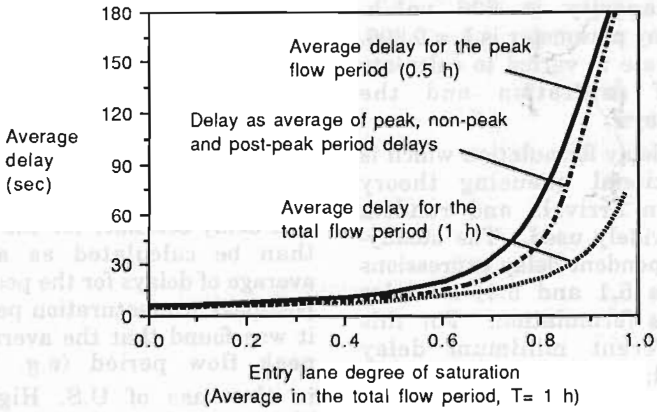

Fig. 9 – The average delay for the peak flow period, total flow period using the average of peak, post-peak and non-peak delays, and using the average degree of saturation in the total flow period

Table 3. The circulating flow, capacity, minimum delay and delay parameter values for the peak, non-peak and total flow periods for the example shown in Fig. 9

Flow Period	Circulating Flow q_c (veh/h)	Entry Capacity Q_e (veh/h)	Minimum Delay d_m (sec)	Delay Parameter k
Total (T = 1.0 h)	800	663	4.48	0.825
Peak (T_p = 0.5 h)	900	606	5.38	0.906
Non-peak ($T - T_p$ = 0.5 h)	700	721	3.71	0.743

Thus, the roundabout delay results shown in Fig. 9 confirm the findings of Akcelik and Rouphail (1991) for signalised intersections. Therefore, the adoption of an analysis period which reflects a peak flow period of duration 15 to 30 minutes is recommended for single period analysis using Eqn (5.4).

7 CONCLUSION

Implementation of a comprehensive roundabout (and other unsignalised intersection) analysis method in the well-established SIDRA package (now in use by over 150 organisations/sites in 24 countries) will provide a comprehensive intersection design and evaluation tool useful for practitioners as well as researchers.

The use of an improved gap-acceptance-based capacity model, a time-dependent delay model which handles the question of demand peaking in a simple way, and a lane-by-lane analysis method will be the particular strengths of the methodology offered by the SIDRA package. This will be complemented by a user-friendly and powerful input data editor which makes extensive use of picture facilities to help the design process.

Further work is in progress to develop formulae for estimating queue length and stop rates for roundabouts and other unsignalised intersections.

NOTATION

D_c Central island diameter (metres)

D_i Inscribed diameter ($D_i = D_c + w_c$) (metres)

w_c Circulating road width (metres)

w_L Average entry lane width (metres)

n_c Number of circulating lanes

n_e Number of entry (approach) lanes (including any short lanes)

Q_e Capacity of entry (opposed) stream

q_e Flow rate of entry (opposed) stream

q_c Circulating (opposing) flow rate

q_d Dominant entry lane flow rate

q_s Subdominant entry lane flow rate

r_{ds} Ratio of entry lane flows (the ratio of dominant to subdominant stream flow rate, $r_{ds} = q_d/q_s$)

α Mean critical gap in the opposing stream (seconds)

β Follow-on time (headway) of the entry stream (seconds)

β_d Dominant entry stream (lane) follow-on time (seconds)

β_s Sub-dominant entry stream (lane) follow-on time (seconds)

β_o Follow-on time when the circulating flow rate is zero ($\beta_o = \beta$ for constant follow-on times)

Q_m Maximum capacity of the opposed stream ($Q_m = 3600/\beta$)

Δ Intra-bunch (minimum) headway in the opposing traffic stream

φ Proportion of free vehicles in the opposing traffic stream

λ, Parameter in the capacity equation

d_m Minimum delay experienced by an entering vehicle at near zero entry flow conditions (seconds)

d Average delay per vehicle (seconds)

x Degree of saturation (flow/capacity ratio) (called utilisation ratio in queueing theory – not to be confused with the lane utilisation ratio, ρ)

ρ Lane utilisation ratio defined as the ratio of the degrees of saturation of a given lane and the dominant (critical) lane

(Flow rates and capacities are in units of *vehicles per hour* or *vehicles per second.*)

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