

Roundabout Model Calibration Issues and a Case Study

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ABSTRACT

This paper discusses issues related to calibration of models for analyzing roundabout capacity and performance. A traffic model framework is presented to help with assessment of traffic models in a general framework, considering all aspects of models relevant to roundabout operation. While the discussion focuses on analytical models, the issues raised are also relevant to microsimulation models. Discussion on roundabout models should not concentrate on capacity alone, and instead, modeling requirements for estimating both capacity and performance (delay, queue length, etc) should be considered together. Various aspects of field observations relevant to the calibration effort are discussed. These include issues related to the definition and measurement of capacity, delay and queue length, including the effect of unequal lane utilization. Delay criteria for level of service definition are also discussed. Two basic calibration methods that can be used for gap-acceptance and linear regression methods are described. Further aspects of model calibration discussed include the environment factor, adjustment for the arrival flow / circulating flow ratio, lane utilization factor, heavy vehicle factor, driver response time and calibration of models for operating cost, emissions and fuel consumption. A case study is presented to compare capacity estimates from the gap-acceptance and linear-regression methods, including a calibration example.

INTRODUCTION

Effective use of models to analyze intersection capacity, performance and level of service may require significant calibration effort. The Highway Capacity Manual (1) defines calibration as "*The process of comparing model parameters with real-world data to ensure that the model realistically represents the traffic environment. The objective is to minimize the discrepancy between model results and measurements or observations.*" The nature of the model in use determines the calibration effort. Therefore, a good understanding of the basic premises of the model is an essential step in model calibration. A model framework for identifying basic characteristics of road traffic models is presented for this purpose. This framework and many aspects of the discussion presented in this paper are applicable to roundabout and other intersection models as well as all traffic facilities generally.

The key input and output parameters used by a model for representing the *real world* need to be identified for calibration purposes since resources available for model calibration are usually limited. Capacity, performance (delay, queue length, etc) and level of service characteristics of a traffic facility are the key output parameters which are of interest to the practitioner.

Intersection capacity and performance characteristics are influenced by both local *driver behavior* and particular intersection conditions including *intersection geometry* and *traffic control*. The driver behavior is related to local road, driver and vehicle characteristics. Therefore, the model input parameters representing driver behavior, vehicle characteristics, the intersection geometry and control need to be identified for the purpose of calibration.

To a great extent, all model input parameters (and other parameters that are not available as input parameters but are accessible as default parameters) related to intersection geometry and driver behavior are important for calibrating the traffic model to represent particular intersection conditions. For roundabouts and other unsignalised intersections, gap-acceptance parameters (especially follow-up headway and critical gap) are the key parameters representing driver behavior. The overall roundabout geometry (configuration of approach roads, number of approach and circulating road lanes, allocation of lanes to movements) affects the capacity and performance directly. The gap-acceptance parameters as well as the approach and circulating road lane use are affected by the roundabout geometry as well as the overall demand flow levels and patterns, which in turn affect the capacity and performance significantly. The calibration issues related to these parameters are discussed in some detail.

Model calibration effort may involve comparison of traffic models offered by different software packages, for example an analytical and a simulation model. Various issues need to be recognized in relation to this. In particular, users of microsimulation models should not assume that a more detailed model will necessarily result in reduced model error (2). This is because, it is likely that, while model specification error will decrease with increased model detail (complexity), the total measurement error will increase due to the increased model complexity (more variables each with an associated degree of measurement error). This consideration applies to all models, simulation or analytical.

Also importantly, the consistency of definitions and measurement methods for traffic performance variables such as delay (stopped, geometric, etc) and queue length (cycle average, back of queue, etc) must be ensured in comparing models as well as in comparing model estimates with values observed in the real world.

A case study is presented for comparing capacity estimates from the gap-acceptance based Australian (aaSIDRA) (3-14) and the linear-regression based UK (TRL) methods (15-17). A calibration example is given for the case study.

aaSIDRA used in the analyses reported in this paper employs an *empirical* gap-acceptance method to model roundabout capacity and performance. The aaSIDRA model allows for the effects of both roundabout *geometry* and *driver behavior*, and it incorporates effects of *priority reversal* (low critical gaps at high circulating flows), *priority emphasis* (unbalanced O-D patterns), and *unequal lane use* (both approach and circulating lanes). Capacity can be measured as a *service rate* for each traffic lane in undersaturated conditions (v/c ratios less than 1) according to the HCM definition of capacity to represent *prevailing conditions*. This is in contrast with measuring *approach capacity* in oversaturated conditions. See the Appendix for graphs and a table summarizing the survey data, indicating the empirical nature of the aaSIDRA gap-acceptance model for roundabouts.

MODEL FRAMEWORK

A general framework for classification of road traffic models is presented in *Figure 1* with the purpose of identifying basic characteristics of road traffic models. This two-dimensional framework considers the nature and level of detail offered by a traffic model in handling road geometry and traffic elements. The focus is on the movement of vehicle traffic.

Contrasting models as macroscopic vs microsimulation, deterministic vs microsimulation, empirical vs theoretical, empirical vs analytical, etc are not valid ways of qualifying models. Models do not fall into clear-cut categories, but there is a spectrum (continuum) of models. The framework presented here may be helpful to understand that analytical models or simulation models can be microscopic or macroscopic (and in between).

The US Highway Capacity Manual (HCM) defines an analytical model as "A model that relates system components using theoretical considerations that are tempered, validated, and calibrated by field data.", whereas it defines a simulation model as "A computer program that uses mathematical models to conduct experiments with traffic events on a transportation facility or system over extended periods of time." HCM Glossary (Chapter 5) gives various other definitions related to models and HCM Chapter 31 presents a useful discussion of different types of models (1).

For the purpose of the framework presented in *Figure 1*, analytical models are defined as those that use direct mathematical computations to determine system states, and simulation models as those that use various rules (mostly in the form of mathematical equations) for movement of vehicles in a system (individually or in platoons). The HCM models are analytical. HCM Chapter 31 states that "*The HCM methods represent traffic flows with variables that reflect flow dynamics. These methods stop short of representing the movements of individual vehicles. The intent is to employ calculations that can be done by hand, using a set of worksheets, or by computer ...*".

| | | ROAD GEOMETRY ELEMENTS | | |
|------------------|---------------------|--------------------------------------|---------------------------------|-----------------------------|
| | | APPROACHES (all lanes aggregated) | LANE GROUPS (2) (or "links") | LANES (or lane segments) |
| TRAFFIC ELEMENTS | Individual vehicles | | | Micro-simulation |
| | Platoons | Macro-simulation | | Meso-simulation |
| | Drive cycles (1) | | | Micro-analytical |
| | Traffic flows | Macro-analytical | Meso-analytical | |
| | Speed-flow models | Macro-analytical | | |

A classification of various software packages according to this framework is presented in www.aattraffic.com/TrafficModels.htm

- (1) Drive cycle may be defined as the vehicle speed-time trace consisting of acceleration, deceleration, cruise and idling elements.
- (2) Lane Group is a set of lanes with one or two shared lanes (e.g. Lane 1: Left-Turn and Through, Lane 2: Through) or a set of exclusive turn lanes (e.g. a single Right-Turn lane).

Figure 1 - A general framework for road traffic models

In the framework presented in *Figure 1*:

- (i) a simulation model can be microscopic, macroscopic or mesoscopic,
- (ii) an analytical model can be microscopic, macroscopic or mesoscopic, and
- (iii) a simulation model can be deterministic or stochastic.

Analytical traffic models such as aaSIDRA (9) usually incorporate stochastic elements (e.g. overflow queue models for traffic at intersections) although each application of the model may produce the same outcome (deterministic). The distinction "stochastic vs deterministic" does not necessarily imply model quality since it is possible to randomize parameters of traffic elements at every level of detail (individual vehicle, platoon, traffic flow, etc).

Contrasting models as "empirical vs theoretical" (as frequently done in the literature in relation to roundabout capacity models) represents a simplistic view since most models have basis in traffic behavior theory and are empirical at the same time. However, the term "empirical model" is usually used to mean "based on statistical analysis of field data without any direct basis in traffic theory".

The framework presented *Figure 1* is limited to vehicle traffic. The issues of different vehicle types and driver types, and the size of the area modeled (single intersection, arterial, network, etc) are further considerations in this context. Different modes of traffic (pedestrians, cyclists, public transport) could be added as a third dimension to this framework, each with its own special considerations. For example, for pedestrians, drive cycles are not applicable, and pathways rather than lanes would be relevant.

FIELD OBSERVATIONS

In model calibration, capacity and performance estimates from a model and those observed in the field are compared, and appropriate parameters in the model are modified in order to match the real-world observations. In this effort, consistency of definitions and measurement methods for traffic performance variables such as delay and queue length must be ensured in comparing model estimates with observed values. The consistency of definitions is also important in comparing estimates of

performance measures from different models. Various comments on capacity, delay and queue length measurements are given below.

It is important that all relevant intersection data used in model calibration should be collected at the same time. For roundabouts, demand volumes on all approaches and the required capacity and performance measures (delay, queue length) need to be observed at the same time due the interactive nature of roundabout operations.

Capacity

Capacity is the main determinant of performance measures such as delay, queue length and stop rate. The relationship between a given performance measure and capacity is often expressed in terms of degree of saturation (demand volume - capacity ratio). Capacity is the maximum sustainable flow rate that can be achieved during a specified time period under prevailing road, traffic and control conditions. The proviso "prevailing conditions" is important since capacity is not a constant value, but varies as a function of traffic flow levels. Capacity represents the *service rate* (queue clearance rate) in the performance (delay, queue length, stop rate) functions, and therefore is relevant to both undersaturated and oversaturated conditions. Conceptually, this is different from the *maximum volume that the intersection can handle* which is the *practical capacity* (based on a target degree of saturation) under increased demand volumes, not the capacity under prevailing conditions.

Important issues in modeling capacity, relevant to all types of intersection, are:

- (i) the level of aggregation in terms of individual lanes, lane groups and approaches, and
- (ii) the method of measuring capacity.

Different methods have been used to measure and model capacities in terms of *level of aggregation* (see *Figure 1*). These are:

- (i) *lane-by-lane* analysis as in aaSIDRA (9,19),
- (ii) analysis by *lane groups* (movements combined according to shared lanes) as in the US Highway Capacity Manual (1), and
- (iii) analysis by total *approach flows*, i.e. all movements in all approach lanes aggregated, as in the UK method for roundabout capacity analysis (15-17,20).

The lane-by-lane method introduces improved accuracy levels in capacity and performance prediction by allowing improved spatial (geometric) modeling of all types of intersection. It also simplifies the analysis method as it eliminates various complicated formulations resulting from the need to aggregate several lanes with different characteristics into a single lane group that represents all lanes.

