VicRoads Supplement to the Austroads Guide to Road Design

Part 4B – Roundabouts

NOTE:

This VicRoads Supplement must be read in conjunction with the Austroads Guide to Road Design.

Reference to any VicRoads or other documentation refers to the latest version as publicly available on VicRoads website or other external source.

VicRoads Supplement to the Austroads Guide to Road Design Updates Record

Part 4B – Roundabouts

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Rev. 1.1 Oct 2010	Various	Minor changes to text, references and layouts.	Principal Advisor – Design, Traffic & Standards
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This VicRoads Supplement has been developed by VicRoads Technical Consulting and authorised by the Executive Director – Network and Asset Planning.

The VicRoads Supplement to the Austroads Guide to Road Design provides additional information, clarification or jurisdiction specific design information and procedures which may be used on works financed wholly or in part by funds from VicRoads beyond that outlined in the Austroads Guide to Road Design guides.

References

AGRD – Austroads Guide to Road Design

AGTM – Austroads Guide to Traffic Management

GTEP – Guide to Traffic Engineering Practice (superseded)

SD – Standard Drawings for Roadworks

VRD/RDG – VicRoads Road Design Guidelines (superseded)

Australian Standards (2010). AS1158: Lighting for roads and public spaces.

Akcelik, R. (1991). Implementing roundabout and other unsignalised intersection analysis methods in SIDRA. Australian Road Research Board, WDTE91/002.

Akcelik, R and Troutbeck, R. J. (1991). Implementation of the Australian Roundabout Analysis Method in SIDRA. In Highway Capacity and Level of Service, (Ed. U. Brannolte) Proceedings of the International Symposium on Highway Capacity, Karlsruhe Balkema Rotterdam July 1991.

George, A. L. (1982). Estimation of geometric delay at a roundabout. Unpublished report to the CRB (now VicRoads).

Middleton, G. (1990). Geometric delays at roundabouts. Roundabout and Unsignalised Intersection Workshop, Queensland University of Technology, November, (Ed. R.J. Troutbeck).

Troutbeck, R. J. (1989). Evaluating the Performance of a Roundabout. Australian Road Research Board Special Report 45.

Troutbeck, R.J. (1990). Traffic Interaction at Roundabouts, Proceedings 15th Australian Road Research Board Conference, Volume 15(5), pp. 17-42.

VicRoads (2008). Lighting of Arterial Roads Policy.

VicRoads (2009). Lighting of Freeways Policy.

VicRoads (2010). Guidelines for Road Lighting Design, TCG006-2-2010.

VicRoads (2010). The Installation of Street Lighting Installation, TCG006B-1-2003.

VicRoads Traffic Engineering Manual Volume 1 – Traffic Management.

VicRoads Traffic Engineering Manual Volume 2 – Signs and Markings.

1.0 Introduction

1.1 Purpose

VicRoads has no supplementary comments for this section.

1.2 Scope of this Part

VicRoads has no supplementary comments for this section.

1.3 Road Safety

VicRoads has no supplementary comments for this section.

1.4 Road Design Objectives

VicRoads has no supplementary comments for this section.

1.5 Traffic Management at Roundabouts

VicRoads has no supplementary comments for this section.

1.6 Safety performance of Roundabouts

VicRoads has no supplementary comments for this section.

1.7 Traffic Capacity of Roundabouts

Example calculations can be found in Appendix VA of this Supplement.

1.8 Signalisation of Roundabout

VicRoads has no supplementary comments for this section.

1.9 Significant Change from the Guide to Traffic Engineering Practice – Part 6: Roundabouts

VicRoads has no supplementary comments for this section.

2.0 Design Principles and Procedure

VicRoads has no supplementary comments for this section.

3.0 Sight Distance

3.1 Introduction

VicRoads has no supplementary comments for this section.

3.2 Sight Distance Criteria

3.2.3 Criterion 3

Note that an absolute minimum sight distance is used in *Austroads Guide to Road Design (AGRD) Part 4B, Figure 3.1.*

3.2.4 Other Visibility Considerations

To enhance the prominence of the roundabout, the kerbs on both the splitter island and central island should be light coloured or painted white. VicRoads practice is to avoid the use of dark coloured kerbing.

Where there is a tram or train crossing incorporated into the roundabout, care needs to be taken to ensure that the negotiation speeds are slower and that drivers are aware of the presence and the location of the rail or tram tracks. Where railway tracks are involved, it is preferable to provide level crossing 'Wig-Wag' signals or 'boom barriers' in conjunction with the roundabout.

On-street trams can be successfully incorporated into a roundabout. As the tracks will pass through the central island, eliminating part of it, care needs to be taken to ensure that residual central island remains large enough to be recognised. Electronic signs have been incorporated at complex urban locations and to reinforce tram priority.

3.3 Truck Stopping Sight Distance

VicRoads has no supplementary comments for this section.

4.0 Geometric Design

The design method outlined in the *AGRD Part 4B* has been adopted by VicRoads, however its use is relatively untested in Victoria. It is known that there may be some issues/challenges that designers will encounter when developing designs - especially with incorporation of non perpendicular approaches and duplicated carriageways at roundabout approaches and departures.

Guidance should be sought from VicRoads regarding the design of roundabouts on VicRoads managed arterial roads when using this design method for complex roundabouts. Additional supporting information regarding the use of the new Austroads design method will be included in future updates of this Supplement as further experience is developed.

It should be noted that use of the new Austroads method generally results in larger diameter roundabouts than previous design methods, especially in lower speed environments. If there are site constraints that restrict the use of the new Austroads design method, VicRoads (the client or contract superintendent) should be consulted regarding the development of an acceptable solution.

4.1 Introduction

VicRoads has no supplementary comments for this section.

4.2 Number of Legs

VicRoads has no supplementary comments for this section.

4.3 Number of Entry, Circulating and Exit Lanes

4.3.1 Number of Circulating Lanes

Correction

The first sentence *"The number of circulating lanes from any particular approach must be equal to or greater than the number of entry lanes on that approach"* is not correct and should read:

"*The number of circulating lanes is determined by the number of through lanes on the approach and also considering the right turn movements from the preceding approach."*

4.4 Central Island

4.4.2 Factors Affecting Central Island Size

Additional Information

Roundabouts with a larger inscribed diameter, and consequently a larger central island, have a slightly greater entry capacity. In general, a large central island provides greater separation between adjacent conflict areas and makes it easier for entering drivers to determine whether vehicles, already on the circulating carriageway, are exiting or continuing on around the circulating carriageway. Larger central islands are usually necessary for roundabouts in high speed areas and at multi-leg intersections. Large central islands can also improve driver recognition of the form of intersection treatment.

4.4.3 Minimum Central Island Radius

VicRoads (the client or contract superintendent) should be consulted prior to the adoption of absolute figures in *AGRD Part 4B*, *Table 4.1*.

4.5 Entry Geometry

VicRoads has no supplementary comments for this section.

4.6 Circulating Carriageway

4.6.1 Design Vehicle and Vehicle Swept Paths

To cater for vehicles that only occasionally use the roundabout, it may be acceptable for these vehicles (i.e. checking vehicle) to use multiple lanes to make their turn. The road authority should be consulted regarding the appropriate checking vehicle to be used if this is not specified.

4.6.3 Encroachment Areas

Aprons shall not be adopted on the outside the circulating carriageways of roundabouts to assist larger vehicles making right turns, as this allows for higher speeds through the roundabout by cars. Other design changes should be made to ensure the vehicle can make this movement on the circulating pavement.

4.7 Exit Curves

VicRoads has no supplementary comments for this section.

4.8 Entry and Exit Widths

VicRoads has no supplementary comments for this section.

4.9 Separation between Legs

VicRoads has no supplementary comments for this section.

4.10 Superelevation, Gradient and Drainage

VicRoads has no supplementary comments for this section.

4.11 Special Treatments

VicRoads has no supplementary comments for this section.

5.0 Pedestrian and Cyclist Treatments

5.1 Introduction

VicRoads has no supplementary comments for this section.

5.2 Pedestrians

VicRoads has no supplementary comments for this section.

5.3 Cyclists

5.3.3 Bicycle Lanes at Single-lane Roundabouts

When designing a roundabout to cater for cyclists through the roundabout, the geometric design criteria set out in *AGRD Part 4B*, *Section 4.5.5* must be measured **from the kerb and not the edge** of the bicycle lane.

Swept paths should not encroach into the bicycle lane.

5.3.4 Multi Lane Roundabouts on Arterial Roads

Where it is proposed to provide circulating bicycle lanes on multi lane roundabouts on arterial roads, proposals shall be subject to review prior to design development and at the functional design stage by a panel comprising representatives of the following VicRoads Business Areas:

- VicRoads Region: Relevant regional representative
- Network and Asset Planning: Road and Traffic Standards; Bicycle and Pedestrian Programs
- Road Safety and Network Access: Road Engineering Safety
- Technical Consulting: Road Design, Standards & Traffic
- Major Projects: Relevant project team representative if the project is being proposed through Major Projects.

The proposer of the treatment from the Region or Major Project should convene the panel.

As a minimum, the following information should be available for discussion at reviews:

- Strategic importance of the bicycle route in the network;
- Predicted numbers and types of cyclists using the roundabout (recreation, commuter etc.);
- Predicted traffic volumes including turning volumes;
- Predicted commercial vehicle volumes and mix.

6.0 Pavement Markings and Signing

Refer to *VicRoads Traffic Engineering Manual (TEM) Volume 2* for details on pavement markings and signing.

6.1 Introduction

<u>Additional information</u>

Hazard boards can be placed in large splitter islands as shown in Figure V6.1 below.

6.2 Single-lane Local Street roundabout

VicRoads has no supplementary comments for this section.

6.3 Multi-lane Arterial Road Roundabout

VicRoads has no supplementary comments for this section.

7.0 Roadway Lighting at Roundabout

Roadway lighting is to be in accordance with the VicRoads lighting policies.

The following documents should be referenced:

- *Guidelines for Road Lighting Design, TCG006-2-2010* (VicRoads, 2010).
- *The Installation of Street Lighting, TCG006B-1-2003* (VicRoads, 2010).
- *VicRoads Lighting of Arterial Roads Policy* (VicRoads, 2008).
- *VicRoads Lighting of Freeways Policy* (VicRoads, 2009).
- *AS1158: Lighting of Roads and Public Spaces* (Aust. Stds, 2010).

8.0 Landscaping and Street Furniture

VicRoads has no supplementary comments for this section.

9.0 References

VicRoads has no supplementary comments for this section.

Appendices

Appendix C Methods of Improving Roundabout Entries – Figure C2

Correction

The speed "S_c" value in box on the right hand side of Figure C2 should be 40km/h (not 48km/h as shown).

Appendix D.1 Linemarking of Multi-lane Roundabouts

Refer to *VicRoads TEM Volume 2* for linemarking of multi-lane roundabouts.

Appendix D.2 Single-lane Exits Adjacent to Two Circulating lanes

The use of 'spiral' linemarking is acceptable in Victoria.

Appendix VA Performance of Roundabouts

This appendix contains additional information on capacity analysis.

Appendix VB Worked Examples

This appendix contains information on gap acceptance parameters.

Appendix VC Trial Installations

This appendix contains additional information on trial installations.

Appendix D Case Studies

This appendix contains additional case studies.

Commentaries

VicRoads has no supplementary comments for this section.

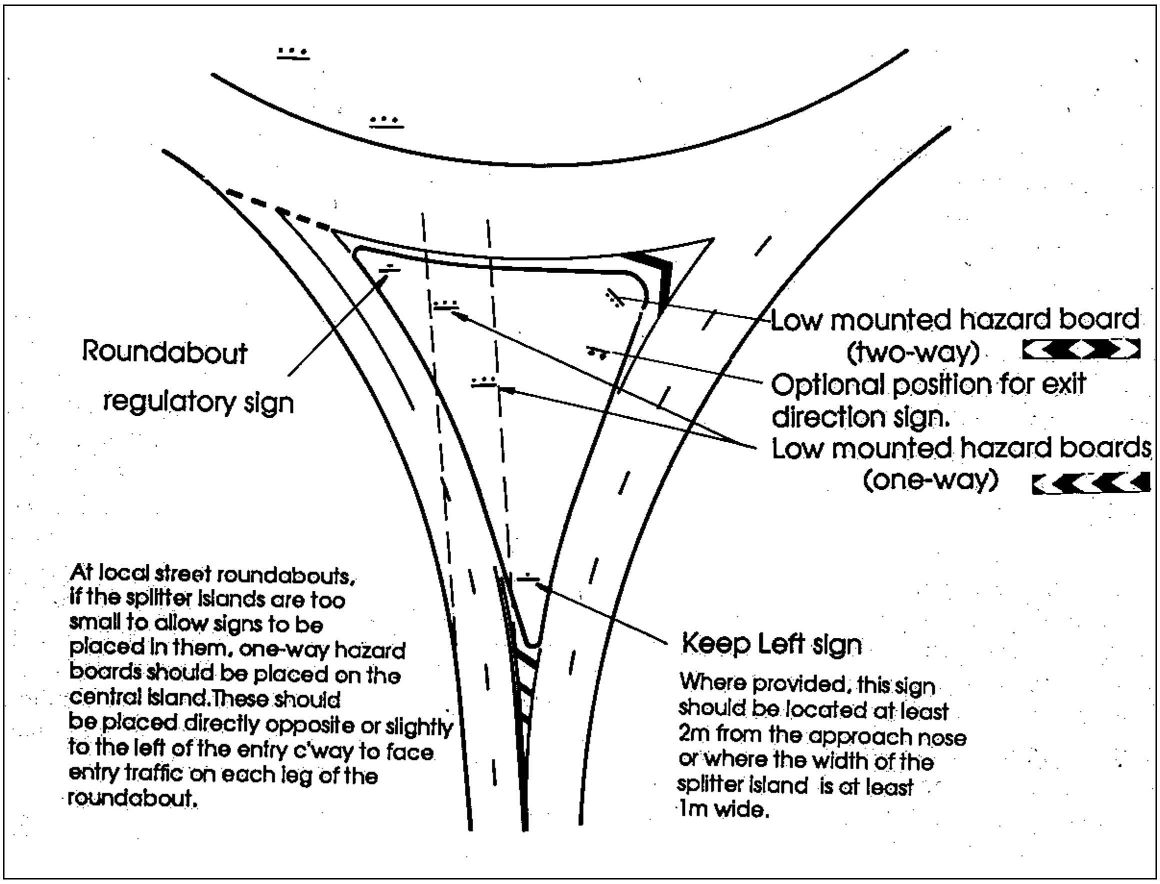
Tables

VicRoads has no supplementary comments for this section.

