ROUNDABOUT DESIGN GUIDELINES

STATE OF MARYLAND
DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION

ROUNDABOUT DESIGN GUIDELINES

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FORWARD

The mission of the Maryland State Administration is to build and maintain a safe and efficient highway system. There is evidence that roundabouts reduce accidents. Maryland has a highway system that is well maintained, well planned, well organized and has a steadily declining accident rate. The future holds many challenges for the Administration in continuing to fulfill its mission. These challenges include performing the same or more work with less staff, maintaining a high level of service with limited resources, enhancing the environment, preserving the highway system and fulfilling the requirements of the Intermodal Surface Transportation Act of 1992 (ISTEA). Given the above, we must constantly review new ideas, processes and technologies while remaining focussed in accomplishing our mission. This guide brings us one step closer to where we want to be.

This guide was developed to set forth a standard approach to the planning, design, and construction of roundabouts in the State of Maryland, given that no federal guidelines exist. This text borrows information from recognized experts in the planning, design and construction of roundabouts; namely the Australian Design Guide (3). Generally, major concepts for safety and design should follow AASHTO Design Guide. This guide supplements these federal Guidelines until such time when formal guidelines are established.

We believe Austroads is a leader in the planning, design and implementation of roundabouts. We commend Australia, the European Community and others for recognizing the true benefits of roundabouts early and enhancing their design such that they operate safely and efficiently. We have selected the Australian Design guide as a model because its design procedure most closely represents current procedures already adopted by the Maryland State Highway Administration. We would like to extend our sincere thanks to those individuals at Austroads responsible for developing "Guide to Traffic Engineering Practice - "Roundabouts". Most importantly we thank them for allows us to reprint much of what they had developed.

It is intended that this guide be used to standardize the approach to roundabouts in Maryland. It is only a guide. **It is being issued as an interim document for one year from April 1, 1995. We expect many changes in the upcoming months and subsequent years to come.** We encourage users to suggest improvements and send them to :

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1.0 INTRODUCTION

Well designed roundabouts have proven to be safe and efficient forms of intersection control in the countries that have adopted modern guidelines. These countries include Great Britain, Australia, France, Germany, Spain, Norway, The Netherlands, among other countries.

These guidelines are intended to be temporary and updated as often as is necessary. Design guidelines from other countries form the basis of this document. Over time, these guidelines will be analyzed as to their applicability to the driving conditions in Maryland. Input into these guidelines from users is encouraged.

Roundabouts operate by gap acceptance, in that approaching drivers must give way to circulating traffic in the roundabout. The proven safety performance of most roundabouts is due to the low relative speeds of all vehicles and the relative simplicity of decision making required of drivers.

Conditions will be encountered wherein the procedures highlighted in these guidelines cannot be fully implemented. It is expected that the designer make modifications as necessary while ensuring the major concepts of safety and design.

The aims of these guidelines are as follows:

a. to give guidance on where roundabouts may be used
b. to describe the performance and operation of roundabouts
c. to give guidance on design standards for roundabouts so that high standard and uniform design will be encouraged.

The designer of a modern roundabout should be fully aware of the difference between a roundabout and a traffic circle. Basically, there are three main differences; Yield-at-entry, Deflection, and Flare. These are illustrated in Figure 1.1.

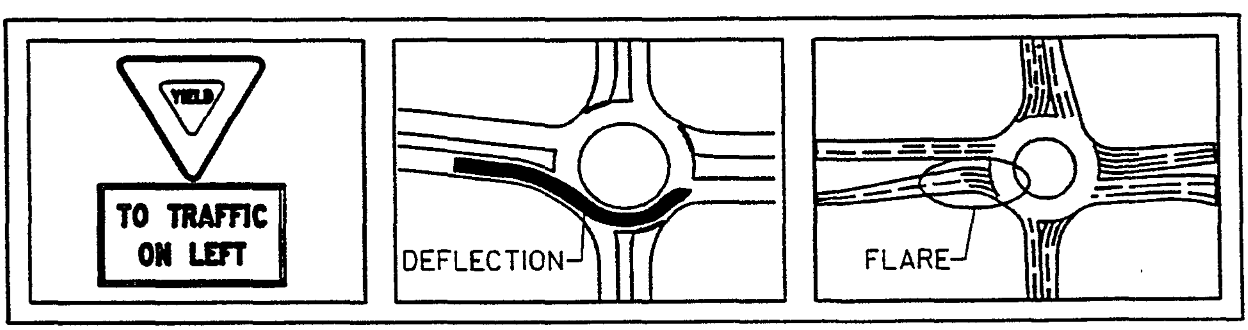


FIGURE 1.1 Yield at entry, Deflection, and Flare

Yield-at-entry allows vehicles in the roundabout to continue through the roundabout to their appointed exit and eliminates gridlock which occurs when entering vehicles are afforded the right-of-way. Yield-at-entry also enables the traffic engineer to design smaller roundabouts.

Deflection is the physical slowing of vehicles through the roundabout which is achieved by causing the driver to curve around the central island. Deflection increases the safety of the intersection by lowering the entry and circulating speeds.

Flare is the widening of the approach to the roundabouts to increase capacity.