Unequal lane utilization (even where shared lane use is allowed) is an important factor that affects the capacity and performance of roundabouts. Lane under-utilization at a roundabout may be due to:

- exclusive lanes as determined by lane marking and signing,
- path overlap on the circulating road due to poor roundabout design, and lack of circulating lane markings,
- a lane that discontinues at the downstream side due to a decreased number of lanes or parked vehicles (downstream short lane),
- a lane with a large proportion of traffic turning left or right at a downstream location (destination effect),
- some interference at the downstream side, e.g. vehicles merging from a slip lane with no clear give-way (yield) lane markings,
- a large number of heavy commercial vehicles or buses (moving or stopping) in the lane,
- turning vehicles in the lane subject to heavy pedestrian conflict at the exit,
- heavy interference by parking maneuvers (parking adjacent to the lane), or
- a short lane (e.g. a turn bay, or a limited queuing space due to parking upstream).

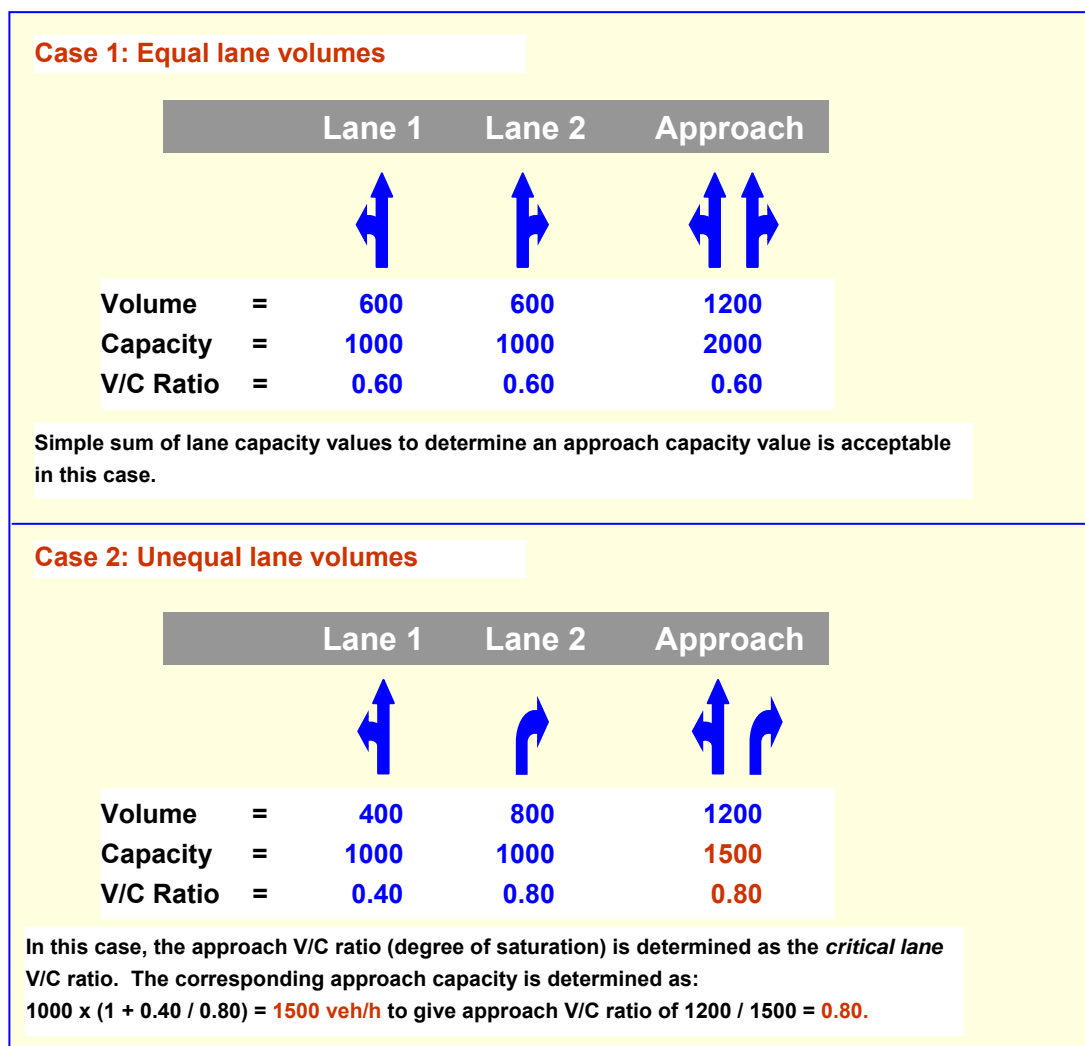


Figure 2 - An example of lane capacity and approach capacity values with equal and unequal lane volumes (equal lane capacity values assumed for simplicity)

Importantly, a simple sum of lane capacity values calculated as the lane group or approach capacity is misleading in the case of lane underutilization since such an aggregate capacity value does not reflect the critical lane volume - capacity ratio, and therefore may underestimate delays and queue lengths significantly. A simple example is given in *Figure 2*. An indirect way of modeling the effect of unequal lane use on lane group and approach capacity is possible (as shown for Case 2 in *Figure 2*) but better modeling is achieved by lane-by-lane modeling of roundabout capacity and performance, including improved modeling of circulating lane use at multilane roundabouts. For detailed discussions on possible inaccuracies that may result by inadequate modeling of approach lane utilization, refer to Chard (20), Akçelik (9,19). Also see the case study presented at the end of this paper.

Two distinct methods of *measuring capacities* at real-life intersections are available:

- (i) measuring departure (saturation) flow rates during saturated (queued) portions of individual green periods at signals or unblocked periods of gap-acceptance cycles at unsignalised intersections (including roundabouts), and the associated proportion of time available for queue discharge, and

- (ii) measuring departure flow rates (volume counts) at the stop or give-way (yield) line under continuous queuing (saturated) conditions over sufficiently long observation periods.

Method (i) is used commonly for signalized intersections, e.g. the method given in HCM Chapter 16, Appendix H (1) and the method given in ARR 123, Appendix E (18). This method is more difficult to implement for unsignalised intersections due to the short duration of gap-acceptance cycles. Method (ii) based on continuous queuing is therefore more common for unsignalised intersections.

In oversaturated conditions, i.e. when the arrival (demand) flow rate exceeds the departure flow rate persistently during an observation period, capacity can be obtained from a simple volume count (as the departure flow rate). In such conditions, the demand flow rate can be measured by counting the number of vehicles arriving at the back of queue, not at the stop or give-way (yield) line. While the demand flow rate is needed for better estimation of delay and queue length, the capacity observation provides a simple method for calibrating the saturation flow rate or follow-up headway and critical gap.

The gap-acceptance method uses the follow-up headway (β) as the queue discharge (saturation) headway. This corresponds to a saturation flow rate of $s = 3600 / \beta$. For example, $\beta = 2.5$ seconds implies $s = 1440$ veh/h. This is the maximum capacity that can be achieved when the opposing flow is close to zero. The capacity is reduced from this value with increased opposing flow rates, due to decreased proportion of acceptable gaps (equivalent to the green time ratio for signals). Australian research on roundabouts has shown that the follow-up headway is reduced with increasing circulating flow rates.

The capacity as a maximum departure flow rate is achieved when there is a continuous queue during acceptable gaps at unsignalised intersections, i.e. under high demand conditions. On the other hand, capacity is defined as the maximum flow rate "under prevailing roadway, traffic, and control conditions" (TRB 2000, Chapter 2). This may appear to present a *conflict* in capacity definition in that prevailing conditions may be undersaturated (low to medium demand conditions).

Capacity estimation should be based on the use of current demand rates for all movements in a way consistent with requirements of performance estimation that the capacity is used as the *service rate* under prevailing conditions. The method of increasing the demand flow rate for the subject approach (or movement) to create oversaturated conditions in order to measure capacity has been suggested in the literature as a way of obtaining capacity estimates from *microsimulation* models. This method is problematic when applied to a *closed (interactive) system* such as a roundabout: if the demand flow of the subject approach is increased to obtain oversaturated conditions, a reduced entry capacity would result for downstream approaches, which would in turn impact the subject approach, and the resulting capacity estimate would not be consistent with prevailing conditions required for performance estimation.

The capacity for prevailing conditions should not be confused with how much demand a facility can handle. The capacity estimated using current traffic demand levels may be misleading since the traffic growth implied by this capacity is likely to result in unacceptable levels of service. In other words, capacity will be less under future (increased) demand scenarios. This is because the capacity depends on opposing flow levels, especially for roundabouts and sign-controlled intersections. Short lane and blocked lane capacities also tend to drop with increased flow levels. Furthermore, the amount of traffic a facility can handle is determined on the basis of a practical (target) degree of saturation or an acceptable level of service limit, rather than actual capacity values.

While high levels of capacity can be obtained at roundabouts under low to medium flow conditions, some significant capacity reduction occurs at *high demand levels* especially at multi-lane roundabouts with *unbalanced* flow patterns. This is confirmed by real-life observations (4,5).

In addition to the circulating flow, the *exiting flow* may also affect roundabout entry capacity under certain circumstances although Australian research determined that exiting flows did not affect the entering traffic in general terms (default value = 0 %). This may occur when the entering drivers are

not able to determine if the conflicting vehicles on the circulating road are going to exit the roundabout or continue to travel in front of them, or are simply hesitant to enter the roundabout when there is a vehicle on the circulating road. A percentage of the exiting flow may be added to the circulating flow in such cases.

Delay

The following should be noted about delay estimates from a roundabout model in comparing field observations with model delay estimates (also comparing delay estimates from different models).

It should be clearly identified if the model estimates the *control delay* that includes both queuing delay and geometric delay, or the *stop-line delay* that excludes geometric delay. Control delay can be considered to be the overall time loss that includes all delays experienced in traveling through an intersection with reference to approach and exit cruise speeds (including all acceleration and deceleration delays, delay due to cruise at a lower speed, and stopped delay). Geometric delay is the delay experienced by a vehicle going through (negotiating) the intersection in the absence of any other vehicles. Various delay definitions are shown in *Figure 3*.