Figures

VicRoads has no supplementary comments for this section.

Figure V6.1: Signing on larger Splitter Islands (from GTEP Part 6, Figure 6.4)



Appendix VA

(from GTEP Part 6: Roundabouts, Section 3 – Performance of Roundabouts)

Traffic Balance

Roundabouts operate best when the traffic flows are balanced. This does not mean that all movements must be of the same magnitude but simply that the predominant movements are "broken up" by circulating traffic so that gaps are provided to allow vehicles waiting on adjacent legs to enter the roundabout without major delays.

Capacity Analysis

This section provides an analytical technique which can be expected to give quite accurate results which reflect current Australian experience and practice.

In situations where a high degree of accuracy is not required, Figures VA3.3, VA3.5 and VA3.6 may be used to obtain general estimates of the capacity of a roundabout.

Procedure

The capacity of a roundabout is influenced by its geometry through the critical gap parameters. The procedure for capacity analysis of each approach is as follows:

Assemble Traffic Data

Cyclic and stochastic variations in traffic flows should be taken into account when assembling the traffic data into the turning movement flows to be used in the analysis.

Figures VA3.1 and VA3.2 show the conversion of typical traffic turning movements at a cross-road type intersection into entry and circulating flows on a roundabout Where the truck flows are less than 5 percent the total vehicle flow is considered to be passenger car units (pcu's). For truck volumes greater than 5 percent the truck flows should be converted to passenger car units. A single unit truck is assumed to be equivalent to 2 pcu's and an articulated vehicle, 3 pcu's. Equivalencies for other vehicle types (such as bicycles or large combination vehicles) may be estimated and used if necessary.

Number of Entry and Circulating Lanes

The number of entry lanes will generally be determined from the number of lanes on the approach carriageway. However an entry may be widened or flared, particularly if there are heavy turning movements.

It is usually assumed that the number of circulating lanes will equal the number of entry lanes at any approach. This assumption can be relaxed later if required.

Figure VA3.3 is a plot of approach (entry) flows and circulating flows and the acceptability of a single or multi-lane roundabout. The shaded bands indicate the limits for a single lane roundabout and a two lane roundabout. For instance, if conditions at a roundabout give a point in the upper shaded area of Figure VA3.3, then a two lane roundabout may be acceptable depending on the entry geometry and the acceptable degree of saturation. The user may need to evaluate both a two lane and a three lane roundabout in this case. Figure VA3.3 is based on the acceptable degree of saturation being less than 0.8.

Figure VA3.1: Typical Turning Movement Diagram

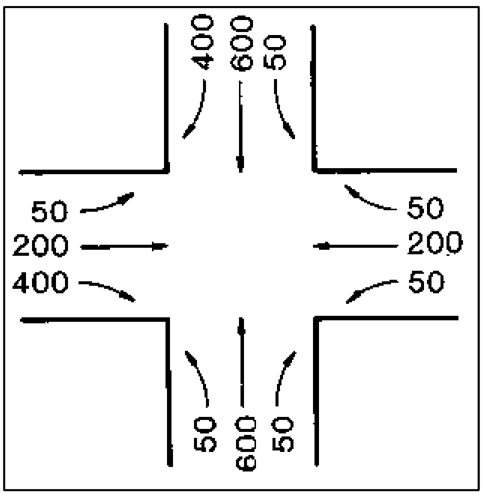


Figure VA3.2: Roundabout Entry and Circulating Flows

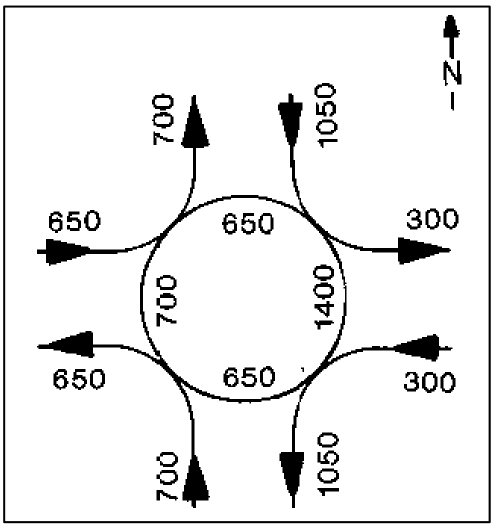
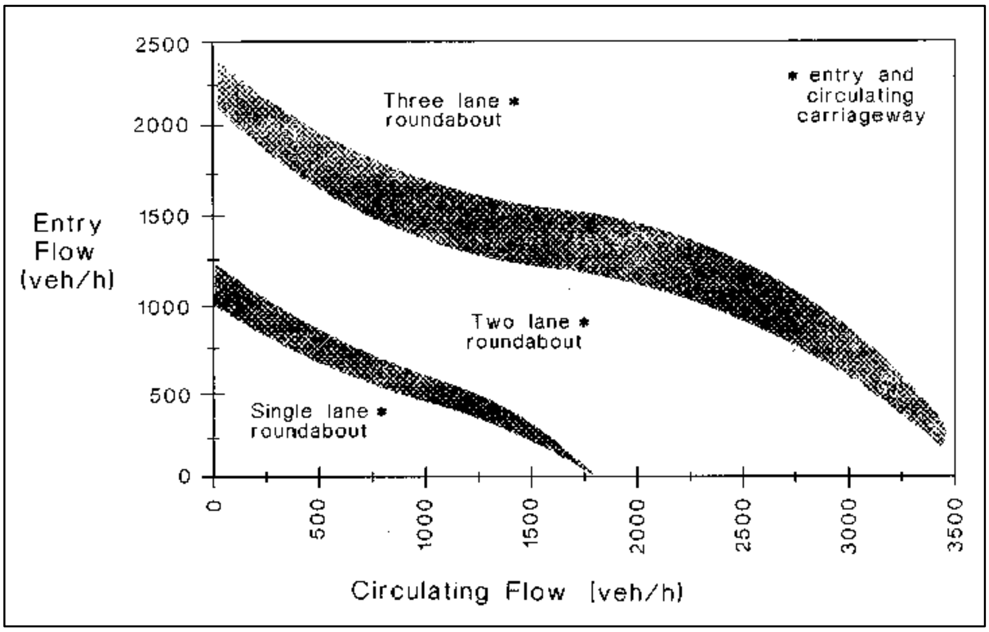


Figure VA3.3: Required Number of Entry and Circulating Lanes



Note: The shaded bands indicate conditions in which either treatment may be suitable depending on the geometry and acceptable operating conditions.

Record the geometric values

From the functional plans of the roundabout or from actual measurement, record the values for:

- the inscribed diameter, Di.
- the number of entry lanes, n_e.

  n_e is 1 for entry widths less than 6 m,

  n_e is 2 for entry widths between 6 and 10 m, and

  n_e is 3 for entry widths greater than 10 m.
- the number of circulating lanes, n_c.

  n_c is equal to 1 for circulating carriageway widths less than 10 m,

  n_c is 2 for widths greater than or equal to 10 m and less than 15 m, and

  n_c is 3 for widths greater than 15 m.

Note: For some circulating carriageways between 8 m and 10 m wide and with circulating flow rates greater than 1000 veh/h, there may be two effective lanes and n_c may be set to 2. See the discussion later in this section.

- the average entry lane width, (or the entry width divided by the number of entry lanes).

Refer to AGRD Part 4B, Section 4 for a procedure for the geometric design of roundabouts.

Classify the entry lane type.

Classify the entry lanes as either dominant or sub-dominant. Where there are two or more entry lanes, one entry lane 'dominates'. That is the drivers in this lane tend to influence the behaviour of drivers in other entry lanes at the approach. The entry lane with the greatest flow is chosen to be the dominant stream; other entry lanes will then be sub-dominant. If there are 3 entry lanes, two will be sub-dominant and only one will be a dominant stream. If there is only one entry lane at an approach then this lane is considered as a 'dominant' lane (Troutbeck, 1989).

Estimate the critical gap acceptance parameters.

Gap acceptance parameters are affected by the geometry of the entry. Geometrics which offer an easier entry path give lower gap acceptance values. These parameters are also a function of the circulating flow. At higher circulating flows, the circulating speeds are lower and drivers are more willing to accept smaller gaps. Also at higher circulating flows, more circulating drivers slow and allow entering drivers to move in front of them. This leads to priority sharing or even a reversal of priority. Further discussion of the interactions is given in Troutbeck (1989 and 1990).

(a) For a single lane entry

Table VA3.1 lists the dominant stream follow-up headway (t_{fd}). If there is one circulating lane (n_c=1), these values are used for the entry stream. If there are 2 or more circulating lanes (n_c=2 or 3), then the values in Table VA3.1 should be increased by 0.39.

The ratio of the critical acceptance gap to the follow-up headway (t_{ad}/t_{fd}) is given in Table VA3.2. The critical acceptance gap is the product of the appropriate values from Table VA3.1 and Table VA3.2.

(b) For Multi-lane Approaches

To estimate the entry lane flows at approaches with two or more lanes, it can be assumed that drivers wishing to turn right will use the right hand entry lanes and the drivers turning left will use the left hand lanes. However in some situations lanes may be marked with signs or pavement arrows to restrict them to particular traffic movements and the lane arrangement so marked would be used in the analysis. The through traffic then needs to be proportioned to the appropriate lanes to finalise the lane entry flows.

While the above provides the most accurate assessment, it is pointed out that estimates of approach capacity are not significantly affected by the distribution of traffic in the lanes.

The entry lane with the greatest flow at an approach is termed the `dominant' lane and the traffic in this lane is termed the dominant stream. Other lanes contain subdominant streams.

The critical gap parameters for an approach with two or more entry lanes are estimated using Tables VA3.1, VA3.2, VA3.3 and VA3.4.

Table VA3.1 gives values for the follow-up headway for the dominant stream. These values are adjusted if the number of entry lanes differs from the number of circulating lanes. The adjustment values are given Table VA3.3.

Table VA3.4 gives the values of the sub-dominant stream follow-up headway (t_{fs}) as a function of the dominant stream follow-up headway (t_{fd}) and the ratio of dominant stream entry flow to the sub-dominant stream entry flow.

The critical acceptance gap values for each lane are given by the product of the follow-up headway (from Tables VA3.1 and VA3.4) and the ratios in Table VA3.2. As stated above, critical acceptance gap values need to be calculated separately for each entry lane. Refer to Appendix VB or Troutbeck (1989) for an example of these calculations.

Estimate the characteristics of the circulating traffic

As the entering drivers give way to all circulating vehicles, the circulating traffic can be considered as if it were all in one lane. There are, however, circulating stream characteristics that change with flow and the number of circulating lanes.

The greater the number of circulating carriageway lanes, the shorter will be the average headway between bunched vehicles in all lanes. If there are two or more circulating lanes then the average headway (t) between bunched vehicles is about 1s and if there is only one lane the average headway is 2s.

If a circulating carriageway equal to or greater than 10 m wide carries a circulating flow greater than 1000 veh/h it can be assumed to effectively operate as two streams and the average headway between bunched vehicles (t) will be 1s. (see Table VA3.5). Under these conditions the vehicles might travel in an offset pattern as shown in Figure VA3.4 and users should decide whether or not the circulating carriageway will be considered to have one or two effective lanes. It may be preferred to consider all single lane roundabouts to have only one effective lane regardless of the circulating flow and hence an average headway between bunched vehicles of 2s. This action would be conservative. Note that if it is considered that there will be two effective circulating streams, then the number of circulating lanes (n_c) should be set to 2. Table VA3.3 may then need to be consulted when estimating the follow-up headways.