2.0 USE OF ROUNDABOUTS

2.1 GENERAL

Roundabouts should be considered at a wide range of intersection types including but not limited to; freeway terminal interchanges, state route intersections, and state route/local route intersections. Roundabouts perform better at intersections with roughly similar traffic volumes and at intersections with heavy left turning movements. Roundabouts can improve safety by simplifying conflicts, reducing vehicle speeds and providing a clearer indication of the driver's right-of-way compared to other forms of intersection control.

2.2 SITE SELECTION CRITERIA

The following site selection guidelines are intended as general guidelines only. The designer should determine the applicability of a roundabout at a particular intersection by considering the following items:

- Capacity Analysis of all methods under consideration
- Cost/Benefit Analysis
- Percentage of Truck Traffic
- Bicycle and Pedestrian Traffic
- Right-of-Way Consideration
- Parking Requirements
- Compatibility with Adjacent Intersection
- Safety Aspects
- Effect of Possible Traffic Growth
- Speed of Traffic
- Installation and Maintenance Costs

ROUNDABOUT SHORTLISTING GUIDELINES

Introduction

The following guidelines are based on existing design manuals from England, Australia, and other countries, and video tapes of existing roundabouts. The guidelines are not meant to be rigid but should be used in conjunction with engineering judgement, and traffic analysis. For example, it could be stated that a roundabout should not be placed where there is an existing signal in close proximity (i.e. Chevy Chase Circle) because the queues from the signal may extend temporarily into the roundabout. Intuitively, this would not seem to be an appropriate place for a roundabout, however traffic analysis may indicate that a roundabout may work better than any other solution. The proposed intersection treatment, therefore, should be chosen based on the advantages/disadvantages, benefits/costs that it provides.

Location

- High Accident Location (with left turn or right angle accidents)
- Capacity/Delay Problem Intersection
- Intersection in which traffic signal was requested but not warranted.
- 4-Way Stops

Traffic Volume and Composition

- Heavy Delay on Side Street
- Flow Distribution with Heavy Left Turn Movement (makes signals less efficient -no impact on roundabout)
- DHV of 7000 or Less (initially)

Right-of-Way

- Generally take no more right-of-way than comparable solution using signals.

Appropriate Sites for Roundabouts

- Heavy delay on minor road.
- Traffic signals result in greater delay.
- Intersection with heavy left turning traffic.
- Intersection with more than four legs or unusual geometry.
- At rural intersections (including those in high speed areas) at which there is an accident involving crossing traffic.
- Where major roads intersect at "Y" or "T" junctions.
- At locations where traffic growth is expected to be high and where future traffic patterns are uncertain or changeable.
- At intersections where U-turns are desirable.
- At Freeway Interchange Ramps.
- High accident intersection where right angle accidents are prominent.

Inappropriate Site for Roundabouts

- Where a satisfactory geometric design cannot be provided.
- Where a signal interconnect system would provide a better level of service.
- Where it is desirable to be able to modify traffic via signal timings.
- Where peak period reversible lanes may be employed.
- Where the roundabout is close to existing signals and queueing from the signal could be a problem.

2.3 COST/BENEFIT ANALYSIS

A cost/benefit analysis shall be completed for all intersections in which a roundabout is being considered. See procedure outlined in Appendix A.

3.0 PERFORMANCE OF ROUNDABOUTS

3.1 GENERAL

Until more data is gathered concerning the performance of roundabouts in Maryland, the Maryland State Highway Administration recommends that designers use the Australian practice at this time.

Australian practice for determining the capacity and delay of roundabouts is based in gap acceptance theory, and the techniques have been researched in Horman and Turnbull (1974), Avent and Taylor (1979) and Troutbeck (1984, 1986 and 1990). The most recent methodology developed by Troutbeck (1989), uses the empirical results of field observations made in four Australian capital cities.

These field studies identified a number of driver behavior aspects that affect the analysis of capacity and delay. These are:

- That entering vehicles generally give way to all circulating vehicles. Entering drivers are often unsure that a circulating driver to their left may intend to leave at the next exit and travel across their paths. Consequently, entering drivers tend to give way to all circulating vehicles, even where the circulating roadway is two or more lanes wide. An exception is when vehicles are entering from an auxiliary "right turn only" lane. If this auxiliary lane and the entry curve is designed so that entering drivers are protected from the circulating traffic, they will generally proceed without "giving way" to any circulating vehicle.

- That at multi lane entries, vehicles are prepared to enter simultaneously alongside other entering vehicles at the same approach.

- That drivers entering in different lanes of the same approach will behave differently.

- That exiting vehicles have no effect on drivers entering at the same leg unless the negotiation speed is high or the roundabout is small and the entering drivers have difficulty in determining whether a vehicle is exiting or not.

These findings influence the capacity and delay calculations. The principal departure from the 1986 Australian guide is that the drivers in each entry lane on a particular approach behave differently. This means that each entry lane will have a different capacity and vehicle delay. As a consequence, if the number of entry lanes is doubled then the capacity is not quite doubled.