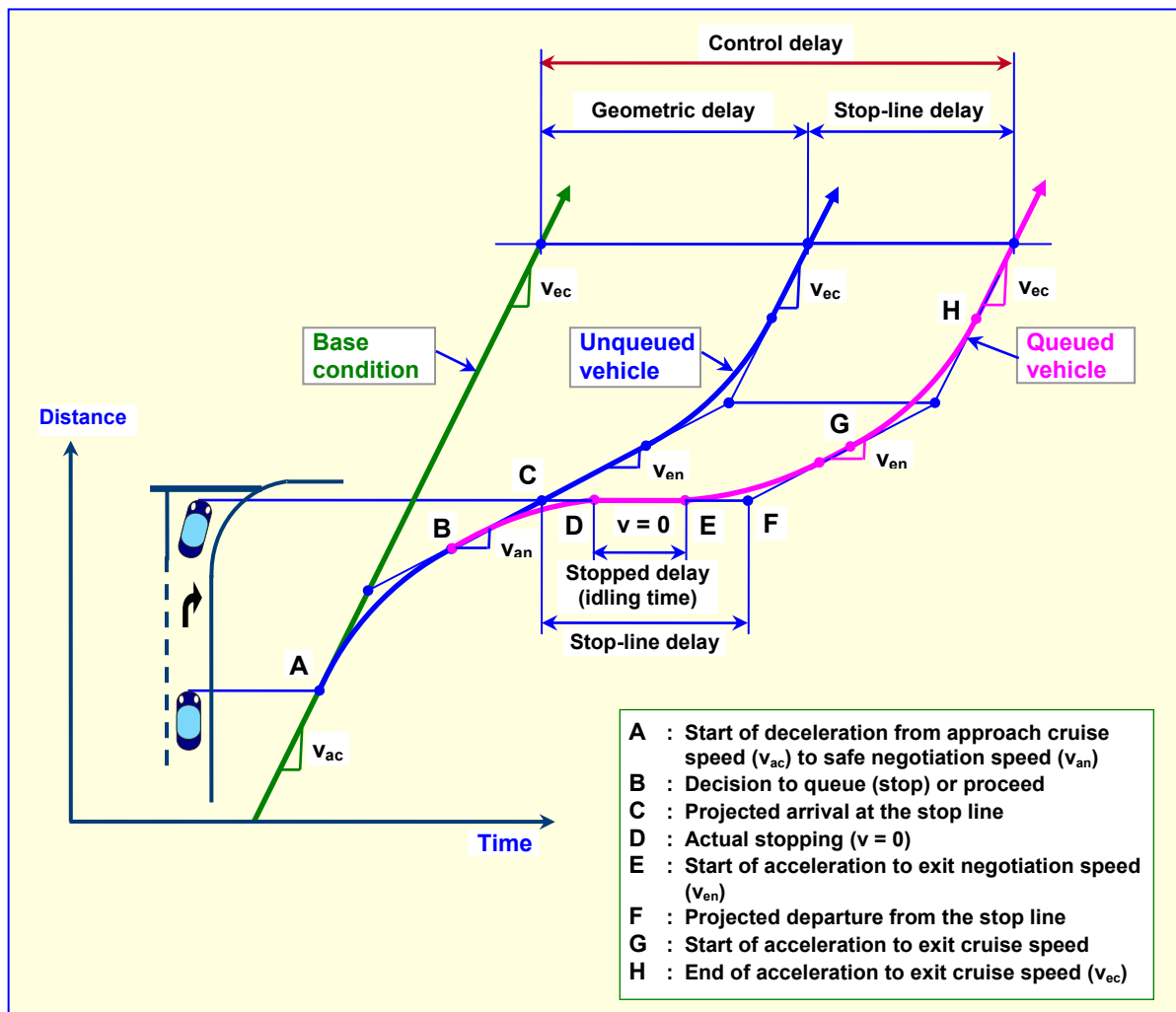


Figure 3 - Definition of control delay, geometric delay, stop-line delay, and stopped delay experienced by a turning vehicle at an intersection (case when the approach and exit cruise speeds are the same, $v_{ac} = v_{ec}$, and the approach and exit negotiation speeds are the same, $v_{an} = v_{en}$)

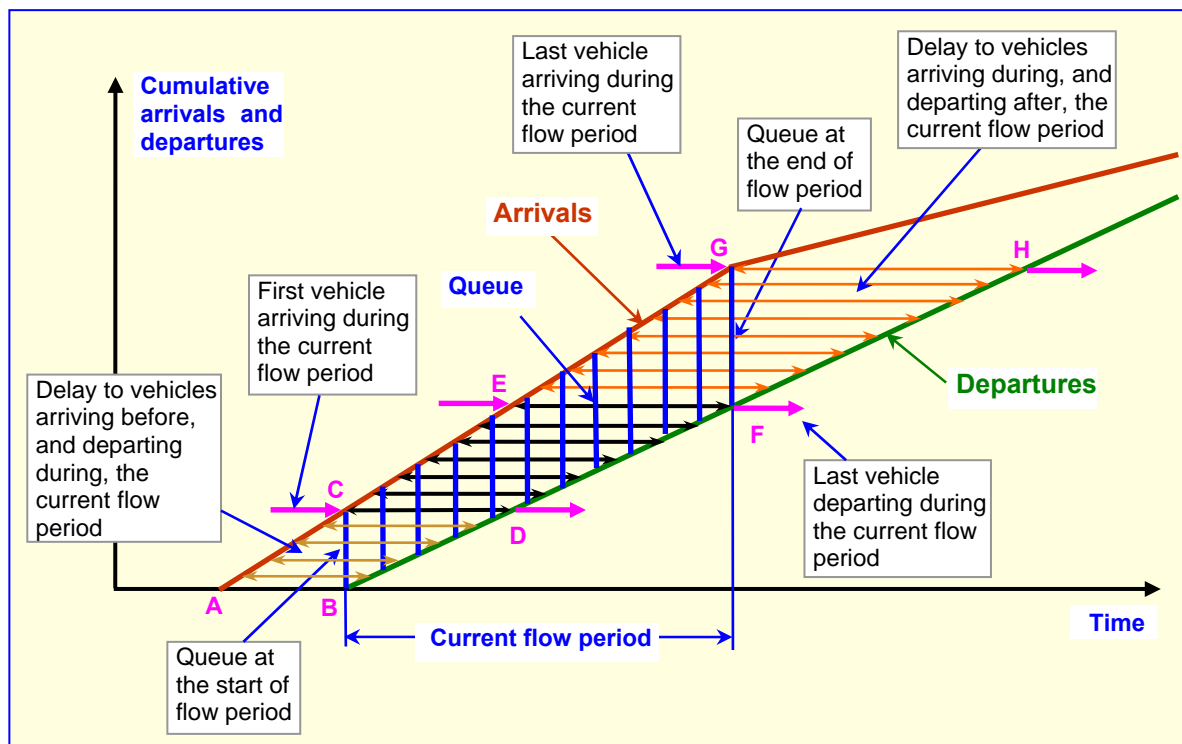


Figure 4 - Delays experienced by vehicles in oversaturated conditions

Delay estimated by traffic models is usually the *average* for all vehicles, queued and unqueued. At roundabouts, all unqueued vehicles experience the geometric delay. Unqueued through movements at signals (arriving and departing during the green period) experience zero delay. These are taken into account in the average delay to all vehicles.

For high volume-capacity ratios (especially for oversaturated traffic conditions), it is important to identify if the average delay is for vehicles *arriving* during a given flow period (observation period) and includes the delay experienced after the end of the flow period (and does not include the delay experienced by vehicles that arrived before, and departed during, the current observation period). *Figure 4* shows the delays experienced by individual vehicles (horizontal lines) and the queue counts (vertical lines) for a deterministic oversaturation model (given here to explain the concepts involved). The delay experienced by vehicles *arriving* during a given flow period corresponds to the *path-trace* (instrumented car) method of measuring delays.

An alternative delay survey method is the *queue-sampling* method, e.g. as described in HCM Chapter 16, Appendix A (1). This involves counting the number of vehicles in the queue at regular intervals, e.g. every 10 seconds, during the observation period (see *Figure 4*). Delays obtained using the path-trace method agree with the queue sampling method of measurement for low to medium degrees of saturation (v/c ratios), but the difference between the two methods is significant for oversaturated conditions (volume-capacity ratio > 1).

The HCM delay survey method recommends continuing the queue count at the end of the survey period until all vehicles that arrived during the survey period depart. Similarly, vehicles which are in the queue at the start of the survey period should be excluded from the queue count for delay survey purposes. The queue-count method is difficult for oversaturated conditions due to queue build up (it may not be possible to observe the back of queue). The delay measured by the queue-count method

corresponds to the cycle-average queue observation (see the discussion below). The model delay to match this should exclude the geometric delay.

Queue Length

As in the case of delay, it is important to ensure that the queue length estimated by the model matches the queue length observed in the field (and that consistent definitions are used when comparing queue length estimates from different models). It is necessary to establish if the queue length estimated by a model is the *cycle-average queue*, the *back of queue*, or some other queue length measure (e.g. queue length at the start of the unblocked period, i.e. when an acceptable gap occurs in the circulating stream), and if the *average* queue length or some *percentile* (70th, 85th, 90th, 95th and 98th) queue length value is presented in model output.

The traditional gap-acceptance and queuing theory models estimate the cycle-average queue lengths rather than the back of queue. HCM Chapter 16 uses the back of queue for signalized intersections while HCM Chapter 17 uses the cycle-average queue for unsignalised intersections (1). Figure 5 shows the back of queue and cycle-average queue concepts used in aaSIDRA for signalized and unsignalised intersections including roundabouts (9). The cycle-average queue length incorporates all queue states including zero queues. The back of queue is relevant to the design of appropriate queuing space (e.g. for short lane design), and is useful for determining a queue spillback condition (e.g. blockage of an upstream intersection).