Table VA3.1: Dominant Stream Follow-up Headways (t_{fd}) (Initial values in seconds)

Inscribed Diameter (m)	Circulating Flow (veh/h)					
	0	500	1000	1500	2000	2500
20	2.99	2.79	2.60	2.40	2.20	2.00
25	2.91	2.71	2.51	2.31	2.12	1.92
30	2.83	2.63	2.43	2.24	2.04	1.84
35	2.75	2.55	2.36	2.16	1.96	1.77
40	2.68	2.48	2.29	2.09	1.89	1.70
45	2.61	2.42	2.22	2.02	1.83	1.63
50	2.55	2.36	2.16	1.96	1.76	1.57
55	2.49	2.30	2.10	1.90	1.71	1.51
60	2.44	2.25	2.05	1.85	1.65	1.46
65	2.39	2.20	2.00	1.80	1.61	1.41
70	2.35	2.15	1.96	1.76	1.56	1.36
75	2.31	2.11	1.92	1.72	1.52	1.33
80	2.27	2.08	1.88	1.68	1.49	1.29

From Troutbeck (1989)

Note:
The values of the follow-up headway are given to two decimal places to assist in interpolation. The adopted value may be rounded to one decimal place.

- Flows above about 1700 v.p.h are not applicable to single lane circulating carriageways. (Shaded area in table)
- The ratio of the critical acceptance gap to the follow-up headway (t_{ad}/t_{fd}) is given in Table VA3.2. The critical acceptance gap is the product of the appropriate values from Table VA3.1 and Table VA3.2.

Table VA3.2: Ratio of the Critical Acceptance Gap to the Follow-up Headway (t_{ad} / t_{fd})

Number of circulating lanes	one			more than one		
Average entry lane width (m)	3	4	5	3	4	5
Circulating flow (veh/h)						
0	2.32	1.98	1.64	2.04	1.70	1.36
200	2.26	1.92	1.58	1.98	1.64	1.30
400	2.19	1.85	1.52	1.92	1.58	1.24
600	2.13	1.79	1.45	1.85	1.51	1.18
800	2.07	1.73	1.39	1.79	1.45	1.11
1000	2.01	1.67	1.33	1.73	1.39	1.10
1200	1.94	1.60	1.26	1.67	1.33	1.10
1400	1.88	1.54	1.20	1.60	1.26	1.10
1600	1.82	1.48	1.14	1.54	1.20	1.10
1800				1.48	1.14	1.10
2000				1.41	1.10	1.10
2200				1.35	1.10	1.10
2400				1.29	1.10	1.10
2600				1.23	1.10	1.10

From Troutbeck (1989)

Note: Values of the ratio may be interpolated for intermediate widths of entry lane.

- For single lane circulating carriageways, if the critical gap calculated from Tables VA3.1 and VA3.2 is less than 2.1s, use 2.1s.
- For multi-lane circulating carriageways, the minimum value of critical gap should be 1.5s.

Table VA3.4: Sub-dominant Stream Follow-up Headway t_{fs}

Dominant stream follow-up headway t_{fd} (s)	Sub-dominant follow-up headway (t_{fs})(s) — Ratio of flows Dominant flow/Sub-dominant flow				
	1.0	1.5	2.0	2.5	3.0
1.5	2.05	1.99	1.94	1.89	1.84
1.6	2.10	2.07	2.05	2.02	1.99
1.7	2.15	2.15	2.15	2.15	2.15
1.8	2.20	2.23	2.25	2.28	2.30
1.9	2.25	2.30	2.35	2.40	2.46
2.0	2.30	2.38	2.46	2.53	2.61
2.1	2.35	2.46	2.56	2.66	2.76
2.2	2.41	2.53	2.66	2.79	2.92
2.3	2.46	2.61	2.76	2.92	3.07
2.4	2.51	2.69	2.87	3.05	3.23
2.5	2.56	2.76	2.97	3.17	3.38
2.6	2.61	2.84	3.07	3.30	3.53
2.7	2.70	2.92	3.17	3.43	3.69
2.8	2.80	3.00	3.28	3.56	3.84
2.9	2.90	3.07	3.38	3.69	4.00
3.0	3.00	3.15	3.48	3.82	4.15

From Troutbeck (1989)

Table VA3.3: Adjustment Times for the Dominant Stream Follow-up Headway

Number of circulating lanes	Number of entry lanes		
	1	2	3
1	0.00	- 0.39	—
2	0.39	0.00	- 0.39
3	—	0.39	0.00

Note: Add or subtract these factors from the initial values from Table VA3.1

The operation of the circulating stream also affects the average percentage of vehicles which are in bunches. As the flow increases, more vehicles are in bunches.

The proportion of bunched vehicles, (Θ), is evaluated from the circulating flow, the number of effective circulating lanes (characterised by the average headway between bunched vehicles) and the proximity of the roundabout to signalised intersections or other situations which increase bunching. Troutbeck (1989) gives equations for estimating the proportion of free vehicles, i.e. those not in bunches. Values for the proportion of bunched vehicles have been developed from these equations and the revised values are listed in Table VA3.6, Also see Akcelik and Troutbeck, 1991). It is suggested that the values given in this Table be then adjusted according to the proximity of the roundabout to nearby signalised intersections or other situations which will influence the approaching traffic conditions and the circulating flow at the roundabout. Values should be increased or decreased by no more than 0.2 based on judgement of the extent of bunching caused.

The proportion of bunched vehicles is expected to range from 0 for random traffic to about 0.8 for heavily platooned traffic. Values as high as 0.8 to 0.9 have been observed in extreme cases. This is equivalent to an average platoon length of 1 to about 3 or 4 vehicles in most conditions and up to 10 vehicles under the worst conditions.

Calculate Absorption Capacity and Degree of Saturation.

The absorption capacity of each entry lane is calculated from the entry lane gap acceptance parameters (t_a and t_f) applicable to the dominant lane and to each sub-dominant entry lane and the circulating flow characteristics (Q_c, τ, and Θ).

The appropriate equation is:

$$C = \frac{3600\,(1-\Theta)\,q_c\,e^{-\lambda(t_a-\tau)}}{1-e^{-\lambda t_f}}$$

Eq. 3.1

C = the absorption capacity of an entry lane in veh/h

Θ = the proportion of bunched vehicles in the circulating streams

q_c = the flow of vehicles in the circulating streams in veh/s

t_a = the critical acceptance gap relevant to the dominant or sub-dominant lanes respectively.

t_f = the follow on headway relevant to the dominant or sub-dominant lanes respectively.

τ = the minimum headway in the circulating streams, and these are related by:

$$\lambda = \frac{(1-\Theta)\,q_c}{1-\tau q_c}$$

Eq. 3.2

Note that the capacity predicted by Equation 3.1 is the expected steady-state capacity, or the maximum entry flow rate and it is not the "practical capacity". See discussion on degree of saturation below.

The above analysis method and equations are more comprehensive than may be necessary for some purposes. Figures VA3.5 and VA3.6 may be used to obtain a quick estimate for use in the planning and preliminary layout of a roundabout at a particular site.

Figure VA3.5 refers to a single lane roundabout with a 4m wide entry lane and one circulating lane. The results in Figure VA3.6 reflect the operating conditions of a roundabout with two 4m wide entry lanes and two circulating lanes. For very high circulating flows (exceeding about 1700 veh/h for single lane circulating flows, or about 3400 veh/h for multi-lane circulating flows), the entry capacities from Equation 3.1 approach zero. In such cases, a minimum entry capacity may be assumed.

Figure VA3.4: Location of Vehicles in a Wide Single Lane

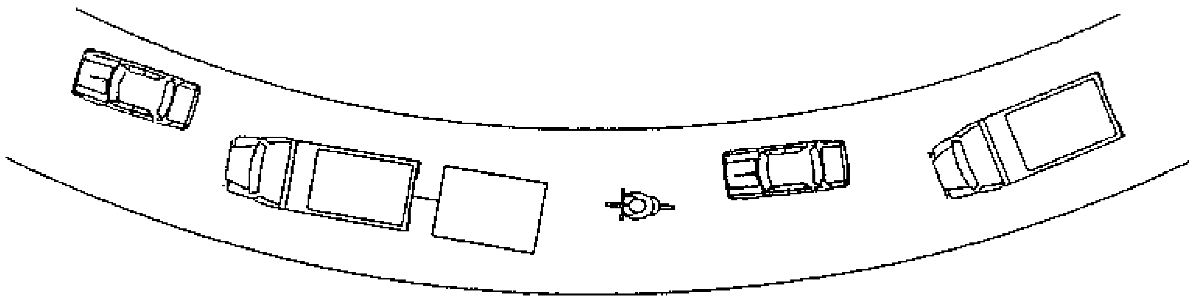


Table VA3.5: Average headway between bunched vehicles in the circulating traffic and the number of effective lanes in the circulating carriageway.

	Circulating Carriageway Width			
	less than 10 m		greater than or equal to 10 m	
	Number of effective lanes	Headway between bunched vehicles (τ) (s)	Number of effective lanes	Headway between bunched vehicles (τ) (s)
Circulating Flow <1000 veh/h	1	2	2	1
>1000 veh/h	1 (or 2)	2 (or 1)	2	1

Calculate Degree of Saturation

The degree of saturation of an entry lane is the arrival flow divided by the entry (absorption) capacity of the lane:

$$x = \frac{Q_m}{C} \qquad \text{Eq. 3.3}$$

Where

Q_m = entry lane arrival flow in veh/h, and

C = entry lane capacity in veh/h (from Equation 3.1 or Figures VA3.5 and VA3.6)

The degree of saturation during the design period for an entry lane should be less than about 0.8 to 0.9 for satisfactory operation, although this may not be always practicable.

Within this range of degree of saturation, designers should consider using the delays as a more appropriate measure of performance.

The maximum (practical) degree of saturation corresponds to the concept of "practical capacity". For example, if practical degree of saturation (x) is 0.85, practical capacity is 0.85 C, where C is the entry capacity from Equation 3.1. The practical degree of saturation is also used for "spare capacity" calculations.

Judgement may be exercised in the assessment of the acceptability of the degree of saturation or delays taking into consideration factors such as environment, locality, possible alternative intersection treatments, cost and the period that the roundabout can be expected to have less satisfactory performance characteristics than an alternative treatment.

Table VA3.6: Proportion of Bunched Vehicles

Number of effective circulating lanes	one	more than one
Average headway between bunched vehicles, τ, (s)	2.0	1.0
Circulating flow (veh/h)		
0	0.250	0.250
300	0.375	0.313
600	0.500	0.375
900	0.625	0.438
1200	0.750	0.500
1500	0.875	0.563
1800	1.000	0.625
2000		0.667
2200		0.708
2400		0.750
2600		0.792

Figure VA3.5: Entry capacity for a single lane roundabout with a 4 m wide entry lane and one circulating lane.