The usual terms used to define gap-acceptance behavior are the critical acceptance gap, t_a, and the follow-up time, t_f. The critical acceptance gap is the minimum acceptable gap that will be acceptable to a homogeneous and consistent population of drivers. The follow-up time is the minimum headway between minor stream vehicles which enter in the longer gaps in the circulating traffic. In both cases the units are in seconds. The gap acceptance terms represent the average for all drivers as the average predicted capacity and average delay values are required. In the theory, it is assumed that all drivers will accept a gap greater than the critical acceptance gap. It is also assumed that drivers are consistent and behave exactly the same each time a gap is offered. That is, a driver does not reject a gap only to accept a shorter one later. However in practice, drivers are different from one another and often act inconsistently because they are not always able to make accurate assessments of gap durations. Also, drivers do occasionally reject a gap then accept a shorter one. Calculations based on these assumptions nevertheless give estimates of capacity which are reasonably consistent with observations (Troutbeck 1989, Catchpole and Plank, 1986 and Plank and Catchpole, 1984).

As the drivers in each entry lane behave differently, each entry lane will be given different critical gap and follow-up headway parameters.

3.2 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.

3.3 ANALYSIS OF THE CAPACITY OF ROUNDABOUTS

This section provides an analytical technique which can be expected to give quite accurate results which reflect current Australian experience and practice. SIDRA Software is recommended and is available through McTrans at the University of Florida.

In situations where a high degree of accuracy is not required, Figures 3.3, 3.4 and 3.5 may be used to obtain general estimates of the capacity of a roundabout.

3.3.1 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 3.1 and 3.2 show the conversion of typical traffic turning movements at a cross-road type intersection into entry and circulating flows on a roundabout.

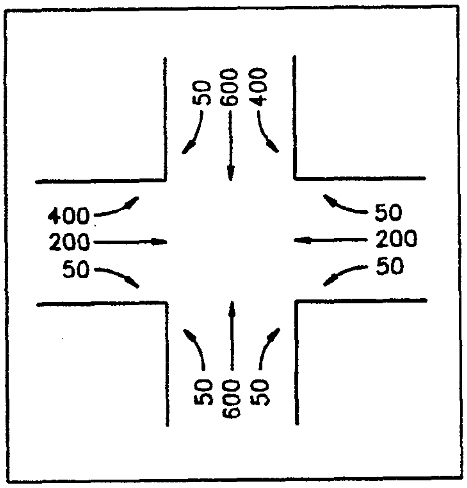


FIGURE 3.1 Typical turning movement diagram

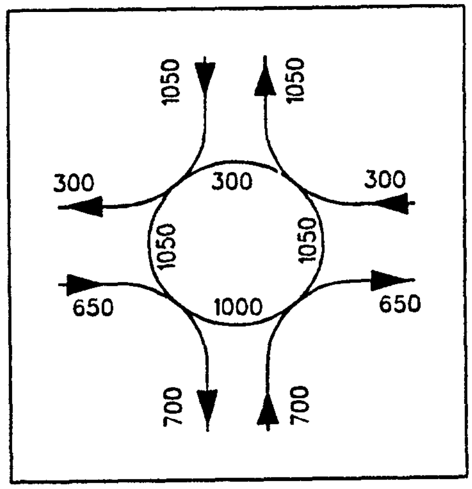


FIGURE 3.2 Roundabout entry and circulating flows

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 approaches. 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 3.3 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 3.3, 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 3.3 is based on the acceptable degree of saturation being less than 0.8.