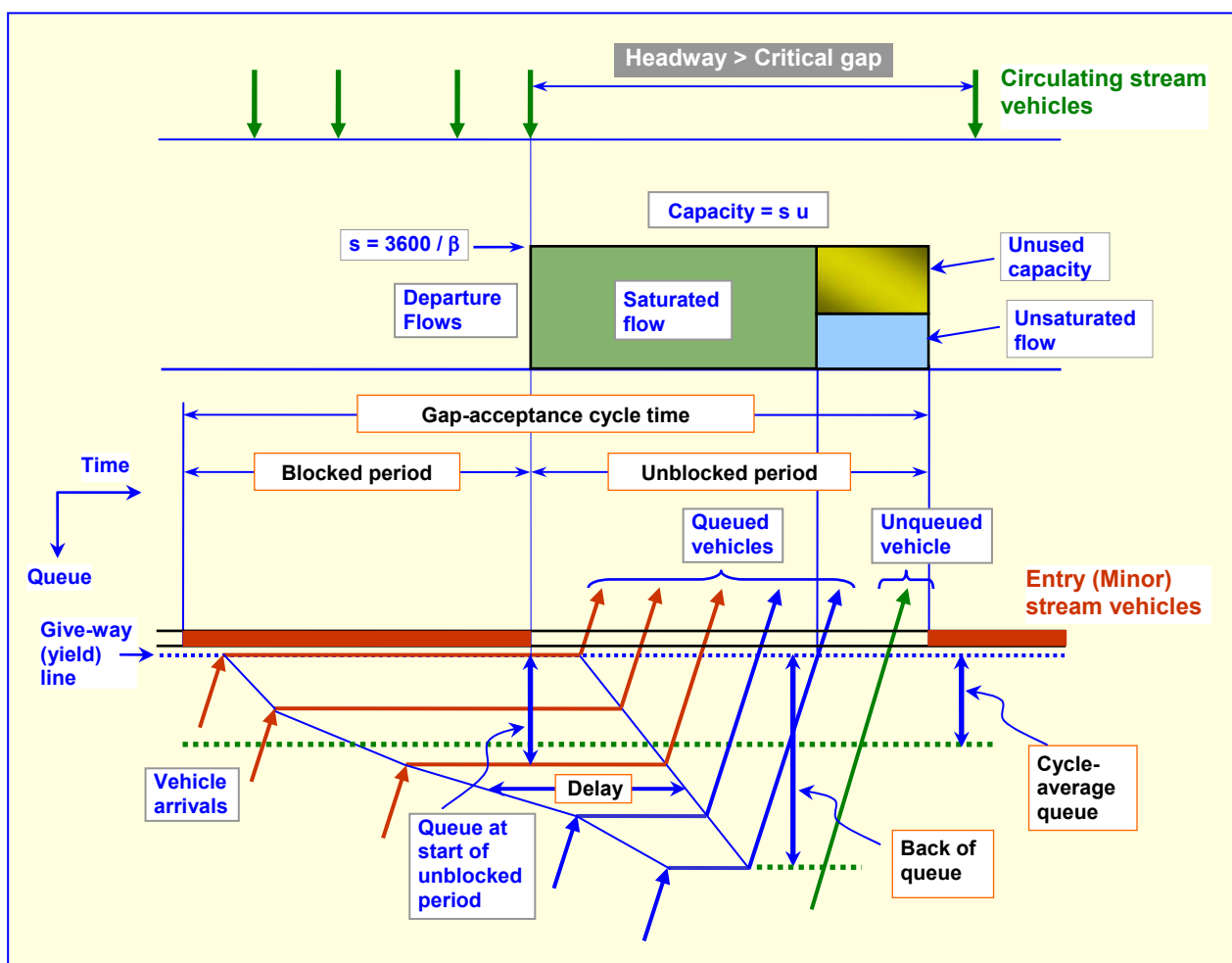


Figure 5 - Modeling of cycle-average queue and back of queue values for roundabouts

The cycle-average queue length information may be available from the queue sampling method of delay measurement. The cycle-average queue length is usually estimated as the product of average delay and flow rate. This may be unreliable when the delay includes the geometric delay and the delay experienced after the end of analysis (survey) period, and excludes the delay experienced by vehicles arriving before, and departing during, the current flow period (as discussed above).

Roundabout Level of Service

The HCM (1) does not specify level of service criteria for roundabouts. Stanek and Milam (21) used the HCM stop-sign control criteria for roundabout level of service. In aaSIDRA, the HCM signalized intersection level of service criteria are used for roundabouts (9).

The HCM justifies the use of different LOS criteria for sign-controlled intersections as follows: "... different transportation facilities create different driver perceptions. The expectation is that a signalized intersection is designed to carry higher traffic volumes and experience greater delay than an unsignalized intersection.". Roundabouts can handle much higher demand flow levels than minor movements at stop-sign controlled intersections. The HCM expression "and experience greater delay than an unsignalized intersection" could be interpreted as "and drivers at a signalized intersection can tolerate higher levels of delay than a stop-sign controlled intersection".

The criteria for roundabouts could be between those used for signalized intersections and stop-sign control. Further work on this issue should also take into account the different performance of roundabouts relative to other intersection types at low to medium vs high demand flow conditions.

CALIBRATION METHODS

Having ensured that there are no definitional problems in comparing observed values and model estimates, a significant difference may be found between observed and estimated values. In such a case of overestimation or underestimation in capacity, delay or queue length estimates, input and default values of key model parameters need to be modified in order to match the observed values.

Two basic methods for calibration that can be used for gap-acceptance and linear regression methods are:

- (i) modify the follow-up headway and critical gap values so that estimates of capacity, delay or queue length values match the observed values, as provided with the aaSIDRA method (8);
- (ii) modify the intercept value of the linear capacity - circulating flow equation, as provided with the UK linear regression method (15).

These two methods are illustrated in *Figures 6 and 7* in terms of calibration based on observed capacity values. Where considered relevant to observed conditions, a percentage of exiting flow should be added to the circulating flow rate in *Figures 6 and 7*.

The process of observation and calibration, and implications on model estimates are summarized in *Table 1* in more general terms. This table relates to the basic calibration methods described below. Further aspects of model calibration are also discussed.

aaSIDRA Gap-Acceptance Method

In aaSIDRA (9), the roundabout capacity is estimated from:

$$Q = s u = (3600 / \beta) u \quad (1)$$

where $s = 3600 / \beta$ is the saturation flow rate (veh/h), β is the follow-up headway (*saturation headway*) and u is the unblocked time ratio.

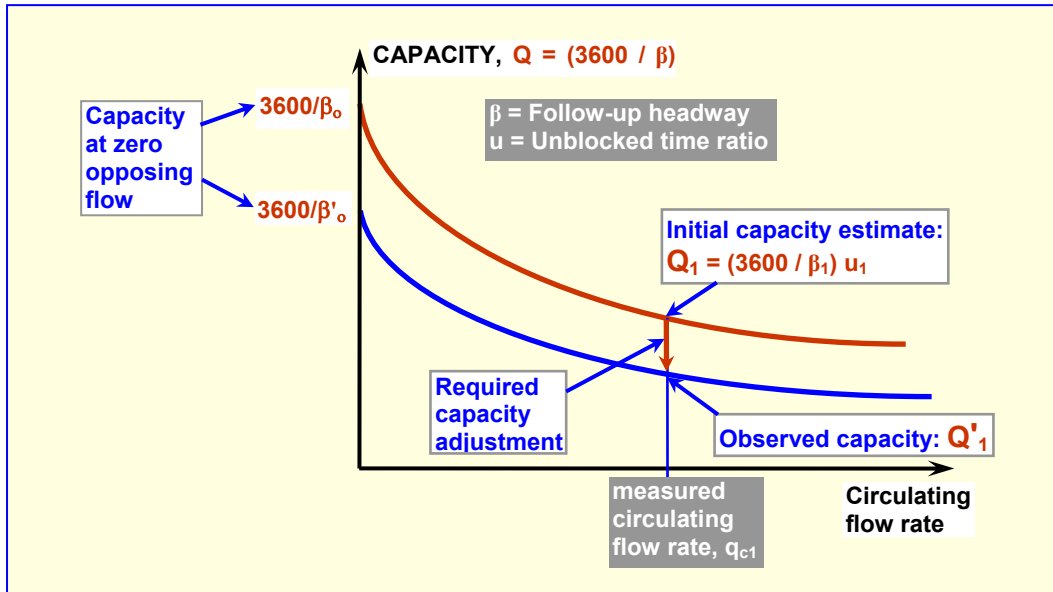


Figure 6 - Calibration of gap-acceptance parameters to match observed capacity

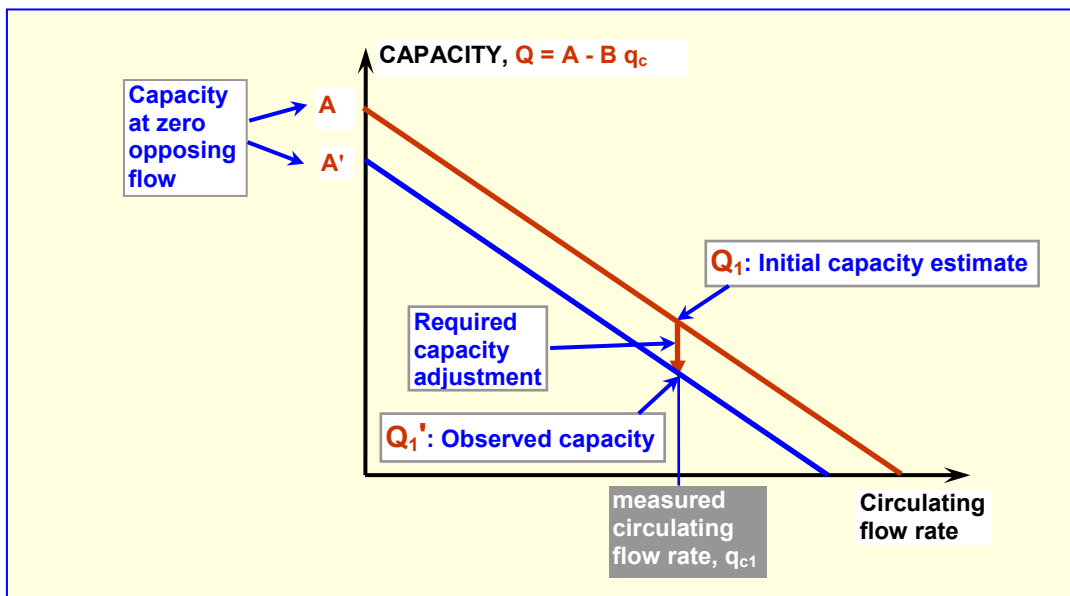


Figure 7 - Adjustment of the intercept of linear regression equation to match observed capacity

As seen in Figure 6, the maximum capacity is obtained under very low circulating flow conditions (for example, $\beta_0 = 3.0$ s means a maximum capacity of $3600 / 3.0 = 1200$ veh/h). The follow-up headway and unblocked time ratio decrease with increasing circulating flow rate. The net result is decreased capacity with increasing circulating flow rate.

The capacity estimate for a given circulating flow rate (q_{c1}) is $Q_1 = (3600 / \beta_1) u_1$. To match the observed capacity value of Q'_1 , the follow-up headway to be specified (β'_1) instead of the estimated value (β_1) can be calculated from:

$$\beta'_1 = \beta_1 (Q_1 / Q'_1) \tag{2}$$

This is an approximate value calculated assuming that the unblocked time ratio (u_1) does not change. The critical gap value can also be adjusted proportionally (assuming that the ratio β / α remains unchanged):

$$\alpha'_1 = \alpha_1 \beta'_1 / \beta_1 \quad (3)$$

A new capacity estimate would be obtained using the adjusted gap-acceptance parameters and compared with the measured value. Because of non-linearity of the capacity - circulating flow relationship, a few iterations would be needed to determine the appropriate values of gap-acceptance parameters.

If desired, the final value of the ratio (β'_1 / β_1) can be used as a factor that modifies the capacity equation generally through parameters β and u in *Equation (1)*. The aaSIDRA software package provides calibration facilities to make this process easy.