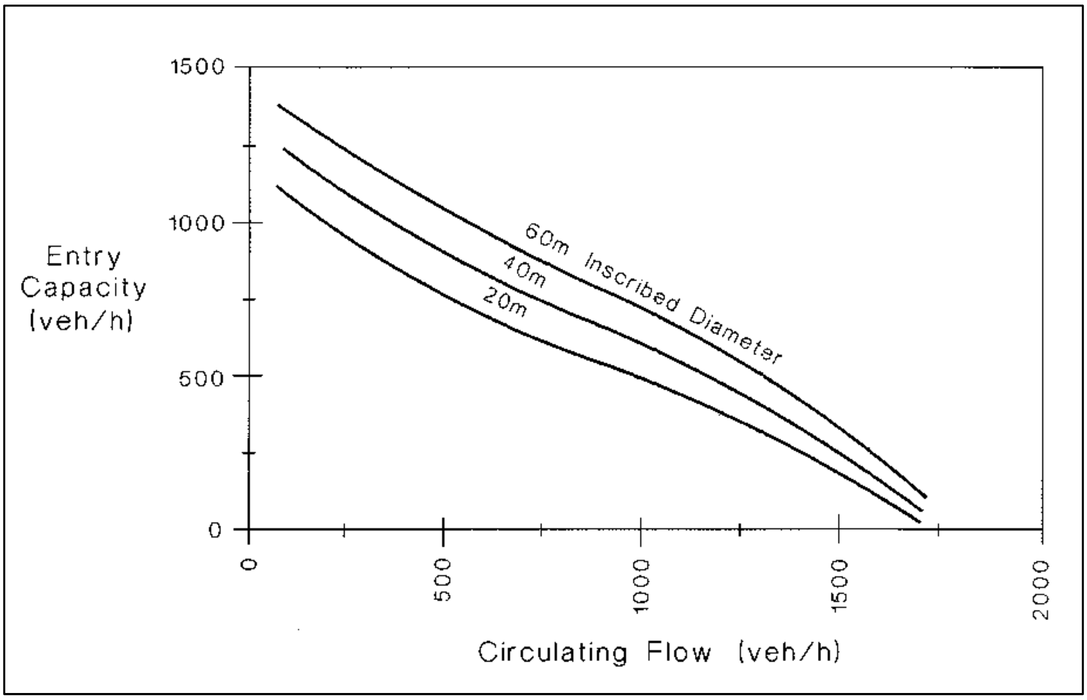
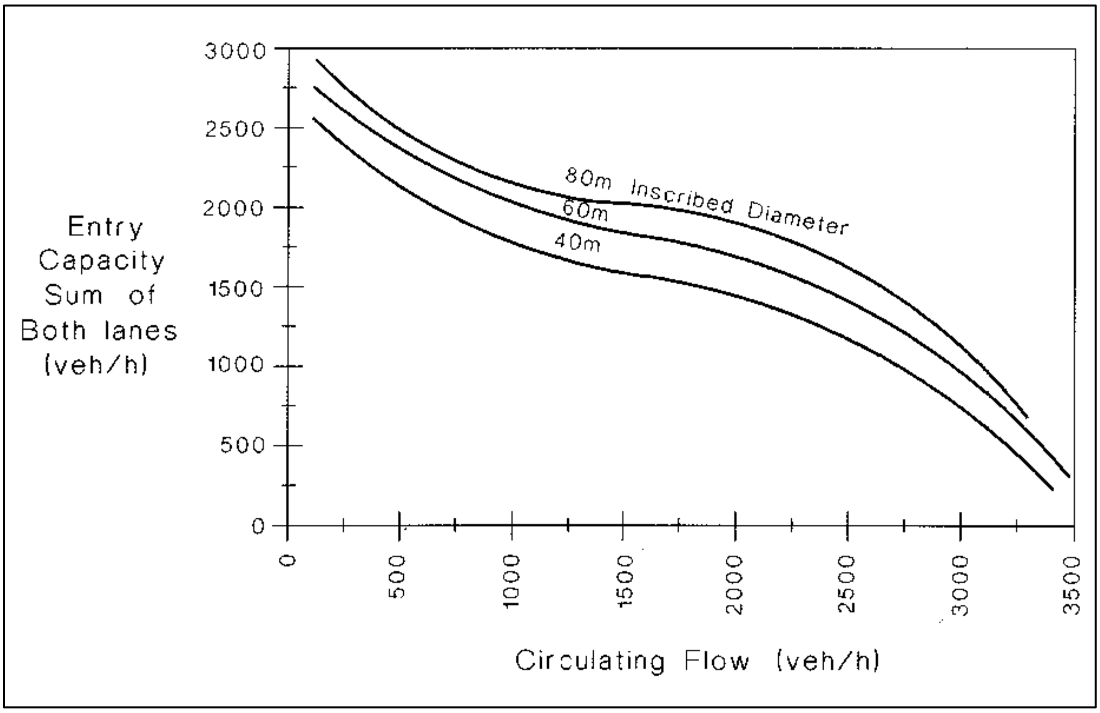


Figure VA3.6: Entry capacity for a roundabout with two 4m wide entry lanes and two circulating lanes.



Delays at Roundabouts

There are two components of the delays experienced at roundabouts, namely queuing and geometric delay. Queuing delay is the delay to drivers waiting to accept a gap in the circulating traffic. Geometric delay is:

- The delay to drivers slowing down to the negotiation speed, proceeding through the roundabout and then accelerating back to normal operating speed; or
- The delay to drivers slowing down to stop at the end of the queue and, after accepting a gap, accelerating to the negotiation speed, proceeding through the roundabout and then finally accelerating further to reach normal operating speed. It excludes the time to wait for an acceptable gap.

In some instances it may be appropriate to consider only the queuing delay, e.g. when approximate results only are required, or when making a comparison with a 'STOP' or 'GIVE WAY' controlled approach at an intersection. In these cases, the geometric delay for traffic entering from the side (controlled) road approach would experience about the same geometric delay as at a roundabout. In most cases it may be desirable to consider the total delay e.g. when the results are required for a comparison with traffic signals or in an economic analysis. Total delay is the sum of the queuing delay and the geometric delay.

Queuing Delay

To calculate the average queuing delay, first calculate the minimum delay for the conditions when the entering traffic flow is very low using:

$$w_h = \frac{e^{\lambda(t_a - \tau)}}{(1-\Theta)q_c} - t_a - \frac{1}{\lambda} + \frac{\lambda\tau^2 - 2\tau\Theta}{2(\lambda\tau + 1 - \Theta)}$$

Eq. 3.4

where the gap acceptance parameters t_a, τ, Θ and λ . are as in Equation 3.1 and the circulating flow q_c is in veh/sec. For all practical purposes the average queuing delay per vehicle is given by:

$$w_m = w_h + 900\,T\,[\,Z + \sqrt{Z^2 + \frac{mx}{CT}}\,]$$

Eq. 3.5

where:

w_m = average delay per vehicle in seconds

w_h = minimum delay in seconds when the entering traffic is very low (from Equation 3.4)

T = duration of the flow period in hours, i.e. the time interval during which an average arrival demand Qm persists (use 1h or 0.5h)

Z= x-1

x = degree of saturation of the entry lane (= Qm/C as in Equation 3.3)

C = entry lane capacity in vehicles per hour

m = a delay parameter given by

$m = w_h\ C/450$

The second term of Equation 3.5 accounts for the queuing delays due to the presence of a queue in the entry lane. Equation 3.5 is a time-dependent formula (Akcelik 1991, Akcelik and Troutbeck 1991) derived from the steady-state formula given by Troutbeck (1989). It is applicable for near-capacity and oversaturated conditions. The flow period parameter becomes important for high degrees of saturation, i.e. the delays are insensitive to the flow period for low degrees of saturation.

Geometric Delay

The geometric delay for vehicles differs depending on whether the vehicles have to stop or not. George (1982) developed a method for calculating the average geometric delays as follows:

Average geometric delay:

$dg = P_s\ d_s + (1-P_s)d_u$

where:

P_s = the proportion of entering vehicles which must stop,

d_s = the geometric delay to vehicles which must stop,

$(1-P_s)$ = the proportion of entering vehicles which need not stop,

d_u = is the geometric delay to vehicles which need not stop This equation has been also documented by Middleton (1990).

The proportion of entering vehicles which must stop, Ps, can be estimated using Figures VA3.7 and VA3.8 depending on the number of circulating lanes. This proportion depends on the entry and circulating lane flows. Increase either of these flows and the proportion of entering drivers stopped will increase. The near linear lines in these Figures result from the gap acceptance parameters and the level of bunching in the circulating stream being a function of the circulating flow. Tables 3.7(a) and (b) have been developed to allow d_s and d_u to be estimated. These enable the geometric delay to be calculated for each approach to a roundabout. Geometric delay is different for each traffic movement - left turn, right turn and straight on, at each approach and each should be calculated separately.

Figure VA3.7: Proportion of vehicles stopped on a single lane roundabout.

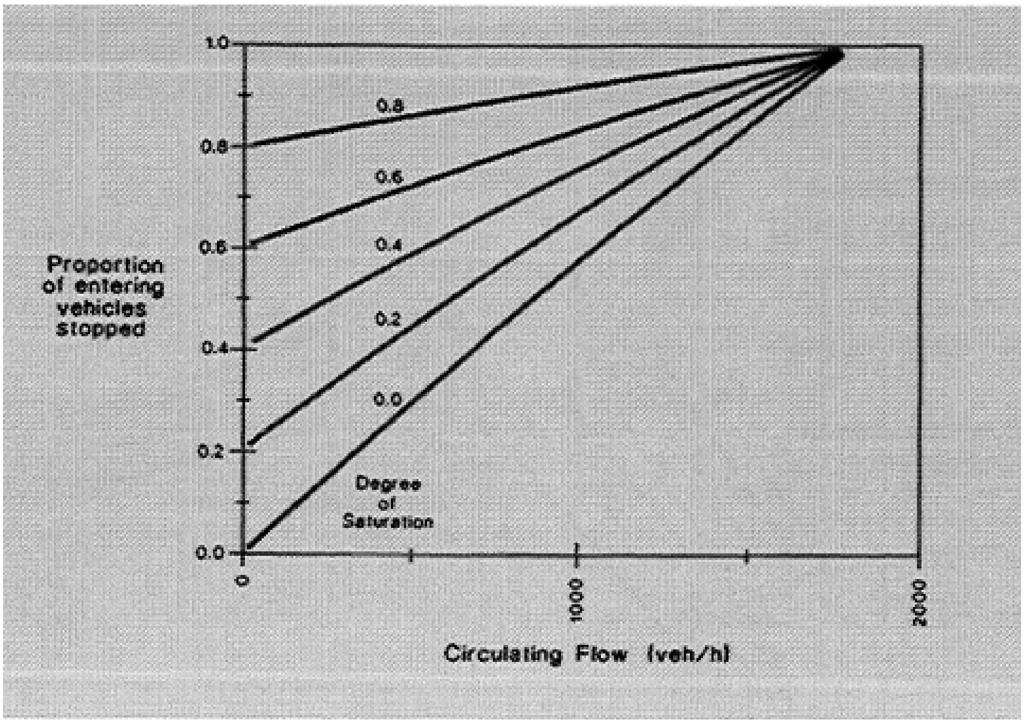


Figure VA3.8: Proportion of vehicles stopped on a multi-lane entry roundabout.

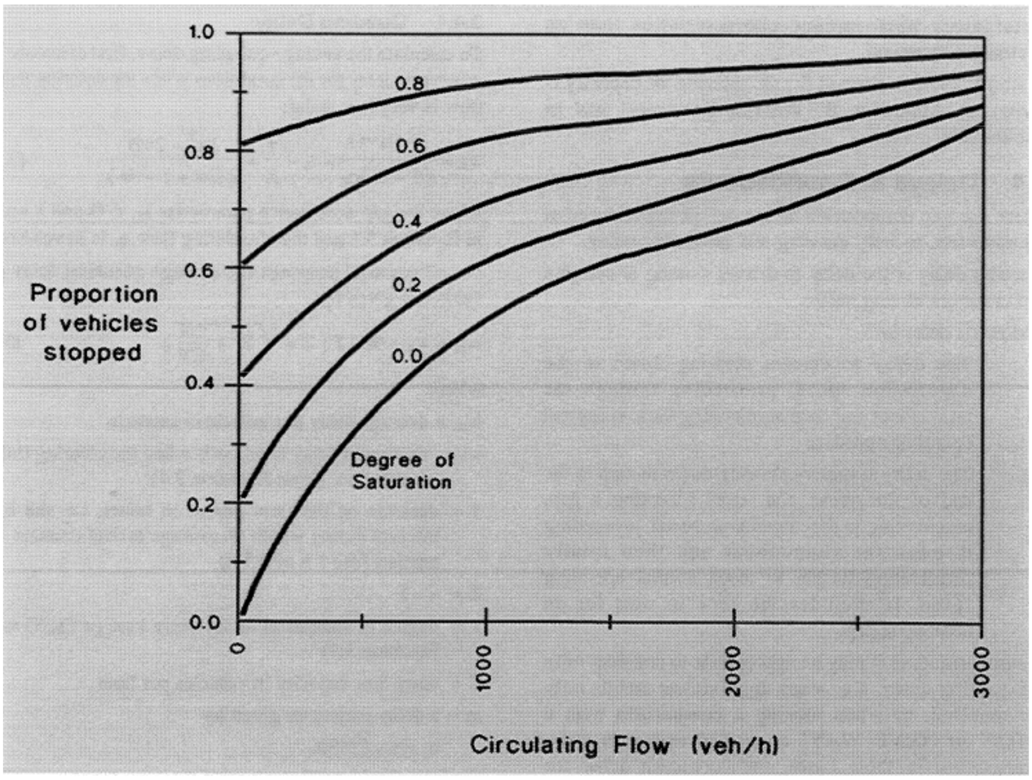


Table VA3.7(a): Geometric delay for stopped vehicles (seconds per vehicle).