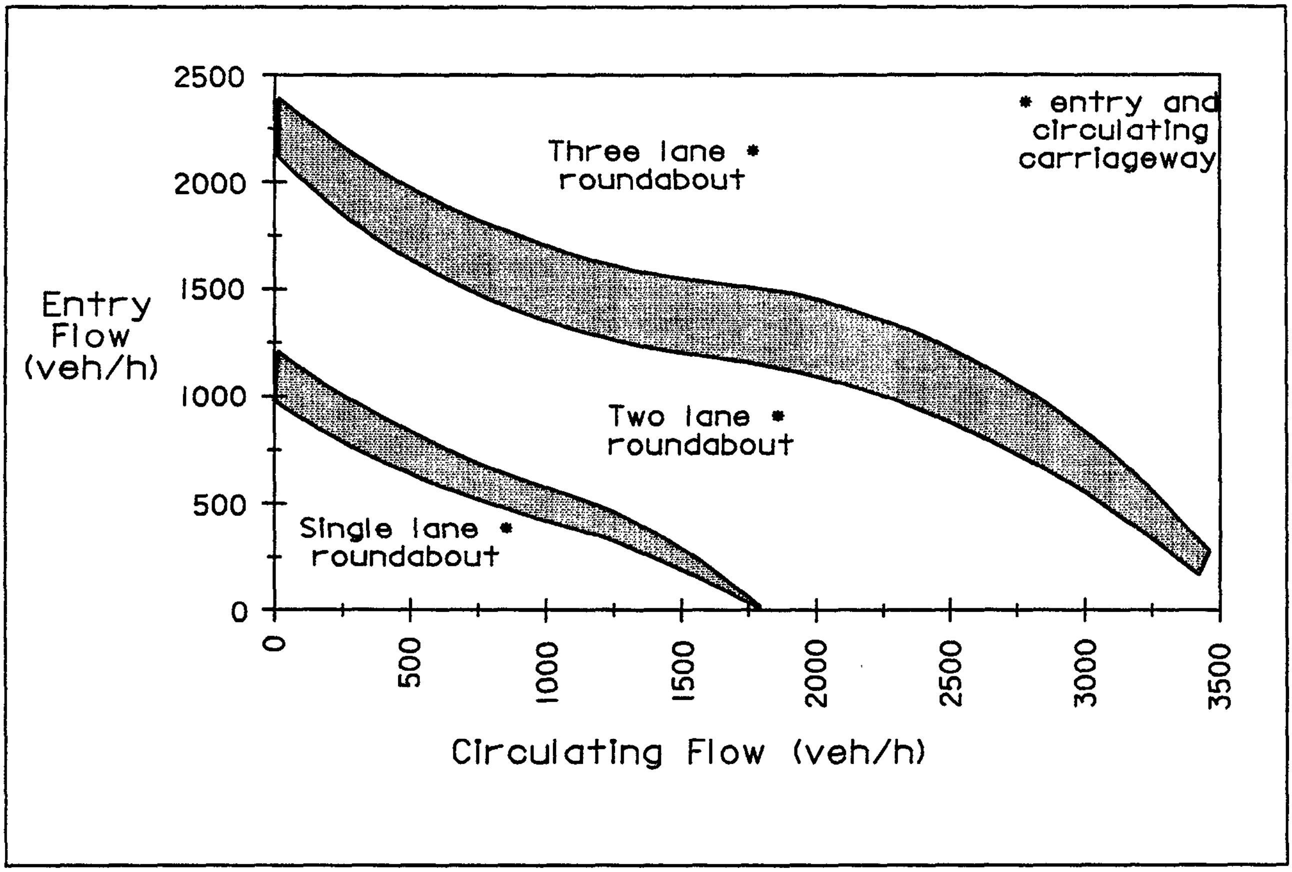


FIGURE 3.3 Required number of entry and circulating lanes

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 18 feet
  n_e is 2 for entry widths between 18 and 33 feet
  n_e is 3 for entry widths greater than 33 feet

- the number of circulating lanes, n_c.
  n_c is equal to 1 for circulating roadway widths less than 33 feet
  n_c is 2 for widths greater than or equal to 33 feet and less than 50 feet
  n_c is 3 for widths greater than 50 feet.

NOTE: For some circulating roadways between 25 feet and 33 feet 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 the entry lanes).

Refer to 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 behavior 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 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).

Inscribed Diameter (ft)	Circulating flow (veh/h)					
	0	500	1000	1500	2000	2500
66	2.99	2.79	2.60	2.40	2.20	2.00
82	2.91	2.71	2.51	2.31	2.12	1.92
98	2.83	2.63	2.43	2.24	2.04	1.84
115	2.75	2.55	2.36	2.16	1.96	1.77
131	2.68	2.48	2.29	2.09	1.89	1.70
148	2.61	2.42	2.22	2.02	1.83	1.63
164	2.55	2.36	2.16	1.96	1.76	1.57
180	2.49	2.30	2.10	1.90	1.71	1.51
197	2.44	2.25	2.05	1.85	1.65	1.46
213	2.39	2.20	2.00	1.80	1.61	1.41
230	2.35	2.15	1.96	1.76	1.56	1.36
246	2.31	2.11	1.92	1.72	1.52	1.33
262	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 vph are not applicable to single lane circulating roadway (shaded area in table).
- The ratio of the critical acceptance gap to the follow-up headway (t_{ad}/t_{fd}), is given in Table 3.2. The critical acceptance gap is the product of the appropriate values from Table 3.1 and Table 3.2.

TABLE 3.1 Dominant Stream Follow-up Headways (t_{fd}). (Initial values) in seconds.

Number of circulating lanes	one			more than one		
Average entry lane width (ft)	10	13	16	10	13	16
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)

- For single lane circulating roadways, if the critical gap calculation from Tables 3.1 and 3.2 is less than 2.1 s, use 2.1 s.
- For multi-lane circulating roadways, the minimum value of critical gap should be 1.5 s.

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

TABLE 3.2 Ratio of the Critical Acceptance Gap to the Follow-up Headway (t_{ad}/t_{fd})

(a) For a single lane entry

Table 3.1 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 3.1 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 3.2. The critical acceptance gap is the product of the appropriate values from Table 3.1 and Table. 3.2.

(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 finalize 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 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 3.1, 3.2, 3.3 and 3.4.

Table 3.1 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 in Table 3.3.

Table 3.4 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 3.1 and 3.4) and the ratios in Table 3.2. As stated above, critical acceptance gap values need to be calculated separately for each entry lane. Refer to Troutbeck (1989) for an example of these calculations.

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 3.1.			

TABLE 3.3 Adjustment Times for the Dominant Stream Follow-up Headway

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 3.4 Sub-dominant Stream Follow-up headway t_{fs}

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 number of circulating 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 (τ) between bunched vehicles is about 1 s and if there is only one lane the average headway is 2 s.

If a circulating roadway equal to or greater than 33 feet 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 (τ) will be 1 s. (see Table 3.5). Under these conditions the vehicles might travel in an offset pattern and users should decide whether or not the circulating roadway 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 2 s. 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 3.3 may then need to be consulted when estimating the follow-up headways.

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 (characterized by the average headway between bunched vehicles) and the proximity of the roundabout to signalized 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 3.6. (Also see Akçelik 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 signalized 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.

	Circulating Roadway Width			
	less than 35 feet		greater than or equal to 35 feet	
	Number of effective lanes	Headway between 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

TABLE 3.5 Average headway between bunched vehicles in the circulating traffic (τ) and the number of effective lanes in the circulating roadway.