UK Linear Regression Method

The calibration method recommended for the UK linear regression model (15) is similar to the method described above. As seen in *Figure 7*, this model estimates capacity from a linear equation:

$$Q = A - B q_c \quad (4)$$

where A is the intercept (maximum capacity obtained under very low circulating flow conditions) and B is the slope of the line (the rate of decrease in capacity with increasing circulating flow rate).

The capacity estimate for a given circulating flow rate (q_{c1}) is $Q_1 = A - B q_{c1}$. To match the observed capacity value of Q'_1 , the intercept to be specified (A') instead of the estimated value (A) can be calculated from:

$$A' = Q'_1 + B q_{c1} \quad (5)$$

The intercept calculated from *Equation (5)* is an approximate value based on the assumption that the slope (B) does not change. Based on this assumption, the capacity equation is then:

$$Q = A' - B q_c \quad (6)$$

Table 1 - Model calibration process

| Method | Observed parameters | Gap-acceptance parameters | Capacity | Delay or Queue Length |
|---|---------------------------|---|---|---|
| aaSIDRA and other gap-acceptance methods | Gap-acceptance parameters | Specify observed gap-acceptance parameters. | Capacity estimate is modified by observed gap-acceptance parameters. | Affected by modified capacity estimate (indirect effect), and observed gap-acceptance parameters (direct effect). |
| | Capacity | Specify modified gap-acceptance parameters to match observed capacity. | Observed capacity is achieved. | Affected by observed capacity (indirect effect), and modified gap-acceptance parameters (direct effect). |
| | Delay or queue length | Specify modified gap-acceptance parameters to match observed delay or queue length. | Capacity estimate is affected via modified gap-acceptance parameters. | Observed delay or queue length is achieved using modified gap-acceptance parameters (direct effect) and the resulting modified capacity estimate (indirect effect). |
| UK and other linear regression methods | Capacity | Not applicable | Specify modified intercept to match observed capacity. | Affected by observed capacity (indirect effect). |
| | Delay or queue length | Not applicable | Specify modified intercept to match observed delay or queue length. Capacity estimate is affected via modified intercept. | Observed delay or queue length is achieved using modified capacity estimate (indirect effect). |

Further Aspects of Model Calibration

The above methods are limited to the calibration of basic aspects of gap-acceptance and linear regression models. The method described for aaSIDRA can be applied to any gap-acceptance method, e.g. the HCM method (1), and the method described for the UK roundabout model can be applied to any linear regression model, e.g. the German linear regression model (22) and the FHWA model (23). However, model calibration is not limited to basic capacity model calibration, and involves all aspects of the analysis methodology. Some of these model calibration issues are discussed below.

The aaSIDRA roundabout capacity model can be calibrated to reflect local road and driver characteristics and particular intersection conditions if necessary. This can be achieved using the *Environment Factor* and *Adjustment Level for Arrival Flow / Circulation Flow Ratio* parameters. These parameters affect the follow-up headway and critical gap (therefore capacity) values of all lanes at a roundabout.

Environment Factor

The aaSIDRA roundabout capacity model can be calibrated to reflect local road and driver characteristics and particular intersection conditions using an *Environment Factor*. This parameter can be used to calibrate the capacity model to allow for less restricted (higher capacity) and more restricted (lower capacity) roundabout environments. This factor represents the general roundabout environment in terms of roundabout design type, visibility, significant grades, operating speeds, size of light and heavy vehicles, driver aggressiveness and alertness (driver response times), pedestrians, heavy vehicle activity (goods vehicles, buses or trams stopping on approach roads), parking turnover and similar factors affecting vehicle movements on approach and exit sides as well as the circulating road as relevant. These are taken into account in terms of their impact on vehicles entering the roundabout.

Higher capacity conditions could be a result of factors such as good visibility, more aggressive and alert driver attitudes (smaller response times), negligible pedestrian volumes, and insignificant parking and heavy vehicle activity (goods vehicles, buses, trams stopping on approach roads).

Lower capacity (more restricted) conditions could be a result of factors such as compact roundabout design (perpendicular entries), low visibility, relaxed driver attitudes (slower response times), high pedestrian volumes, and significant parking and heavy vehicle activity (goods vehicles, buses, trams stopping on approach roads).

The default value of the Environment Factor is 1.0. This factor adjusts the dominant lane follow-up headway at zero circulating flow. As a result, the dominant lane follow-up headway values at all circulating flows are adjusted. This leads to the adjustment of subdominant lane follow-up headway, as well as adjustments of critical gaps for all lanes. Capacity increases with decreasing value of the Environment Factor, e.g. 0.95 will give higher capacities compared with the default value of 1.0, while 1.05 will give lower capacities. Effect of the Environment Factor on capacity is shown for an example in Figure 8. Note that, in the aaSIDRA method, gap-acceptance parameters depend on approach lane characteristics identified as dominant lane (higher lane volume, lower follow-up headway and critical gap values) and subdominant lane (lower lane volume, higher follow-up headway and critical gap values) as indicated by Australian research on roundabouts (10,12,13).

Adjustment Level for Arrival Flow / Circulation Flow Ratio

In order to avoid underestimation of capacities at low circulating flows, aaSIDRA decreases the dominant lane follow-up headway as a function of the ratio of arrival (entry lane) flow to circulating flow. As with the Environment Factor, the adjustment (reduction) of the dominant lane follow-up headway results in reduction of the subdominant lane follow-up headway as well as the critical gap values for both dominant and subdominant lanes. As a result capacities are increased for all entry lanes.

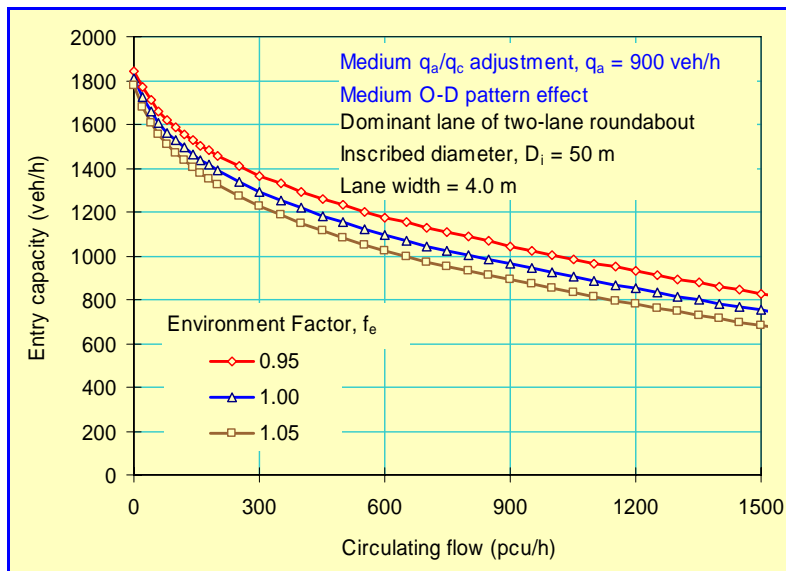


Figure 8 - Effect of the Environment Factor on capacity for the dominant lane of a two-lane roundabout (inscribed diameter = 50 m, average lane width = 4.0 m, Medium O-D pattern effect, Medium entry flow / circulating flow adjustment, entry flow rate = 900 veh/h, no heavy vehicles)

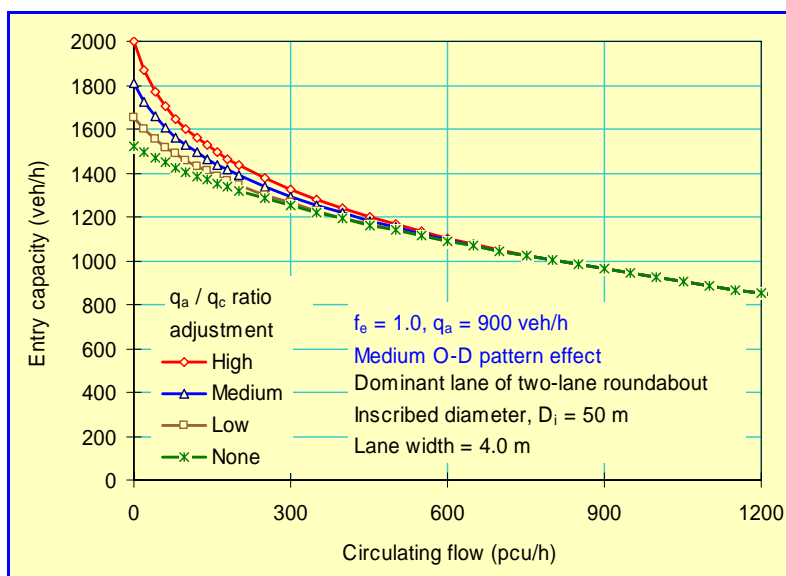


Figure 9 - Effect of the Adjustment Level for Arrival Flow / Circulation Flow Ratio on capacity for the dominant lane of a two-lane roundabout (example as in Figure 8)

The user can calibrate the roundabout capacity model by choosing the level of this adjustment according to the observed or expected local driver behavior characteristics (level of adjustment can be High, Medium, Low and None). The selected level determines the adjusted dominant lane follow-up headway at zero circulating flow. The adjustment (decrease in follow-up headways and critical gaps, therefore increase in capacity) is effective for low to medium circulating flow rates. Capacity is highest when High is selected, and lowest when None is selected (see Figure 9).

Lane Utilization Factor

Lane utilization is one of the most important factors that affect the capacity and traffic performance at all facilities (intersections as well as midblock). For example, unequal lane utilization in opposing traffic lanes at traffic signals means higher opposing volumes and longer queue clearance times, and therefore reduces the opposed (permitted) turn capacity. Similarly, the utilization of circulating road lanes is important at roundabouts in determining the capacity of the entry lane giving way (yielding) to the circulating stream in front of it. This subject has been discussed above in some detail with an example given in *Figure 2* (also see the Case Study section).

The calibration effort should aim to replicate the observed lane flows at roundabout approaches. This is a key aspect of model calibration for all traffic facilities. aaSIDRA allows the user to specify lane utilization factors in order to allow for lane underutilisation (unequal lane utilization) observed in the field. The resulting lane flows estimated by aaSIDRA can be compared with the observed lane flows and the lane utilization ratio can be modified for the estimated lane flows to match observed values.

Heavy Vehicle Equivalent for Gap Acceptance

aaSIDRA gap-acceptance models allow for the effect of heavy vehicles on the capacity of an opposed traffic stream (entry stream at roundabouts) by using a *heavy vehicle equivalent for gap acceptance*. This parameter represents the passenger car equivalent of a heavy vehicle for the purposes of gap-acceptance capacity estimation. It is used to calculate a *heavy vehicle factor* according to the proportion of heavy vehicles in the traffic stream.