Approach speed V_a (km/h)	Distance* around roundabout D (m)	Negotiation speed through roundabout V_n, (km/h)							
		15	20	25	30	35	40	45	50
40	20	10	8	7	7	7			
40	60	19	15	12	9	7			
40	100		22	17	13	10			
40	140				18	14			
40	180					18			
60	20	13	11	10	10	10	10	10	10
60	60	23	18	15	13	10	10	10	10
60	100		26	21	18	15	12	10	10
60	140				22	19	15	12	10
60	180					23	19	15	10
80	20	17	15	13	13	13	13	13	13
80	60	26	22	19	17	14	13	13	13
80	100		29	25	21	19	16	13	13
80	140				26	23	19	16	13
80	180					27	23	19	16
100	20	20	18	17	17	17	17	17	17
100	60	30	25	22	20	18	17	17	17
100	100		33	28	25	22	20	17	17
100	140				30	26	23	20	17
100	180					30	27	24	20

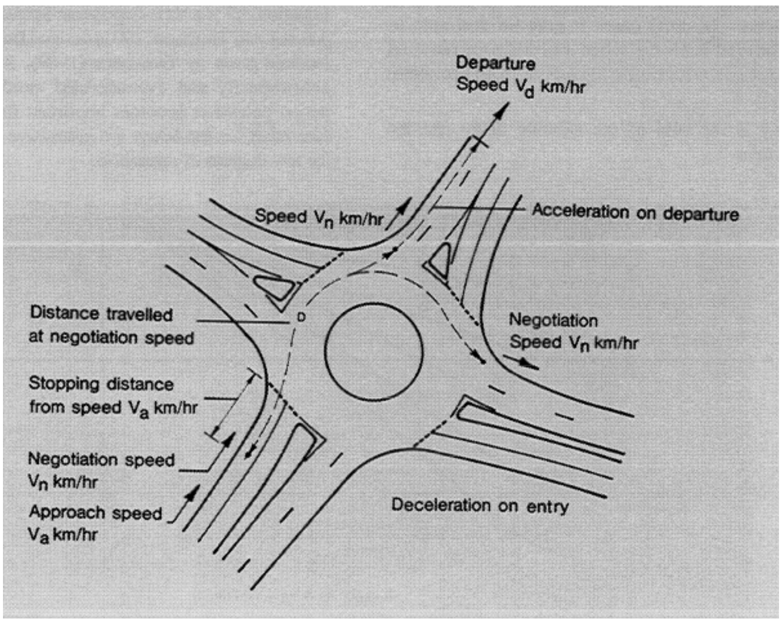
* Refer to Figure 3.9 for the definitions of the dimensions.

Table VA3.7(b): Geometric delay for vehicles which do not stop (seconds per vehicle).

Approach speed V_a (km/h)	Distance* around roundabout D (m)	Negotiation speed through roundabout V_n, (km/h)							
		15	20	25	30	35	40	45	50
40	20	7	4	2	1	0			
40	60	17	11	7	4	0			
40	100		19	13	8	4			
40	140				13	8			
40	180					12			
60	20	11	8	5	4	3	2	1	1
60	60	20	15	11	8	4	2	1	1
60	100		22	17	13	9	5	1	1
60	140				17	13	8	4	1
60	180					17	12	7	2
80	20	14	11	9	7	6	5	4	3
80	60	24	19	15	11	8	5	4	3
80	100		26	20	16	13	9	5	3
80	140				21	17	13	9	4
80	180					21	16	12	7
100	20	18	15	12	10	9	8	7	6
100	60	27	22	18	15	12	9	7	6
100	100		29	24	20	16	13	10	6
100	140				25	20	17	13	12
100	180					25	20	16	

* Refer to Figure 3.9 for the definitions of the dimensions.

Figure VA3.9: Definition of the terms used in Tables VA3.7(a) and VA3.7(b).



Appendix VB

(from GTEP Part 6: Roundabouts, Appendix B – Worked Examples)

B.1 Roundabout Analysis for Urban Arterial Roads

B.1.1 Scenario

The intersection of Miller Road and Tahiti Street is located in the business area of a country town. Shops are located on all four corners and there are significant pedestrian movements through the daytime. There is currently 45° angle parking on both streets, and the parking spaces are fully utilised. "Give Way" signs control traffic in Tahiti Street.

Traffic volumes are relatively consistent through the weekday and the critical peak hourly volumes are detailed below.

Vehicle delays in Tahiti Street are reasonably high, and the intersection has a poor accident history. Many of the reported accidents were right angle accidents, but few of the accidents involved pedestrians. Members of the local community have proposed traffic signals for the site as a means of reducing delays and improving safety.

The task is to examine the options available for improving the performance of the intersection in terms of the following objectives:

- maximise safety for vehicles and pedestrians,
- provide adequate capacity and minimise delays,
- minimise costs,
- maximise car parking spaces.

B.1.2 Consideration of Alternatives

Traffic signals or a roundabout can be considered as alternatives to the existing arrangement. Both treatments should be analysed and their capacity and delays, parking space changes and costs should be compared. The existing safety record should also be reviewed. These analyses and experience with the alternative treatments in similar situations should be used as a basis for selecting the most appropriate treatment.

In this instance only the analysis of the roundabout is provided in detail. Analysis procedures for a traffic signal alternative may be carried out in also.

Roundabout Alternative

The critical peak hour traffic volumes were transcribed by totalling the entry flow, the exiting flow and the circulating flow. The circulating flow applicable to a particular entry does not include the vehicles exiting upstream of that entry, i.e. it only includes the circulating vehicles that continue past the splitter island at that entry. Using the data in Figure VB1 the controlling roundabout flows are as shown in Figure VB2.

The required numbers of entry and circulating lanes are obtained from Figure VB3. For this example all points given by plotting the circulating flow against the entry flow are below the first grey area, thus a single lane roundabout will be suitable.

In this example the basis of the calculations for the north leg (Miller Road) only will be explained. The values for the other approaches will be calculated in a similar fashion.

Given the geometric layout of the roads and width of reservation available at this location, an inscribed diameter of 32m would be suitable, and average entry lane widths of 4m are proposed. It should be noted that only estimated measurements are required at this stage and they may be adjusted later if necessary. These measurements are shown on Figure VB7.

Figure VB1: Turning Movements

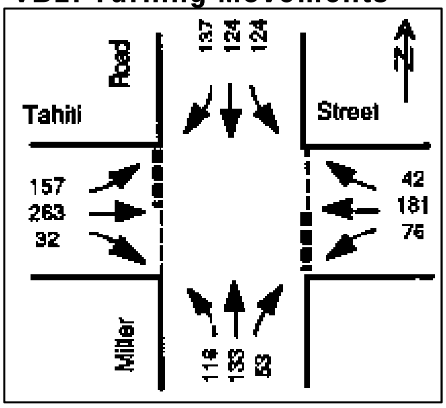
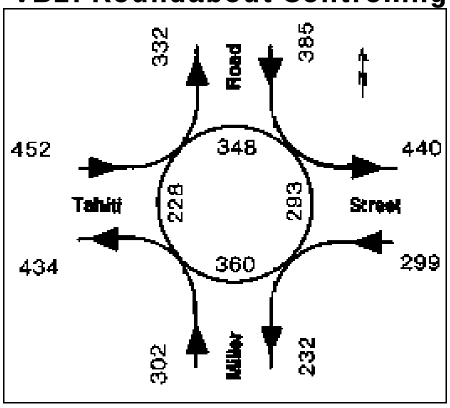


Figure VB2: Roundabout Controlling Flows



Follow-up Headway and Critical Acceptance Gap Values

As there is only one entry stream from the North, this entry stream will be used as a 'dominant' one. The drivers in this stream will have a follow-up time of 2.7s. This is obtained from interpolating between the values in Table VA3.1 (in Appendix VA of this Supplement). The ratio of the critical acceptance gap to the follow-up headway is then 1.85 giving a critical acceptance gap of 1.85 x 2.7 or 5.0s.

Circulating Flow Characteristics and Entry Lane Capacity

The circulating carriageway width is less than 10m, and it is considered to operate as a single lane roundabout. The average headway between the bunched vehicles will then be 2s with about 40 per cent of vehicles in bunches (or following others). These values are given in Table VA3.6 (in Appendix VA of this Supplement).

Using these values then:

$$\Theta = 0.40$$
$$Q_c = 348 \text{ veh/h}$$
$$q_c = 0.0967 \text{ veh/s}$$
$$t_a = 5.0 \text{ s}$$
$$t_f = 2.7 \text{ s}$$
$$\tau = 2 \text{ s}$$

gives a value of λ equal to 0.0719 (from Equation 3.2) and the absorption capacity C, given by Equation 3.1 is 954 veh/h. The degree of saturation is the entry flow divided by the capacity or 385/954 or 0.4.

Queuing Delay

The average queuing delay per vehicle at very low entry flows, w_h, calculated from Equation 3.4 is 1.6s and the total average queuing delay per vehicle, w_m, from Equation 3.5 is 2.7s.

Geometric Delay

To estimate the geometric delay, the proportion of stopped vehicles is estimated to be 0.55 from Figure VA3.7 (in Appendix VA of this Supplement). Given the geometry of the roundabout, the distance D, travelled by the vehicle when turning left, is 30 in as defined in Figure VA3.9 (in Appendix VA of this Supplement). The distance D is 55 in for a straight through and 70 in for a right turn movement respectively. The stopping distance from 30 km/h is close to 20 in. Using Tables VA3.7(a) and 3.7(b) (in Appendix VA of this Supplement), the geometric delay values for vehicles from this northern approach are as follows.

Movement	Geometric delay for stopped vehs (s)	Geometric delay for vehicles not stopped (s)
left	10.8	5.0
through	12.6	7.5
right	14.3	9.3

Since 55 per cent of the vehicles are stopped, then the average geometric delay, for say the left turners, is 10.8 x 0.55 + 5.0 x 0.45 or 8.2s. The geometric delays for the other users are:

Movement	Turning volumes (veh/h)	Geometric delay for movement (s)
left	124	8.2
through	124	10.3
right	137	12.0

The total geometric delay for the approach is calculated from the movement volumes and the delays. For this approach, it is:

$$\frac{124 \times 8.2 + 124 \times 10.3 + 12.0 \times 137}{124 + 124 + 137} = 10.2 \text{ s}$$

This geometric delay should then be added to the queuing delay to give the total average delay for the approach. A summary of the results for this roundabout is in the table below:

Leg	Entry Flow (veh/h)	Entry Capacity (veh/h)	Degree of Satn.	Queueing Delay (s)	Geometric Delay (s)	Delay for Approach (s)
South	302	941	0.32	2.5	9.3	11.8
East	299	999	0.30	1.9	9.5	11.4
North	385	954	0.40	2.7	10.2	12.9
West	452	1069	0.42	1.7	9.4	11.1

The above table illustrates that a one-lane roundabout could easily cater for the traffic volumes. The highest degree of saturation is 0.42 and accordingly the average delays would be reasonable.

Safety

A roundabout could be expected to reduce the accident rate significantly at the intersection. Pedestrian safety would be expected to be also satisfactory since the roundabout would give improved conditions for safe crossing compared with the existing layout.

Alternative Analysis Methods

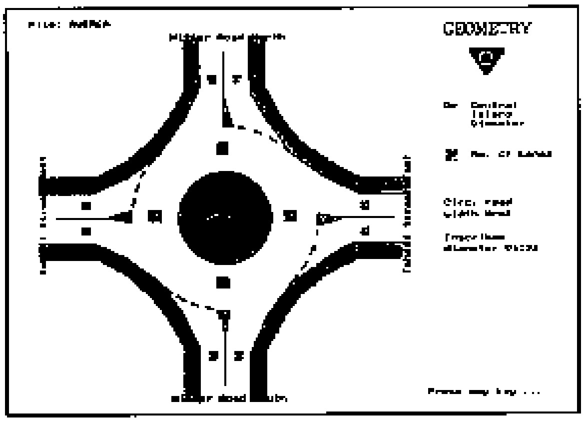
Example B1 has also been analysed using the SIDRA 4 package (Figures VB3 to VB5).

Graphical representation of SIDRA input data for Example B1 is shown in Figure B3 which includes the screen prints of roundabout geometry and volume data pictures and the roundabout approach data screen (from SIDRA input program RIDES). Figure VB4 presents a sample of SIDRA output for Example VB1. The capacity and average queuing delay results from SIDRA are in accordance with the formulae given in this guide (a flow period of one hour was used for performance calculations). Figure VB5 presents the spare capacity and delay graphs obtained using the variable flow scale facility of SIDRA (screen prints form SIDRA graphics output program GOSID). The spare capacities in Figure VB5 are based on a practical degree of saturation of 0.85.

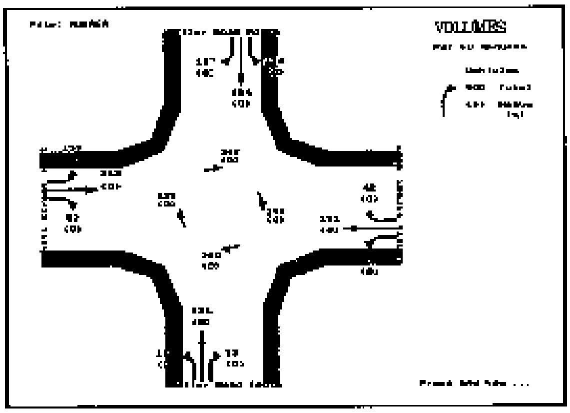
The results in Figure VB5 indicate that substantial decreases in spare capacity and increases in average queuing delay would occur with increasing entry flows (and hence, decreased capacities). By definition, the point where the degree of saturation equals 0.85 represents the zero spare capacity conditions (marked on the graph). This corresponds to a flow scale of 175 per cent, i.e. the practical capacity will be reached when the existing flows increase by 75 per cent. At this point, the average queuing delay is low (12.3s) indicating that a higher degree of saturation could be tolerated. The effect of increased entry and circulating flows on average delay for the intersection is seen to be non-linear with a sharp increase as the arrival flows approach capacity (at flow scale of 195 per cent). The average queuing delay at capacity, i.e. when the degree of saturation equals 1.0, is 44.0s.

Figure VB3: SIDRA Roundabout geometry and volume data pictures and approach data screen for Example VB1.