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 (Qc, τ, and Θ). The appropriate equation is:

$$C=\frac{3600\,(1-\Theta)\,q_c e^{-\lambda(t_a-\tau)}}{1-e^{-\lambda t_f}} \qquad (3.1)$$

Where:

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:

$$\lambda = \frac{(1-\Theta)\, q_c}{1-\tau q_c} \tag{3.2}$$

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 3.4 and 3.5 may be used to obtain a quick estimate for use in the planning and preliminary layout of a roundabout at a particular site.

Figure 3.4 refers to a single lane roundabout with a 13 foot wide entry lane and one circulating lane. The results in Figure 3.5 reflect the operating conditions of a roundabout with two 13 feet wide entry lanes and two circulating lanes.

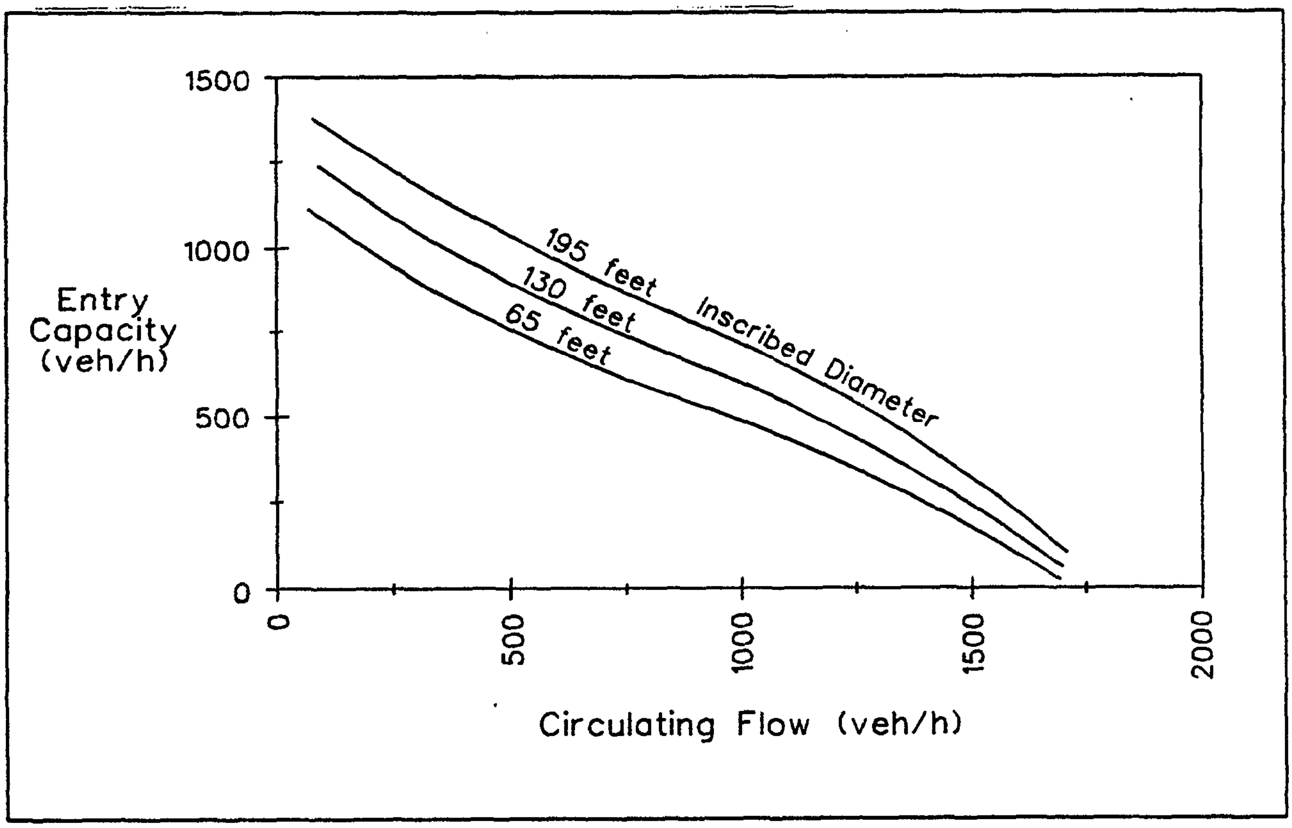


FIGURE 3.4 Entry capacity for a single lane roundabout with a 13 foot wide entry lane and one circulating lane