The heavy vehicle factor is used to adjust (increase) the follow-up headway and critical gap parameters for heavy vehicles in the entry stream, and adjust (increase) the opposing / circulating stream volume to a pcu/h value for heavy vehicles in the opposing / circulating stream. The effect of these adjustments is to decrease the entry stream capacity. This method applies to modeling entry streams at roundabouts, minor streams at two-way sign control, all-way stop sign control as well as filter (permitted) turns at traffic signals. *Figure 10* shows the entry capacity for 5 and 15 per cent heavy vehicles in the entry stream for the dominant lane of a two-lane roundabout.

Where significant heavy vehicle volumes exist (above about five per cent), the calibration method should consider the heavy vehicles effects carefully. At locations where unusually large heavy vehicles exist, the *heavy vehicle equivalent for gap acceptance* can be adjusted in order to match observed conditions.

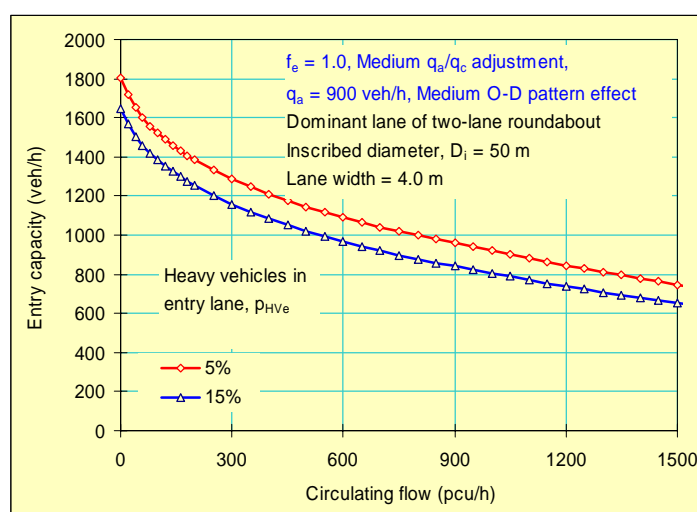


Figure 10 - Effect of the heavy vehicles in the entry stream for the dominant lane of a two-lane roundabout (example as in Figure 8)

Driver Response Time

The saturation headway (follow-up headway for roundabouts) is influenced by the driver response time during queue discharge (average duration between the start times of each vehicle in the queue), jam spacing of vehicles in queue and the queue discharge speed. For the purpose of calibrating microsimulation models, the following relationship is useful (24,25):

$$h_s = t_r + L_{hj} / v_s \quad (7)$$

where

- h_s = the saturation (queue discharge) headway (seconds),
- t_r = driver response time during queue discharge (seconds),
- L_{hj} = jam spacing, or queue space per vehicle, including the vehicle length and the gap distance between vehicles in the queue (metres or feet), and
- v_s = saturation speed (m/s or ft/s).

Equation (7) shows the importance of vehicle length and driver alertness (affecting the gap distance between vehicles in the entry lane queue and queue discharge speed) on the saturation flow rates, and therefore capacity. Thus, the driver response time during queue discharge can be determined from a given saturation headway using the following relationship:

$$t_r = h_s - L_{hj} / v_s \quad (8)$$

The driver response time can be used to assess reasonableness of the assumption about saturation flow rates at signals ($s = 3600 / h_s$) and follow-up headways in gap-acceptance situations ($h_s = \beta$). As a rough rule, the saturation speed can be selected as about 70 per cent of the speed limit for signalized intersections, or about 70 per cent of the negotiation speed of the entry stream at unsignalized intersections. The saturation speed can be observed easily while driving a car, e.g. when the car crosses the stop / give-way (yield) line after accelerating from the queued position. Queue discharge response times of 0.8 s to 1.4 s were observed at signalized intersections in Melbourne and Sydney, Australia (25,26).

For example, using a saturation speed of $v_s = 27 \text{ km/h} = 7.5 \text{ m/s}$ and $L_{hj} = 7.0 \text{ m}$ for a light vehicle, a driver response time of $t_r = 1.6 \text{ s}$ is calculated from *Equation (8)* for a follow-up headway of $h_s = \beta = 2.5 \text{ s}$ (implied maximum capacity = $3600 / 2.5 = 1440 \text{ veh/h}$). If the unblocked time ratio is 0.50 (gaps available 50 per cent of the time), the capacity is 720 veh/h. Consider conditions where the jam spacing is larger, say $L_{hj} = 8.0 \text{ m}$ (due to larger vehicles and drivers leaving more gap behind the vehicle in front in the queue), saturation speed is lower, say $v_s = 18 \text{ km/h} = 5.0 \text{ m/s}$, and driver response time is a little longer, say $t_r = 1.8 \text{ s}$ (more relaxed, hesitant or restricted driver behavior). From *Equation (7)*, a follow-up headway of $h_s = \beta = 3.4 \text{ s}$ is found, which implies a maximum capacity of $3600 / 3.4 = 1059 \text{ veh/h}$. For the unblocked time ratio of 0.50, the capacity is 529 veh/h, a substantial decrease in capacity compared with 720 veh/h above. This is presented to demonstrate the importance of understanding the effect of driver behavior factors on capacity.

Operating Cost, Emissions and Fuel Consumption

In some traffic engineering projects, the interest goes beyond delay and queue length values, and environmental and cost-benefit considerations become very important (27). Models that can provide estimates of emissions, fuel consumption and operating cost need to be calibrated for local vehicle characteristics (selecting light and heavy vehicle parameters that represent local vehicle population) and value of time for local conditions given the wide variation of cost and related factors between different countries around the world, and even between different regions of one country.

aaSIDRA provides a facility for this purpose. Calibration parameters include pump price of fuel, fuel resource cost factor, ratio of running cost to fuel cost, average income, time value factor, average occupancy (persons/veh), light vehicle mass, and heavy vehicle mass.

Models of acceleration and deceleration times and distances, and intersection negotiation speeds, times and distances are also important in determining cost, emission and fuel consumption, as well as geometric delay estimates, and therefore calibration of these models is also important for both analytical and simulation models (27,28).

A CASE STUDY

Previous publications by the author presented a number of case studies of roundabouts ranging from single-lane to three-lane roundabouts from Australia, UK and USA (3-8). One of these case studies is given here for the purpose of discussing capacity estimates from the gap-acceptance based aaSIDRA method and the (TRL) linear-regression method (15-17) and presenting a calibration example.

This two-lane roundabout case (Figures 11 and 12) is based on an example presented by Chard (Case A) who demonstrated the lack of sensitivity of the UK (TRL) linear regression model to different approach lane use arrangements (19,20). The case is presented for driving on the right-hand side of the road and with metric units. The volumes are modified in order to demonstrate the importance of unbalanced flow conditions as well as approach and circulating road lane use issues.

Chard's article addressed prediction problems associated with the "approach" method of traffic modeling which lumps traffic in individual lanes of an approach together irrespective of lane arrangements (exclusive or shared) and any unequal lane utilization (including the case of defacto exclusive lanes). Chard stated that "(the TRL model) can take no account of either unused or unequally used lanes or flared sections on roundabout entry approaches. (The TRL model) is, in fact, completely 'blind' to such occurrences, and as a consequence may produce hopelessly optimistic predictions."

See Figure 2 in the Field Observations section of this paper for a simple example that explains this problem.

Figure 11 shows the case with **exclusive approach lanes and single-lane circulating road**. Approach lane disciplines are as shown in Figure 5a of Chard. Irrespective of specifying a single-lane or two-lane circulating road, all circulating streams would operate effectively as single-lane movements due to exclusive lane arrangements on approach roads (this reduces the capacity of the roundabout). A variety of options would be feasible for approach and circulating lane arrangements for this roundabout, using various combinations of approach roads with exclusive or shared lanes and single-lane or two-lane circulating roads.

Figure 12 shows the case with **shared approach lanes and two-lane circulating road**. This arrangement increases capacities due to the better balance of flows in approach lanes to make use of available lane capacities as well as better opportunity to accept gaps in multi-lane circulating streams. Equal lane utilization is assumed on all approaches (this is the minimum degree of saturation solution).

Analyses are carried out for a 15-min peak period. The hourly flow rates calculated from 15-min peak volumes are shown in Figures 11 and 12 (same for both options). The geometry data (as specified by Chard) are summarized in Table 2. The data given in Table 2 are for the single-lane circulating road case as in Figure 11 (data in US units are also given in Table 2). For the two-lane circulating road case shown in Figure 12, the circulating road width is 10 m and the central island diameter is 20 m. The inscribed diameter is $D_i = 40$ m in both cases.

The circulating flow in front of each approach consists of traffic from one approach only at this roundabout. The circulating flow rate in front of the South approach is significant but not high. These conditions indicate potential for unbalanced flow conditions.

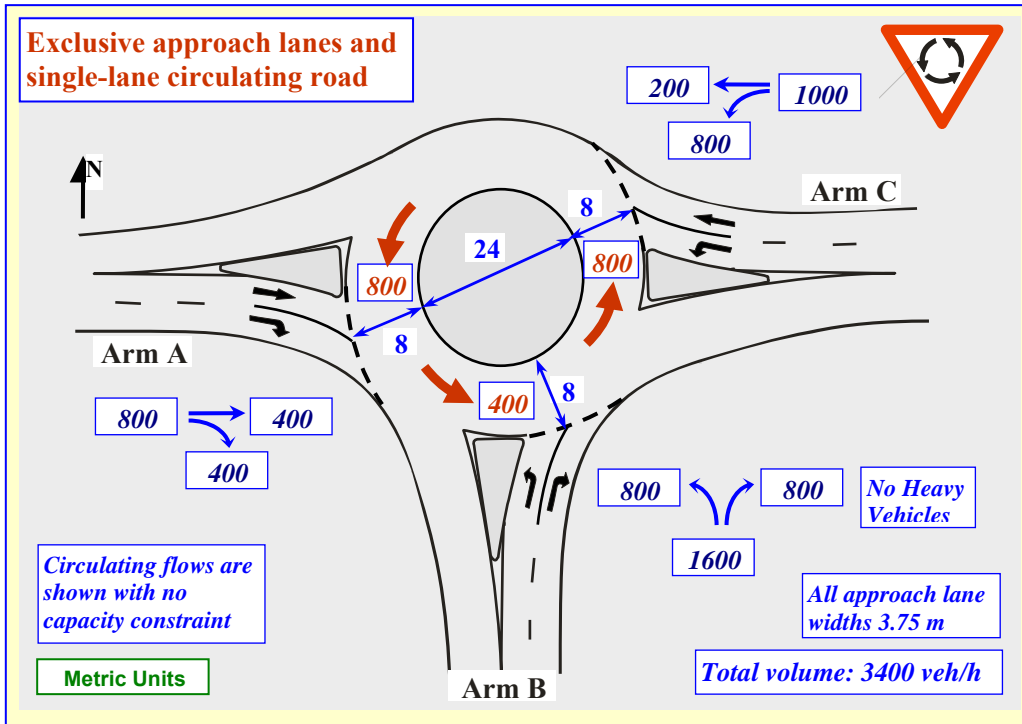


Figure 11 - A T-intersection roundabout case based on an example given by Chard (19,20): exclusive approach lanes and single-lane circulating road