(a) Roundabout geometry



(b) Volumes



(c) Roundabout data for South approach

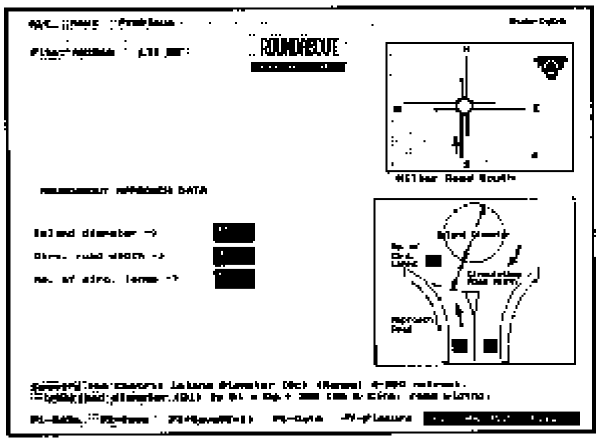


Figure VB4: SIDRA results for Example VB1

A R R B - S I D R A 4.06B

Australian Road Research Board
R. AKCELIK Registered User No. 1
Time and Date of Analysis 11:30 AM, 20 May 1992

AUSTROADS Roundabout Guide (1992), Example B.1 (App.B) * AUSR6A *
Single-lane roundabout
Intersection No.: AUS6A
Roundabout

Table R.1 - ROUNDABOUT BASIC PARAMETERS

Appr	Cent Island Diam (m)	Circ Width (m)	Insc Diam. (m)	No.of Circ. Lanes	No.of Entry Lanes	Av.Ent Lane Width (m)	Prop Free Vehs	Crit Gap (s)	Foll On Time (s)	Circ Flow (pcu /h)
Miller Road South										
S	16	8	32	1	1	4.00	.600	4.94	2.65	360
Tahiti Street East										
E	16	8	32	1	1	4.00	.628	5.05	2.67	293
Miller Road North										
N	16	8	32	1	1	4.00	.605	4.96	2.65	348
Tahiti Street West										
W	16	8	32	1	1	4.00	.655	5.15	2.70	228

Table S.6 - INTERSECTION PERFORMANCE

Total Flow (veh/h)	Total Delay (veh-h/h)	Aver. Delay (sec)	Total Stops (veh/h)	Stop Rate	Perf. Index	Aver. Speed (km/h)	FUEL Rate (mL/km)	FUEL Total (mL/h)
1438	.85	2.1	583	.41	30.44	57.9	88.8	127648.8

Table S.10 - MOVEMENT CAPACITY AND PERFORMANCE SUMMARY

Mov No.	Mov Typ	Arv Flow (veh /h)	Total Cap. (veh /h)	Lane Util (%)	Deg. Satn x	Aver. Delay (sec)	Stop Rate	Longest Queue (veh)	Perf. Index
Miller Road South									
1	LTR	302	952	100	.314	2.4	.42	.6	6.58
Tahiti Street East									
2	LTR	299	1015	100	.295	1.9	.37	.5	6.27
Miller Road North									
3	LTR	385	971	100	.396	2.6	.45	.7	8.37
Tahiti Street West									
4	LTR	452	1069	100	.423*	1.7	.38	.5	9.23

* Maximum degree of saturation
Stop Rate is proportion of stopped vehicles including queueing effects.
Geometric delays not included.

Figure VB5: SIDRA results showing the changes in spare capacity and average delay with increasing entry flows (hence increased circulating flows) for Example VB1

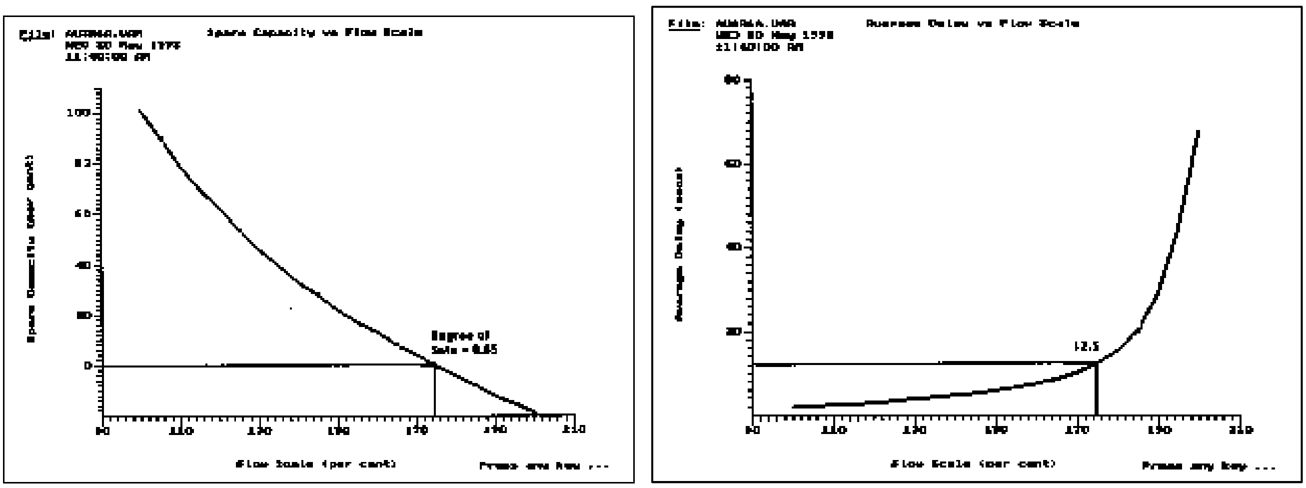
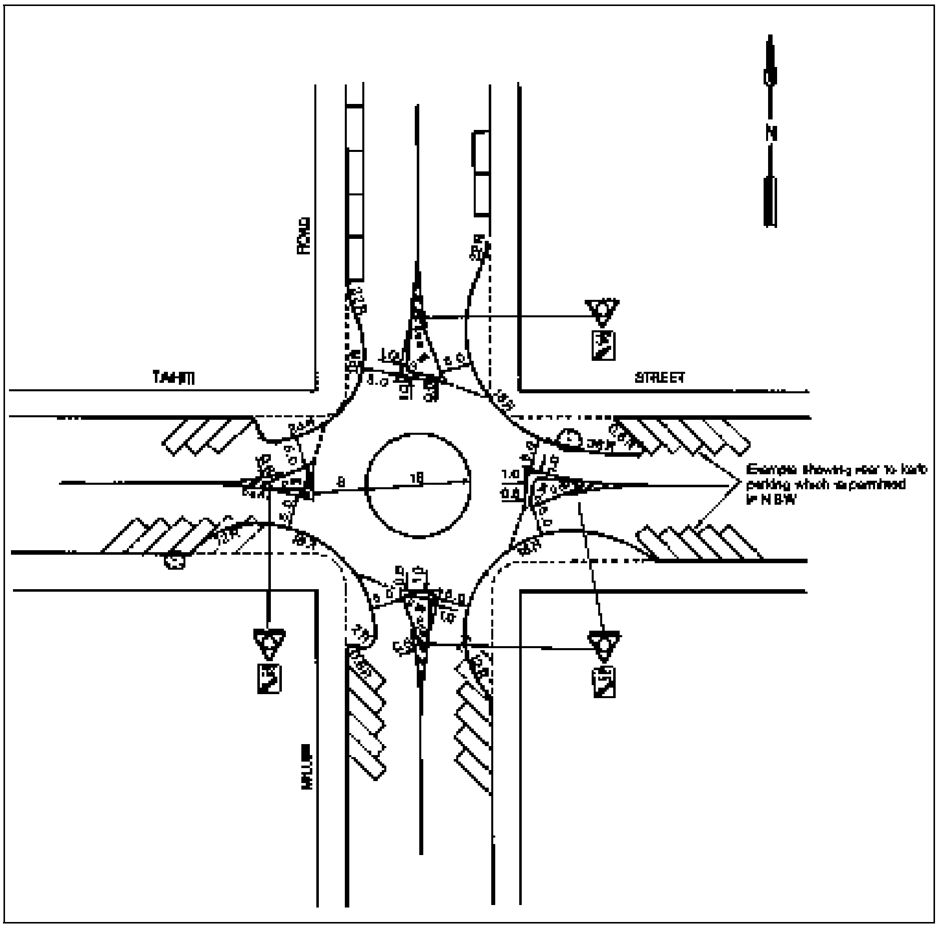


Figure VB7: Example of Urban Roundabout Design



B.1.3 Layout Design for Example VB1

The layout design for the roundabout is shown on Figure VB7. The chosen design has the following features:

- Adequate deflection is achieved.
- The circulating carriageway is sufficiently wide to cater for right turning semi-trailers.
- No new pavement is required.
- Only six car parking spaces are lost, although the angled parking is closer to the intersection than desirable.
- pedestrians have a much shorter crossing distance, and can stage their crossing.
- Additional footpath areas become available for landscaping.

The layout would reduce through vehicle speeds.

In comparing the design with a signalised layout, the roundabout was found to be cheaper with regard to both installation and operating costs. The roundabout also allowed for more parking to be retained.

B.1.4 Conclusion

It was concluded that modifications to the existing layout were warranted, and that the roundabout should be adopted as it could be expected to operate exceptionally well with regard to:(a) safety, (b) delays, (c) capacity, (d) cost and (e) effect on parking.

B.2 Analysis of a Multi-lane Roundabout

The analysis of a multi-lane roundabout is similar to the analysis of a single lane roundabout. The traffic volumes used in this example are based on the following turning movements.

Movement	Approach Leg			
	North	East	South	West
Left	132	76	86	63
Through	782	689	651	571
Right	237	212	179	162

The roundabout flows are shown in Figure VB8. The calculations for this roundabout are given in the spreadsheet in Section 11. Only the analysis of the North leg will be described here. The circulating flow is 912 veh/h and the entry flow is 1151 veh/h. Figure VA3.3 (in Appendix VA of this Supplement) indicates that a two lane roundabout would have a suitable traffic performance.

Entry Geometric Properties

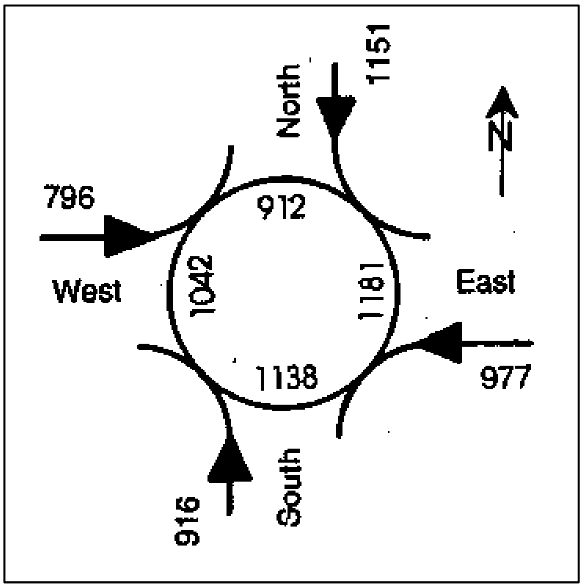
The geometric properties of the roundabout would normally be taken from a trial layout. Here, they are assumed to be as follows:

Inscribed Diameter	Di	= 50 m
Average Entry Lane Width	w_e	= 4 m
Approach speed	V_a	= 60 km/h
Negotiation speed	V_n	= 30 km/h
Number of entry lanes	n_e	= 2
Number of circulating lanes	n_c	= 2
Left-turn distance		= 40 m
Straight through distance		= 60 m
Right-turn distance		= 80 m

Entry Lane Flows

The entry lanes flows are a function of the turning volumes and the proportion of through traffic in each entry lane. Here it is assumed that the left lane will carry, the traffic turning left and half the through traffic. Using the figures for the North entry, the left lane traffic flow would be 132 ± 782/2 or 523 veh/h. Similarly the right hand lane flow is 237 + 782/2 or 628 veh/h. At this approach the right hand lane has the greater flow and will be the dominant stream. The left hand lane will be the sub-dominant stream.

Figure VB8: Roundabout flows for a multi-lane roundabout.



Critical Gap Parameters

The critical gap parameters are evaluated for each entry lane. The dominant stream follow-up time is given by Table VA3.1 (in Appendix VA of this Supplement). For a circulating flow of 912 veh/h and an inscribed diameter of 50 m, the dominant stream follow-up time is 2.18s. As the number of entry lanes equals the number of circulating lanes there is no need to use Table VA3.3 (in Appendix VA of this Supplement) in this example. Table VA3.4 (in Appendix VA of this Supplement)gives the sub-dominant stream follow-up time. The ratio of the dominant entry lane flow to the sub-dominant entry lane flow is equal to 628/523 or 1.20. Using this value and the dominant stream follow-up time, the sub-dominant stream follow-up time is 2.44s. The critical acceptance gaps for both entry streams are evaluated from the ratio of the critical acceptance gap to the follow-up time. Table 3.2 indicates that this ratio is 1.42. The critical acceptance gap for the dominant stream is then 1.42 x 2.18 or 3.09s. The sub-dominant stream critical acceptance gap is then 1.42 x 2.44 or 3.46s. The gap acceptance values for the sub-dominant stream are always greater then the dominant stream values.