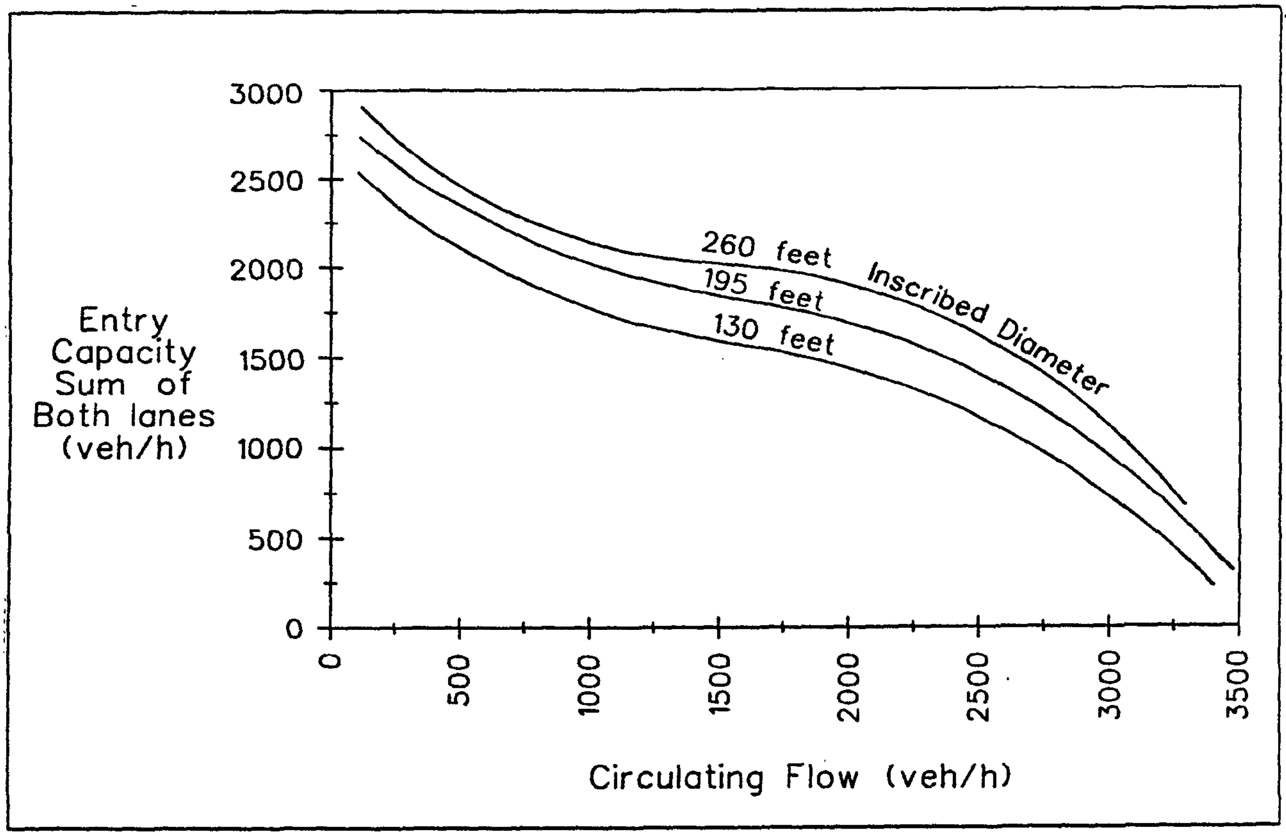


FIGURE 3.5 Entry capacity for a roundabout with two 13 foot wide entry lanes and two circulating lanes

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.

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:

$$x = \frac{Q_m}{C} \qquad (3.3)$$

Where:

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

C = entry lane capacity in veh/h (from equation 3.1 or figures 3.4 and 3.5)

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 always be 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.

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

TABLE 3.6 Proportion of Bunched Vehicles, Θ

3.4 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:

(i) The delay to drivers slowing down to the negotiation speed, proceeding through the roundabout and then accelerating back to normal operating speed; or

(ii) 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 "YIELD" 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.

3.4.1 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:

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

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:

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

where:

w_m = average delay per vehicle in seconds

w_h = minimum delay in seconds when 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 Q_m persists (use 1 h or o.5 h)

Z = x - 1

x = degree of saturation of the entry lane (= Q_m/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 (Akçelik 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.

Approach Speed V_a (MPH)	Distance* around roundabout D (ft)	Negotiated speed through roundabout V_n (MPH)							
		9	12	16	19	22	25	28	31
25	65	10	8	7	7	7			
25	195	19	15	12	9	7			
25	325		22	17	13	10			
25	460				18	14			
25	590					18			
37	65	13	11	10	10	10	10	10	10
37	195	23	18	15	13	10	10	10	10
37	325		26	21	18	15	12	10	10
37	460				22	19	15	12	10
37	590					23	19	15	10
50	65	17	15	13	13	13	13	13	13
50	195	26	22	19	17	14	13	13	13
50	325		29	25	21	19	16	13	13
50	460				26	23	19	16	13
50	590					27	23	19	16
62	65	20	18	17	17	17	17	17	17
62	195	30	25	22	20	18	17	17	17
62	325		33	28	25	22	20	17	17
62	460				30	26	23	20	17
62	590					30	27	24	20

* Refer to Figure 3.8 for the definitions of the dimensions.

TABLE 3.7(a) Geometric Delay for Stopped Vehicles (Seconds per vehicle)

Approach Speed V_a (MPH)	Distance* around roundabout D (ft)	Negotiated speed through roundabout V_n (MPH)							
		9	12	16	19	22	25	28	31
25	65	7	4	2	1	0			
25	195	17	11	7	4	0			
25	325		19	13	8	4			
25	460				13	8			
25	590					12			
37	65	11	8	5	4	3	2	1	1
37	195	20	15	11	8	4	2	1	1
37	325		22	17	13	9	5	1	1
37	460				17	13	8	4	1
37	590					17	12	7	2
50	65	14	11	9	7	6	5	4	3
50	195	24	19	15	11	8	5	4	3
50	325		26	20	16	13	9	5	3
50	460				21	17	13	9	4
50	590					21	16	12	7
62	65	18	15	12	10	9	8	7	6
62	195	27	22	18	15	12	9	7	6
62	325		29	24	20	16	13	10	6
62	460				25	20	17	13	12
62	590					25	20	16	

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

TABLE 3.7(b) Geometric Delay for Vehicles Which Do Not Stop (Seconds per vehicle)

3.4.2 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:

$$d_g = P_s\, d_s + (1-P_s)\, d_u \qquad (3.6)$$

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 also been documented by Middleton (1990).