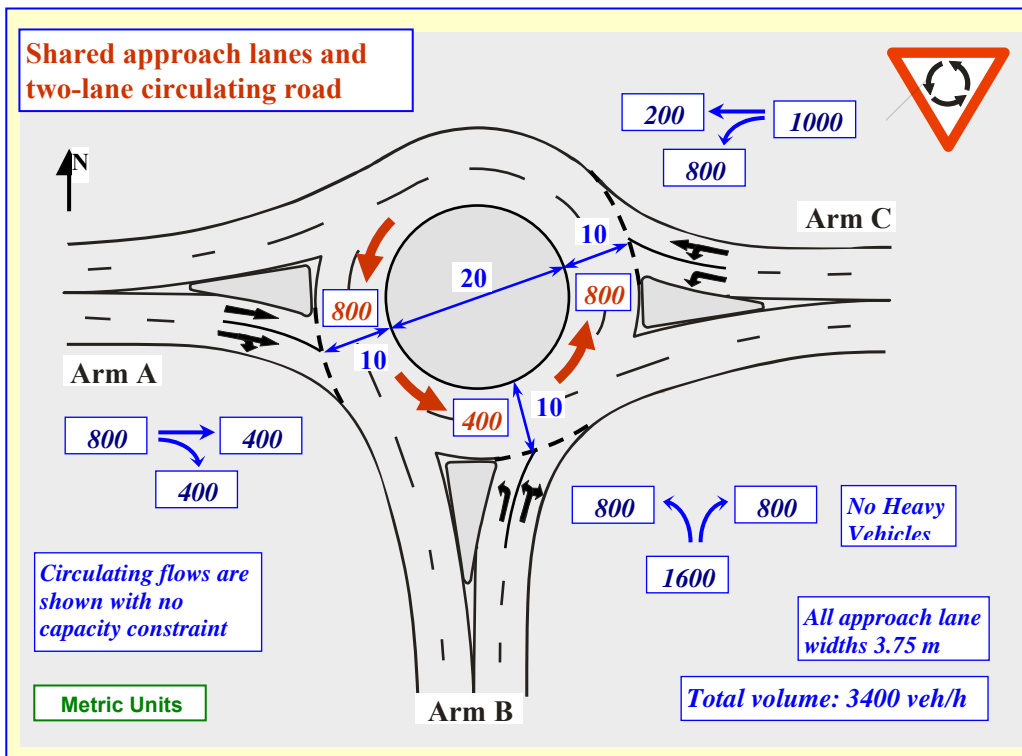


Figure 12 - A roundabout T-intersection case based on an example given by Chard (19,20): shared approach lanes and two-lane circulating road

Table 2 - Geometry data for the T-intersection roundabout

| Approach ID | Approach Name | Average entry lane width | Total entry width | App. half width | Flare length (effective) | Entry radius | Entry angle |
|-------------|---------------|--------------------------|-------------------------|------------------------|--------------------------|-------------------------|----------------|
| | | w_L | w_e | w_a | L_f | r_e | Φ_e (deg) |
| W | Arm A | 3.75 m 13 ft | 7.50 m 26 ft | 6.0 m 23 ft | 10 m 33 ft | 20 m 66 ft | 40 |
| S | Arm B | 3.75 m 13 ft | 7.50 m 26 ft | 6.0 m 23 ft | 10 m 33 ft | 20 m 66 ft | 40 |
| E | Arm C | 3.75 m 13 ft | 7.50 m 26 ft | 6.0 m 23 ft | 10 m 33 ft | 20 m 66 ft | 40 |
| Approach ID | Approach Name | Inscribed diameter | Central island diameter | Circulating road width | No of entry lanes | No of circulating lanes | |
| | | D_i | D_c | w_c | n_e | n_c | |
| W | Arm A | 40.0 m 132 ft | 24.0 m 80 ft | 8.0 m 26.0 ft | 2 | 1 (2) | - |
| S | Arm B | 40.0 m 132 ft | 24.0 m 80 ft | 8.0 m 26.0 ft | 2 | 1 (2) | - |
| E | Arm C | 40.0 m 132 ft | 24.0 m 80 ft | 8.0 m 26.0 ft | 2 | 1 (2) | - |

For the **single-lane circulating road** case ($n_c = 1$) shown in [Figure 11](#), the circulating road width is $w_c = 26$ ft (8 m) and the central island diameter is $D_c = 80$ ft (24 m). These are given above.

For the **two-lane circulating road** case ($n_c = 2$) shown in [Figure 12](#), the circulating road width is $w_c = 33$ ft (10 m) and the central island diameter is $D_c = 66$ ft (20 m). The inscribed diameter is $D_i = 132$ ft (40 m in both cases).

The parameter values in metric and US customary units are not necessarily precisely converted values.

Estimates of capacity, degree of saturation (v/c ratio) and practical spare capacity for the aaSIDRA and UK (TRL) linear regression models are given in [Table 3](#). It is seen that aaSIDRA estimates differ significantly for the single-lane and two-lane circulating road cases whereas the UK (TRL) model estimates for the two cases are the same.

aaSIDRA estimates oversaturated conditions for the East approach (Arm C) in the case of **single-lane circulating road with exclusive lanes** ($x = 1.091$), but estimates satisfactory operating conditions in the case of **two-lane circulating road with shared lanes** ($x = 0.705$). aaSIDRA estimates more favorable gap-acceptance conditions in the case of two-lane circulating flows, and the approach lane use is more balanced with shared lanes. The UK (TRL) model estimates satisfactory conditions for both cases ($x = 0.671$). Assumptions of the "approach" method used in the UK (TRL) model are close to the case of **two-lane circulating road with shared approach lanes**, and therefore in close agreement with the aaSIDRA method. On the other hand, a large discrepancy is found between the two models in the case of **single-lane circulating road with exclusive lanes**.

Using a lane-by-lane method, aaSIDRA identifies critical lanes distinguishing between exclusive and shared lane cases and allowing for any unequal lane utilization. Combined with the unbalanced flow effects resulting from the O-D flow pattern and unfavorable gap-acceptance conditions presented by single-lane circulating flows, aaSIDRA is able to identify oversaturation on the East approach in the case of single-lane circulating road with exclusive lanes. On the other hand, the TRL capacity model combines exclusive and shared lanes to obtain an average approach degree of saturation (similar to the simple example given in *Figure 2*), and therefore cannot identify unequal lane utilization and cannot distinguish between different lane use arrangements.

aaSIDRA estimates of delay, operating cost, fuel consumption and CO₂ emission comparing the case of single-lane circulating road with exclusive lanes vs the case of two-lane circulating road with shared lanes showed that, considering annual values of one hour of traffic operation only, the difference between the two cases amounted to approximately 9,000 person-hours of delay, US\$72,000 in operating cost, 14,000 L of fuel consumption and 34,000 kg of CO₂ emission per year.

Table 3 - Capacity results for the T-intersection roundabout

| App. ID | Approach Name | Total App. Flow (veh/h) | Circul. Flow (1) (pcu/h) | Critical Lane (2) | Critical Lane Flow (veh/h) | Total App. Capacity (veh/h) | Critical Lane capacity (veh/h) | Degree of saturation (v/c ratio) | Practical Spare Capacity ($x_p = 0.85$) |
|---|---------------|-------------------------|--------------------------|-------------------|----------------------------|-----------------------------|--------------------------------|----------------------------------|---|
| aaSIDRA: Single-lane circulating road and exclusive approach lanes | | | | | | | | | |
| W | Arm A | 800 | 733 | 1 (T) | 400 | 1435 | 629 | 0.635 | 34% |
| S | Arm B | 1600 | 400 | 1 (L) | 800 | 2167 | 984 | 0.813 | 5% |
| E | Arm C | 1000 | 800 | 1 (L) | 800 | 1224 | 733 | 1.091 | -22% |
| aaSIDRA: Two-lane circulating road and shared approach lanes | | | | | | | | | |
| W | Arm A | 800 | 800 | 2 (TR) | 431 | 1507 | 812 | 0.531 | 60% |
| S | Arm B | 1600 | 400 | 2 (LR) | 841 | 2050 | 1078 | 0.781 | 9% |
| E | Arm C | 1000 | 800 | 2 (LT) | 537 | 1419 | 762 | 0.705 | 21% |
| UK (TRL) Linear Regression Model: same for both lane arrangements | | | | | | | | | |
| W | Arm A | 800 | 800 | - | - | 1490 | - | 0.537 | 58% |
| S | Arm B | 1600 | 400 | - | - | 1771 | - | 0.904 | -6% |
| E | Arm C | 1000 | 800 | - | - | 1490 | - | 0.671 | 27% |

- (1) The aaSIDRA circulating flow rate for the West approach includes capacity constraint effect due to oversaturation on East approach ($x > 1.0$) in the case of single-lane circulating road. Circulating flows for the UK (TRL) model, as well as the aaSIDRA model for two-lane circulating road, are without any capacity constraint since all approach lanes are estimated to be undersaturated ($x < 1.0$).
- (2) aaSIDRA approach degrees of saturation represent the critical lane degrees of saturation (L: Left, T: Through, R: Right). The UK (TRL) capacity model combines exclusive and shared lanes to obtain an average approach degree of saturation, and therefore cannot identify unequal lane utilization and cannot distinguish between different lane use arrangements.

Calibration Example for the Case Study

For the case of single-lane circulating road with exclusive lanes in the case study presented above (*Figure 11*), let us assume that the capacity of the left-turn lane on Arm C is 800 veh/h (the degree of saturation is $x = 1.0$), i.e. aaSIDRA underestimated the capacity of this lane by a factor of $733 / 800 \approx 0.92$. Using the method explained above, adjustments need to be made to the follow-up headway and critical gap values to obtain a capacity of 800 veh/h for the left-turn lane on Arm C.