Circulating Flow Characteristics

The circulating flow has an average headway between the bunch vehicles of 1s (from Table VA3.5 in Appendix VA of this Supplement). The proportion of bunched vehicles is expected to be 44 per cent. As this roundabout is near a signalised intersection which will tend to bunch up the vehicles a little more, it was decided to increase the bunching by an extra 10 per cent.

Entry Lane Capacities

The capacity of the dominant stream uses the dominant stream gap acceptance parameters. Using Equation 3.1 with

	Θ	= 0.54
	Q_c	= 912 veh/h
	q_c	= 0.2533 veh/s
	T_{ad}	= 3.09 s
	T_{fd}	= 2.18 s
and	τ	= 1 s

The entry capacity for the dominant stream lane is 1050 veh/h.

For the sub-dominant stream, the gap acceptance and circulating stream parameters are

	Θ	= 0.54
	Q_c	= 912 veh/h
	q_c	= 0.2533 veh/s
	T_{as}	= 3.46 s
	T_{fs}	= 2.44 s
and	τ	= 1 s

The capacity for the sub-dominant stream is then 901 veh/h. The degree of saturation for the dominant stream is 0.60 whereas the dominant stream degree of saturation is 0.58. The total entry capacity is not the sum of the capacities of the individual entry lanes unless the degree of saturation in each lane is the same. However given that the degree of saturation for the two entry lanes are approximately the same, a satisfactory practical total entry capacity is given by the sum of the two lane capacities. The approach degree of saturation is then the total entry flow divided by the total entry capacity. Here the approach degree of saturation is 1151/1941 or 0.59.

Queuing Delay

Again using the gap acceptance parameters for each entry lane and the circulating flow parameters, the average queuing delay per vehicle can be calculated using Equations 3.4 to 3.6. Here the average queuing delays per vehicle are 4.08 and 4.73s for the dominant and the sub-dominant streams respectively. The approach queuing delay is then related to the lane flows and is

$$\frac{4.08 \times 628 + 4.73 \times 523}{628 + 523}$$

or

$$4.37 \text{ s}$$

Geometric Delay and Total Delay

The geometric delay is calculated as shown above. In this example, the total geometric delays are 10.4s for the left turners, 12.8s for the drivers proceeding straight on and 15.2s for the right turners. The total delay per vehicle for a movement is then the sum of the geometric delay per vehicle and the total average queuing delay per vehicle. The average delay per vehicle on the approach is the weighted average of these total movement delays and is 17.4s.

B.3 Analysis and Design of a Roundabout on a Rural Arterial Road

B.3.1 Scenario

The intersection of Main Road and Montpelier Road is located about 5 km from the urban development of Troutsville. The cross intersection is in a purely rural area and traffic speeds on all approaches are about 100 km/h. The topography of the area is flat. Peak hour flow rates are shown in Figure B9. The intersection has an extremely high incidence of accidents as is illustrated on the collision diagram (Figure VB10).

It is desired to improve safety at the intersection, bearing in mind the need to also:

- provide adequate capacity and minimise delays;
- minimise costs;
- provided for over-dimensional vehicle movements.

B.3.2 Consideration of Alternatives

The alternatives of a roundabout, signalisation and staggered-T intersection were considered, and compared with the criteria outlined above.

Roundabout Alternative

The critical peak hour traffic volumes were transcribed in Figure VB7.

The degree of saturation and average queuing delays for each leg of a one-lane roundabout were calculated and are given above.

The average queuing delay per vehicle on each approach (a.m. and p.m.) would be less than 2 seconds. These tables indicate that a one-lane roundabout could easily cater for the traffic volumes. A roundabout could also be expected to reduce the accident rate significantly at the intersection.

Other Alternatives

Calculations of capacity and delay should be made for staggered-T treatment. These results along with an assessment of other relevant factors indicated in Section VB3.2 can be included in a detailed comparison of all alternatives.

B.3.3 Roundabout Design

The design for the roundabout is shown in Figure VB12. Significant features of the plan are:

- adequate deflection is achieved;
- the circulating carriageway would allow semi trailers to make all movements;
- special provision is made for over-dimensional vehicles;
- long splitter islands warn drivers to slow down;
- no land acquisition would be required.

B.3.4 Conclusion

It was concluded that the most effective way of reducing the number of accidents was to introduce either a roundabout or a staggered-T intersection. As it is difficult to predict which would be the safer treatment, the choice could be based on cost.

Figure VB9: Peak Hour Flow Rates

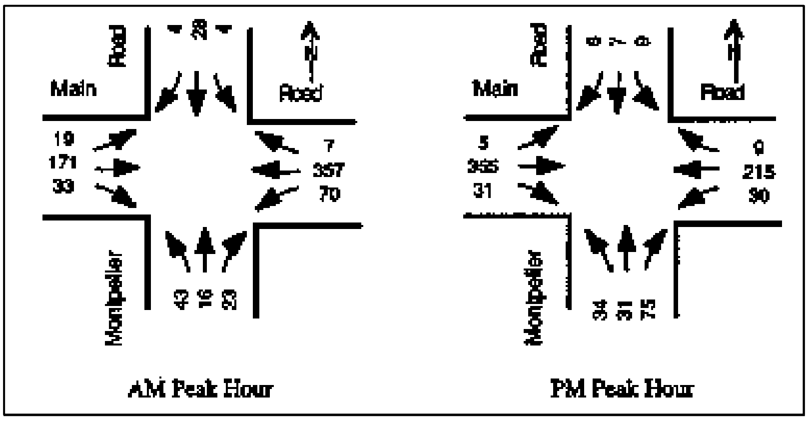
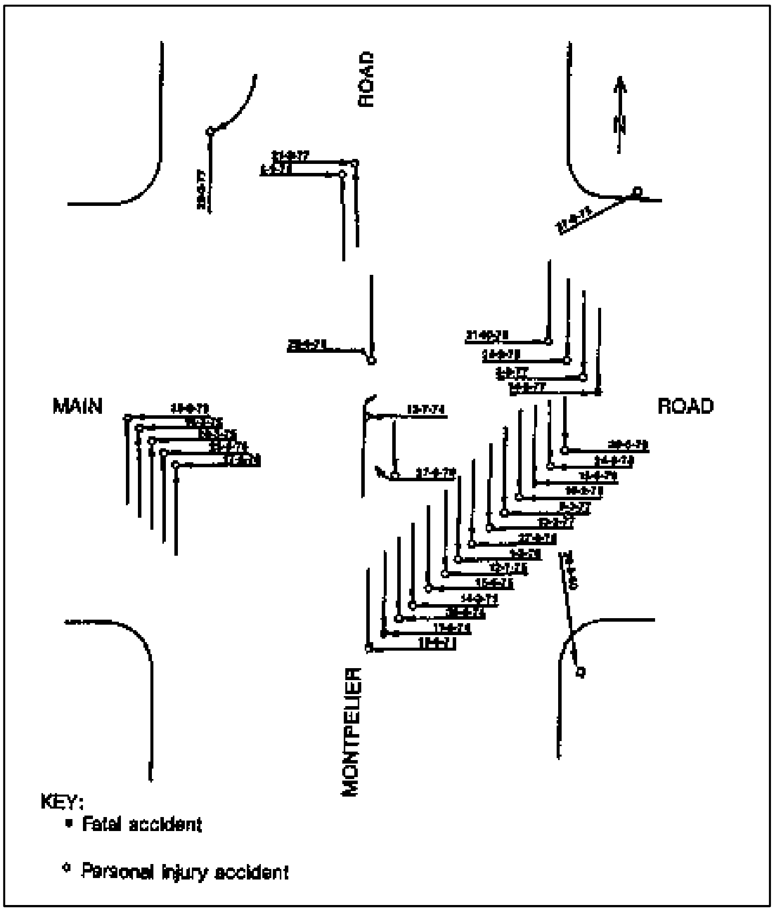
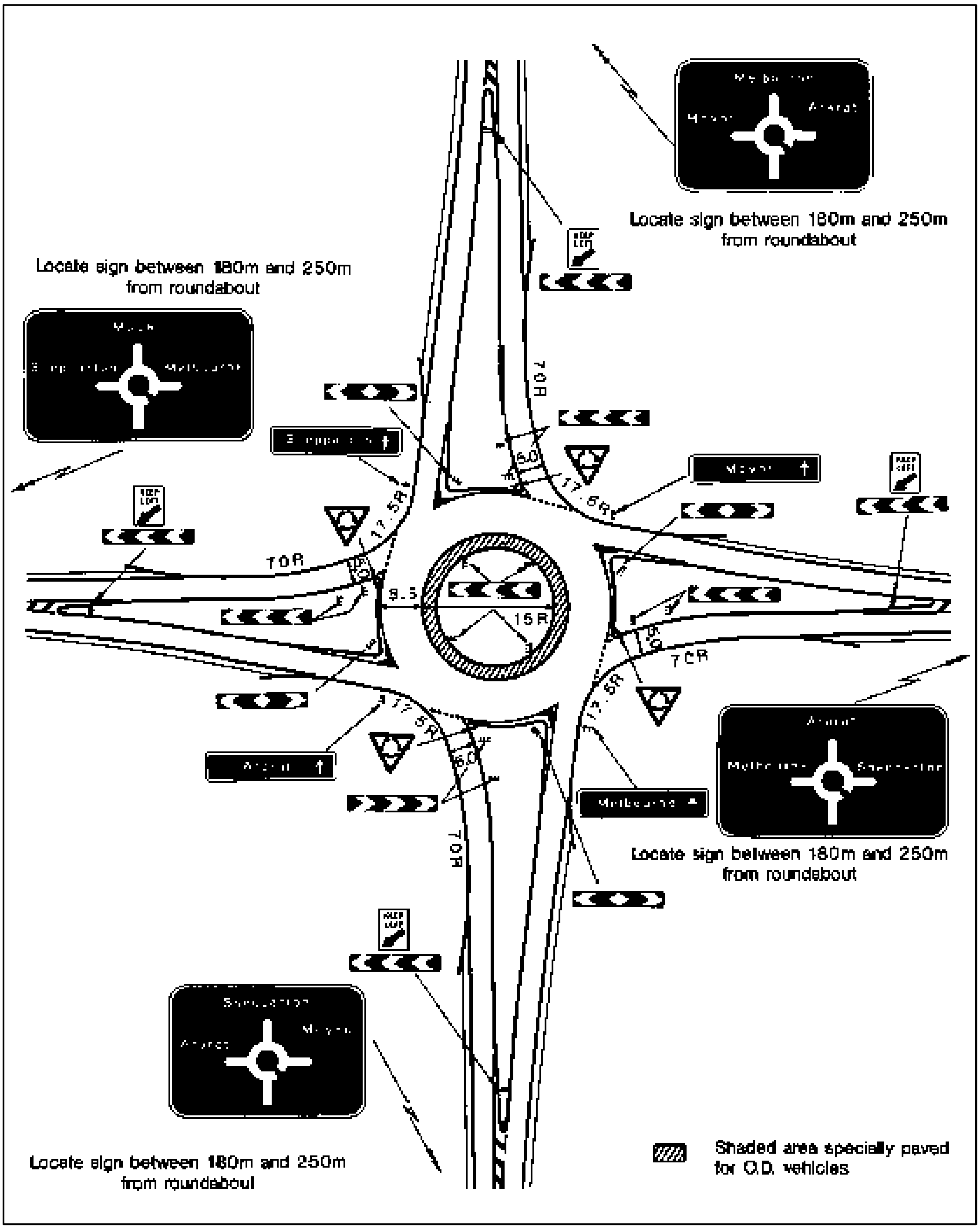


Figure VB10: Collision Diagram for crashes during last 7 years



Values for the a.m. peak.						
Leg	Entry Flow (veh/h)	Entry Capacity (veh/h)	Degree of Satn.	Queueing Delay (s)	Geometric Delay (s)	Delay for Approach (s)
South	302	941	0.32	2.5	9.3	11.8
East	299	999	0.30	1.9	9.5	11.4
North	385	954	0.40	2.7	10.2	12.9
West	452	1069	0.42	1.7	9.4	11.1

Figure VB12: Example of Rural Roundabout Design.



Appendix VC
(from GTEP Part 6: Roundabouts, Section 9 – Trial Installations)

Trial Installations

In general, if the site assessment and evaluation procedures described in this guide have been followed there should be no need for a trial installation of any kind. However, where the site or traffic conditions are very unusual, the use of a trial installation may be appropriate to verify the effectiveness of the treatment or to refine critical aspects of the geometric layout. Trial installations should be used for only a limited period, desirably no longer than about three months, and not more than six months.

This section provides some guidance on the procedures to follow when installing trial roundabouts.

Materials

Removable kerb sections or interlocking concrete blocks, spiked or bonded to the pavement surface with bituminous or thermo-plastic adhesives should be used for any trial installation. While the cost and flexibility of lose materials such as painted stone blocks or sand bags may be an advantage, experience with the use of these materials has not been very satisfactory , particularly if the trial is to last longer than three months.