The proportion of entering vehicles which must stop, P_s, can be estimated using Figures 3.6 and 3.7 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.

3.4.3 Total Average Delay

Total average delay is the sum of the queuing delay and the geometric delay. Again the total delay will not be the same for vehicles making various turns and using different entry lanes. The total average delay per vehicle from an approach must then be estimated using the proportion of vehicles making each movement and their respective delays.

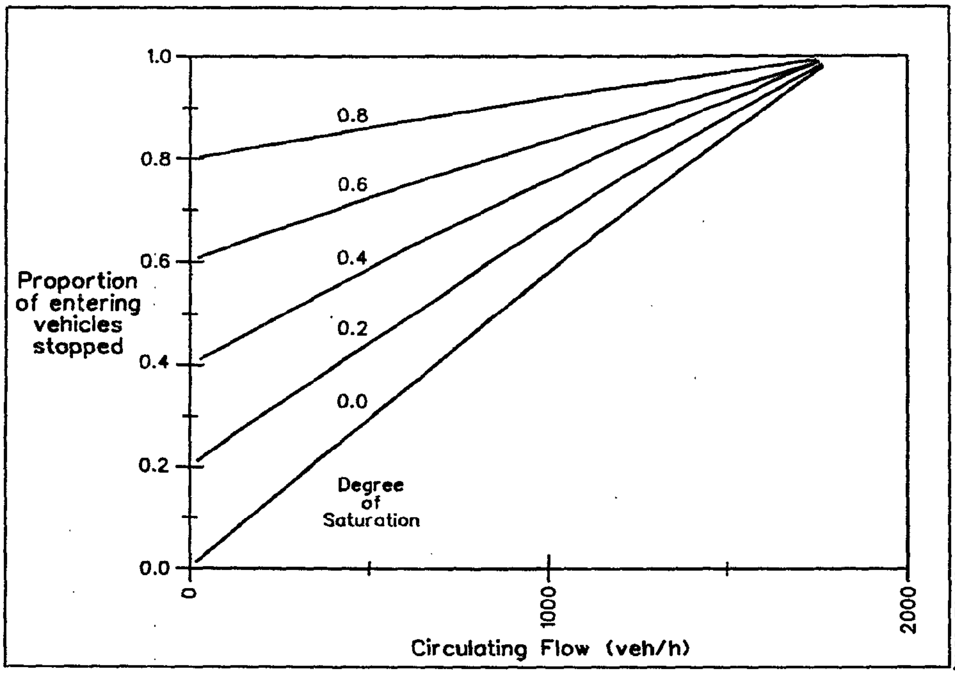


FIGURE 3.6 Proportion of vehicles stopped on a single lane roundabout

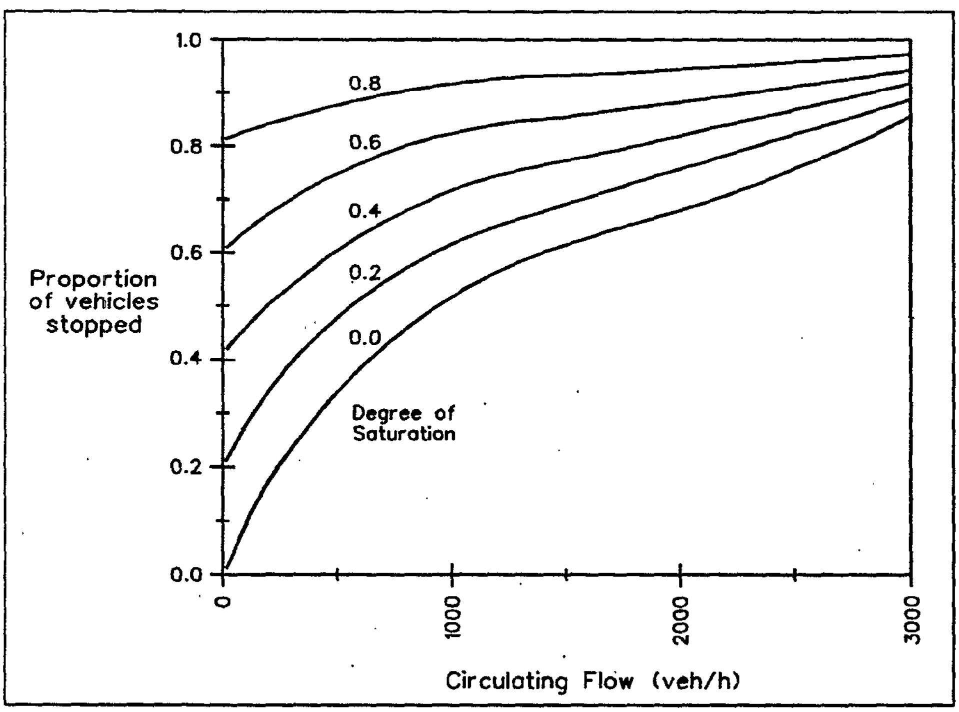


FIGURE 3.7 Proportion of vehicles stopped on a multi-lane entry roundabout.