The follow-up headway and critical gap estimates for the capacity estimate of 733 veh/h given in *Table 3* are $\beta = 1.85$ s and $\alpha = 3.33$ s given the particular roundabout geometry and flow conditions (left-turn lane is the dominant lane). The aaSIDRA sensitivity analysis facility indicates that reducing the gap-acceptance parameters to $\beta = 1.73$ s and $\alpha = 3.13$ s would give a capacity estimate of 808 veh/h ($x = 0.990$). It is seen that the follow-up headway and critical gap values are reduced by a factor of 0.94, which is slightly higher than the capacity ratio of 0.92 calculated above. This is accepted as an approximate solution since it is considered that a more refined calibration is not justified. This calibration could be interpreted as adjusting the general gap-acceptance model for the conditions at this roundabout generally, resulting in slightly decreased degrees of saturation for all lanes.

In the case of the UK (TRL) linear regression model, the calibration effort becomes problematic since the model estimates the total approach capacity only. Chard (20) recommended calibrating the intercept of the capacity equation (method described above) applying it to each lane separately as a single-lane model. This raises the question of capacity definition and measurement in the UK model. Kimber (15) recommended that the survey method should "*count the total inflow from the entry ... during successive minutes during which there is continuous queuing in all available lanes in the approach to the entry. Queuing should occur continuously for periods of twenty minutes or more during peak periods ...*".

The method prescribed by Kimber cannot be not applied to Arm C in the case study due to unequal lane utilization, i.e. the left-turn lane is saturated but the through traffic lane is undersaturated (aaSIDRA estimated $x = 0.407$ with an average back of queue value of 1.3 veh for the through traffic lane). Therefore the approach-based UK (TRL) model would not be applicable to unequal lane use cases and could not be calibrated unless the suggestion by Chard is accepted. However, the application of an approach-based model to selected individual lanes of an approach lane is not consistent with the basic premises of the model since the individual lane capacities were not identified (modeled) in the derivation of the capacity model using linear regression method.

An alternative method of calibration for Arm C for the case of single-lane circulating road with exclusive lanes would be to adjust the capacity model to achieve a degree of saturation of 1.0 for the approach (equal to the critical lane degree of saturation). This would mean that the total approach capacity (as an aggregate value) would be 1000 veh/h, a reduction factor of $1000 / 1490 = 0.671$.

The UK (TRL) capacity equation for the results given in *Table 3* is $Q = 2051 - 0.702 q_c$. To obtain a capacity of 1000 veh/h at $q_{c1} = 800$ veh/h, the adjusted intercept is $A' = 1000 + 0.702 \times 800 = 1562$ veh/h (instead of $A = 2051$ veh/h). Thus, the modified capacity equation is $Q = 1562 - 0.702 q_c$. This is valid only for this specific unequal lane use case, and cannot be considered to be a general model calibration process applicable to other approaches of the roundabout. If applied to other lanes at this roundabout, the degrees of saturation on other approaches would increase substantially (for example Arm B would be oversaturated).

CONCLUDING REMARKS

Calibration effort usually focuses on making the best use of an available model in matching the estimates of capacity, delay, queue length, and other statistics produced by the model with values observed in the field. While such an effort can be successful in specific cases, such success does not guarantee the model validity in a general sense. This applies to all models, analytical and simulation. Discussion on the nature of models from the perspective of a general modeling framework is recommended in order to assess the capabilities of alternative models. Such discussion should not be limited to capacity or individual performance measures, but a more general evaluation of model capabilities should be undertaken.

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The author is the developer of the aaSIDRA model, and comments presented in this paper regarding other models should be read with this in mind.

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APPENDIX

Empirical Basis of the the aaSIDRA Gap-Acceptance Model for Roundabouts

Table 4 - Australian roundabout survey data summary (55 roundabout entry lanes)

| | | | | | | | |
|-----------------|------------------------|--------------------|-------------------------------|----------------------|--------------------------|----------------------------|---------------------------|
| | Total entry width (ft) | No. of entry lanes | Average entry lane width (ft) | Circul. width (ft) | Inscribed Diameter (ft) | Entry radius (ft) | Conflict angle (°) |
| Minimum | 12 | 1 | 10 | 21 | 52 | 13 | 0 |
| Maximum | 41 | 3 | 18 | 39 | 722 | ∞ | 80 |
| Average | 27 | 2 | 13 | 31 | 183 | 128 | 29 |
| 15th percentile | 21 | 2 | 11 | 26 | 93 | 33 | 0 |
| 85th percentile | 34 | 3 | 15 | 39 | 230 | 131 | 50 |
| Count | 55 | 55 | 55 | 55 | 55 | 55 | 55 |
| | Total entry width (m) | No. of entry lanes | Average entry lane width (m) | Circul. width (m) | Inscribed Diameter (m) | Entry radius (m) | Conflict angle (°) |
| Minimum | 3.7 | 1 | 3.20 | 6.5 | 16 | 4 | 0 |
| Maximum | 12.5 | 3 | 5.50 | 12.0 | 220 | ∞ | 80 |
| Average | 8.1 | 2 | 3.84 | 9.6 | 56 | 39.0 | 29 |
| 15th percentile | 6.4 | 2 | 3.34 | 8.0 | 28 | 10.0 | 0 |
| 85th percentile | 10.5 | 3 | 4.48 | 11.9 | 70 | 39.8 | 50 |
| Count | 55 | 55 | 55 | 55 | 55 | 55 | 55 |
| | Follow-up Headway (s) | Critical Gap (s) | Crit. Gap / Fol. Hw Ratio | Circul. flow (veh/h) | Total entry flow (veh/h) | Dominant lane flow (veh/h) | Subdom. lane flow (veh/h) |
| Minimum | 0.80 | 1.90 | 1.09 | 225 | 369 | 274 | 73 |
| Maximum | 3.55 | 7.40 | 3.46 | 2648 | 3342 | 2131 | 1211 |
| Average | 2.04 | 3.45 | 1.75 | 1066 | 1284 | 796 | 501 |
| 15th percentile | 1.32 | 2.53 | 1.26 | 446 | 690 | 467 | 224 |
| 85th percentile | 2.65 | 4.51 | 2.31 | 1903 | 1794 | 1002 | 732 |
| Count | 55 | 55 | 55 | 55 | 55 | 55 | 55 |

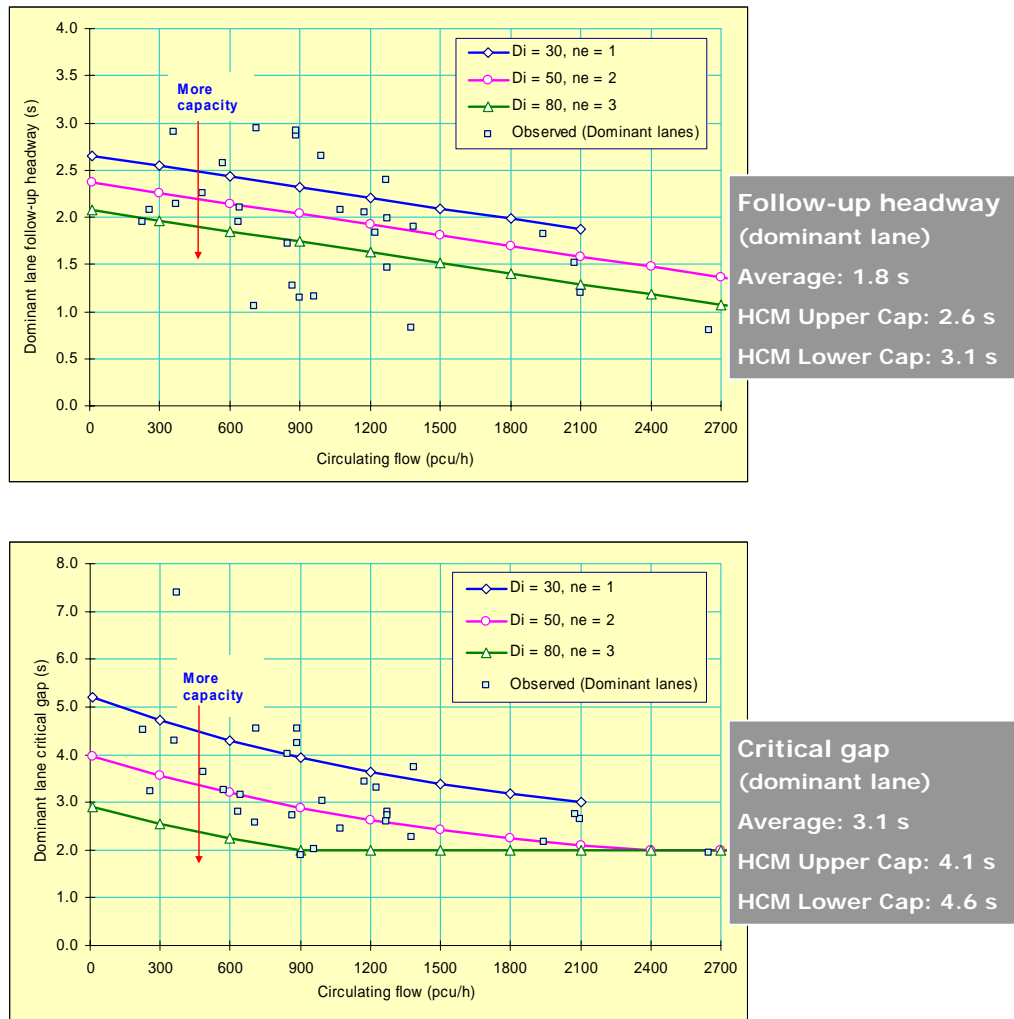


Figure 13 - Follow-up headway and critical gap as a function of roundabout geometry and circulating flow rates (dominant lanes only): Field data and model estimates

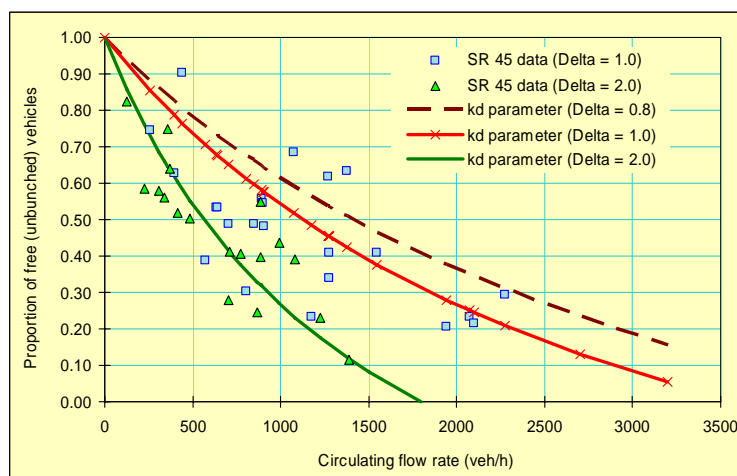


Figure 14 - Proportion of unbunched vehicles in the circulating stream as a function of the circulating flow rate: Field data and model estimates