Construction

When work has been commenced on the installation of a trial roundabout, it is desirable that it be completed as soon as possible to minimise the time drivers are faced with an unfinished layout, or where the traffic priority may not be obvious. If possible all work, including the installation of splitter islands, and associated road furniture and linemarking, should be done in one day.

If it is necessary to leave a trial roundabout in an uncompleted state overnight, the splitter islands should be constructed before the central island. A central island without splitter islands and with only minimal street furniture may confuse drivers and result In wrong-way movements.

Road Furniture

Most operational problems are likely to arise in the days immediately following the installation of a trial roundabout. For this reason, careful attention should be given to the provision of adequate signing and delineation.

All necessary regulatory and warning signs must be installed at temporary and trial roundabouts. It may also be desirable to install appropriate direction signing.

A temporary advance warning sign is desirable on all approaches to inform drivers that a roundabout had been installed. The size of sign required will depend on the status of the road and the nature of the locality (i.e. local, arterial or rural). The size of the sign should be consistent with guidelines set out in TEM Volume 2 and suit the road environment.

If adequate lighting is not available, the delineation of the layout should be enhanced by strong pavement marking including the use of raised retro-reflective pavement markers. Painting the kerbing on all islands is also an advantage.

Temporary roundabouts should be examined under both day and night conditions to determine the effectiveness of the treatment. Initial examinations should take place within one or two days after the installation with inspections and observations periodically throughout the trial period as necessary to evaluate the performance of the treatment.

Appendix VD

(from GTEP Part 6: Roundabouts, Section 12 – Case Studies)

Mickleham Road/Broadmeadows Road, Melbourne. An example of traffic signal metering at a roundabout.

This intersection is in an outer metropolitan area at the junction of two dual carriageway primary arterial roads. Since the roundabout was installed in the early 1980's, there has been considerable growth on all traffic movements except the East to North movement. This is a result of the location of the intersection on the perimeter of the metropolitan area and the road network layout in this area.

The resultant unbalanced flow, particularly in the morning peak period, results in few interruptions to the heavy South to East traffic movement with consequential long delays to the other heavy traffic movement from North to South.

"Metering" traffic signals consisting of 2 aspect (red and yellow) displays were installed on the South approach in 1989 as shown in Figure VD1.1. The signals are actuated by a queue of vehicles extending back along the northern approach to the 'presence' detectors located 90 in upstream of the holding line. A call from the detectors initiates, in effect, a two phase cycle in which only one phase (a red interval) is displayed.

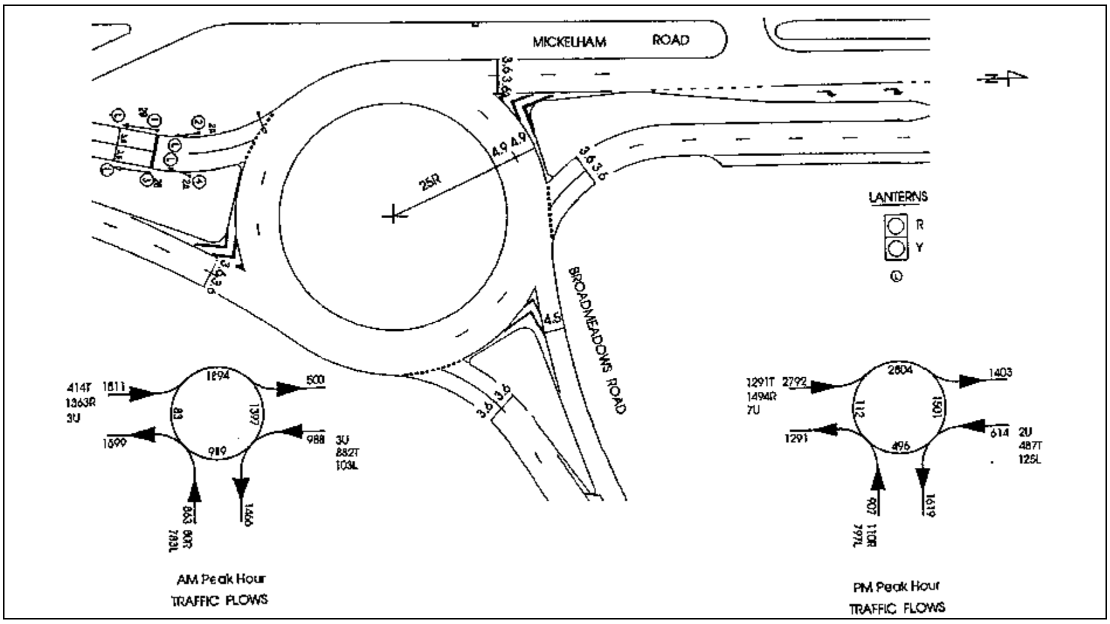
The phasing arrangement is as follows:

- A Phase - minimum green 30s, extendable to 50s,
- B Phase - minimum green 10s, extendable to 15s,
- Inter-green time - 4s yellow and 2s all red,
- Maximum cycle time of 71s.

The system is tuned to balance queuing on the North and South approaches. A reverse side white signal is displayed concurrently with the red signal so that Police, stationed downstream of the signal, can "pick up" violations of the red signal.

Extensive queuing (500 in to 600 m), which occurred regularly during the morning peak period on the Northern approach has been substantially eliminated by the traffic signal operation. Queuing throughout the remainder of the day is insufficient to activate the metering signal. The roundabout continues to provide a high standard of traffic safety and the use of traffic signal metering has avoided the alternative of converting the entire layout to a fully signalised intersection treatment at high cost.

Figure VD1.1: Example of Traffic Metering at a Roundabout.

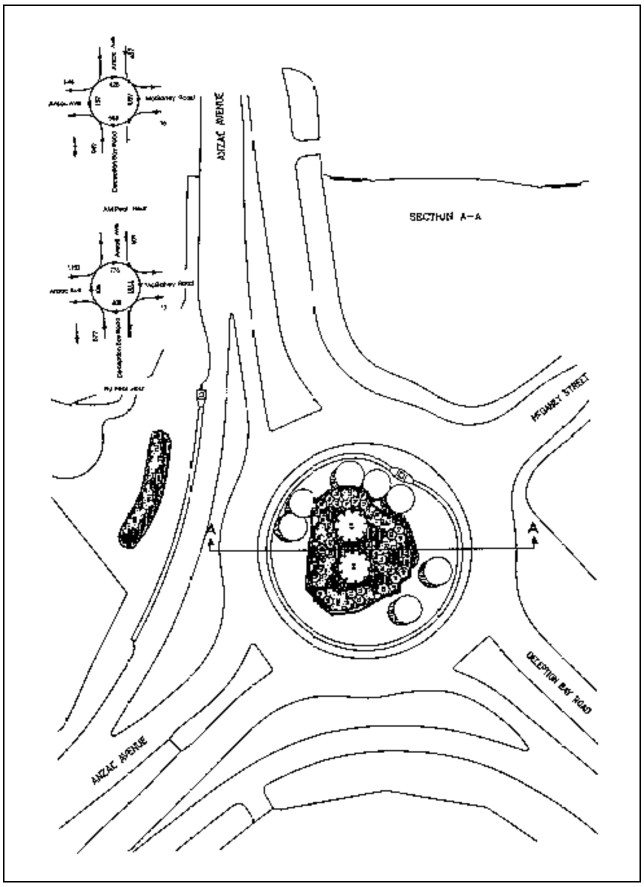


Rothwell Roundabout Redcliffe Road (Anzac Ave) - Deception Bay Road and McGahey Street, Redcliffe City, QLD.

This roundabout was constructed in 1984 to replace an existing 4 way intersection. The layout of the roundabout is shown in Figure VD1.2. The design details differ in two ways from the generally accepted practice as discussed below. The circulating carriageway has been constructed with crossfall inwards toward the central island, thus providing positive superelevation for circulating traffic.

Whilst grading the circulating carriageway in this way can be successful on very large roundabouts and where the topography is favourable, it often results in the circulating carriageway being hidden from the view of approaching traffic on one or more legs of the roundabout. This is the case at this site, as illustrated in the photograph, and drivers have difficulty in recognising the existence of the roundabout.

Figure VD1.2: "Rothwell" Roundabout (Qld)



This problem is difficult to correct, short of major reconstruction, but it can often be compensated for by extra emphasis on direction and warning signing, street lighting and by careful attention to delineation and landscaping.

- The radius of the exit curves are below normal standards causing drivers some difficulty in negotiating the exits.

This type of problem can be accentuated at large roundabouts by the relatively high speed of circulating traffic. This problem is proposed to be addressed by minor reconstruction of the offending exit curves.

Apart from these problems, the roundabout appears to be operating satisfactorily.

Queen Street/High Street, City of Freemantle WA – An Example of the Use of a Mini Roundabout in a City Area.

In central Freemantle motorists have difficulty finding their way around because of the many movements that are prohibited by channelisation at intersections. A major consequence of this in the Holdsworth Street/Queen Street/Henderson Street area is that it makes it difficult to access the Council's major car park in Henderson Street unless the driver is familiar with the street pattern.

A Traffic management scheme was carried out to:

- Make the road pattern simpler and more easily understood by visitors to the City;
- improve access to and egress from the Council's major car park in the Henderson Street;
- improve safety at the Holdsworth Street/Parry Street intersection and the High Street/Parry Street intersection.

Average daily traffic volumes prior to the implementation are shown on the locality plan shown in Figure VD1.3.

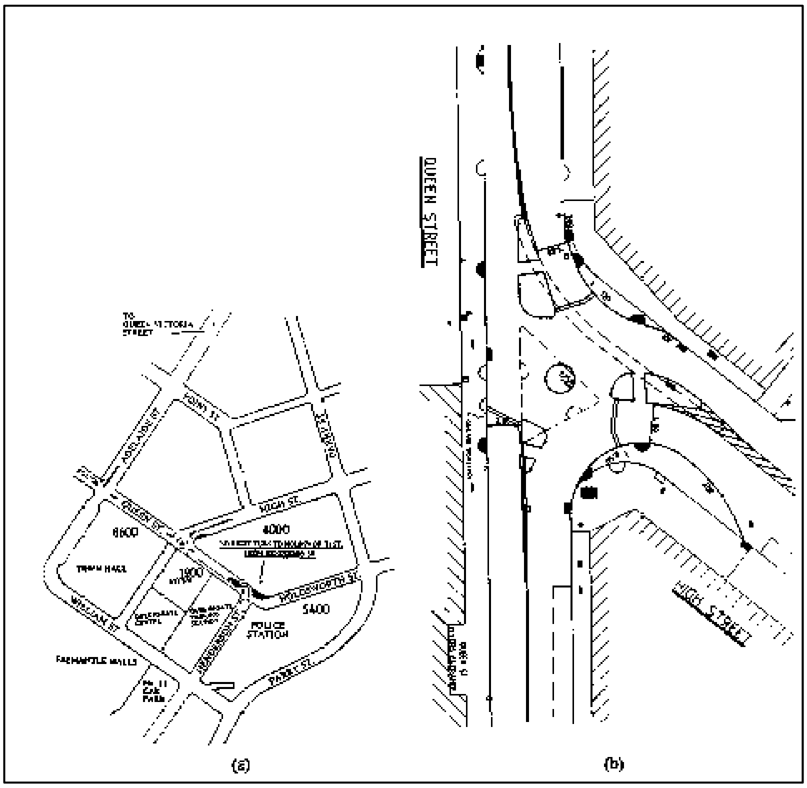
The scheme shown diagrammatically in Figure VD1.3 involved:

- Redesign of the Queen Street/Henderson Street intersection to permit the right turn from Queen Street to Henderson Street and to prohibit movement in an Eastbound direction into Holdsworth Street.
- Redesign the Queen Street/High Street intersection to provide for all traffic movements by use of a mini roundabout.
- Modify the Adelaide Street/Queen Street intersection to provide for the right turn movement form Queen Street to Adelaide Street (East to North).

A 3.5m diameter central island mini roundabout was installed at the High Street/Queen Street intersection using mountable kerbing to accommodate large vehicles turning. This was necessary due to the configuration and constraints of the intersection. It was not possible to achieve any deflection on the Eastern (Queen St) approach, and only minimal deflection on the Western (Queen St) approach.

The high volumes of turning traffic and the low speed environment of the locality have contributed to the control of entry speed on the Queen St approaches and the roundabout operates quite satisfactorily.

Figure VD1.3: Example of a Mini Roundabout



View from High Street

Aerial View – note the small fully mountable central Island.

Unusual Roundabout Usage

The following photographs illustrate some unusual roundabout usage and layouts which operate satisfactorily to solve difficult local site problems.

Twin Roundabouts Beach Road, Bluff Road and Balcombe Road, Melbourne

Freeway Interchange Roundabout Warragul Victoria

Catering for Bus Operation at Roundabouts

Note Bus Bay location.

Note Bus encroachment on to central island.

Catering for Fixed Rail Public Transport at Roundabouts.

Tramway/Light Rail Junction

Roundabout incorporating Railway Level Crossing