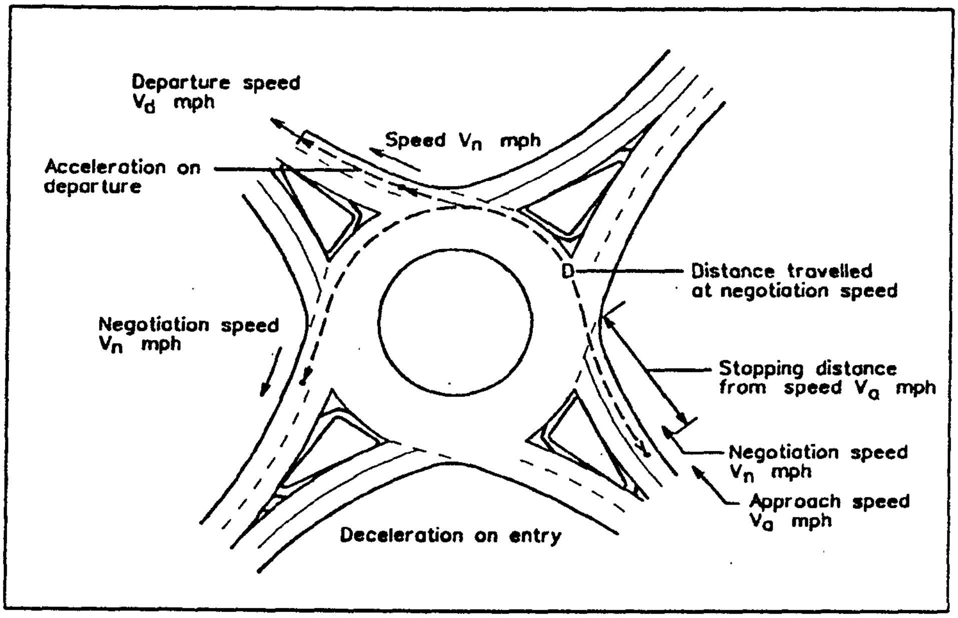


FIGURE 3.8 Definitions of the Terms used in Tables (a) and (b)

3.5 Entry Queue Lengths

The average entry queue length, n_w, under steady state under-saturated conditions is given by the product of the average queuing delay, w_m, and the entry lane flow, Q_m.

$$n_w = w_m Q_m \text{ (for } x < 1) \qquad (3.7)$$

As a rule of thumb, the 95 percentile queue length is three times the average, i.e. a queuing space for $3n_w$ vehicles will be exceeded about 5 per cent of the time. Values of queuing space should be rounded up to the next vehicle.

Note that the distribution is not a Normal distribution and the usual limits do not apply.

3.6 Safety of Roundabouts

The safety performance of roundabouts has been documented in a number of Australian and UK studies. "Before" and "after" type accident studies carried out at intersection involving a wide range of site and traffic conditions at which roundabouts have been constructed, indicate very significant reductions in casualty rates.

Intersection Treatment		Mean Casualty Accident Rate	Typical Range of Casualty Accident Rates
T-Intersections	-Unsignalized	1.5	1.3 - 1.7
	-Signalized	1.4	1.2 - 1.6
Cross-intersections	-Unsignalized	2.4	2.1 - 2.7
	-Signalized	1.7	1.6 - 1.8
Multi-leg Intersections	-Signalized	3.2	2.8 - 3.6
Roundabouts	(high volumes)	0.8	0.6 - 1.1
Roundabouts	(low volumes)	0.4	0.1 - 1.0

TABLE 3.8 Typical Casualty Accident Rates for Different Urban Intersections with Moderate to High Volumes in Victoria, Australia

The previous tabulation (Table 3.8) illustrates the result of comparative studies carried out in Australia. Similar results have been obtained in the UK.

The good safety record of properly designed roundabouts can be attributed to the following factors:

- The general reduction in conflicting traffic speeds (limited to less than 30 MPH) passing through the intersection on all legs.
- Elimination of high angles of conflict thereby ensuring low relative speeds between conflicting vehicles.
- Relative simplicity of decision making at the point of entry.
- On undivided roads, in high speed areas, long splitter islands provide good "advance warning" of the presence of the intersection.
- Splitter islands provide refuge for pedestrians and permit them to cross one direction of traffic at a time.
- Roundabouts always require a "conscious action" on the part of all drivers passing through the intersection, regardless of whether other vehicles are present or not.

An understanding of the above factors and their implications, in respect to the geometric design of roundabouts is essential to the full achievement of the safety benefits of roundabouts.

The safety record of roundabouts with more than three circulating lanes has not been well established. Maycock and Hall (1984) analyzed the influence of the geometry of roundabouts on their accident performance. They did not find circulating roadway width to be a significant factor. The accident potential at roundabouts with 3 circulating lanes would be influenced by the drivers' entry curvature. At this time, there is insufficient data to quantify the safety performance of roundabouts with 3 entry and 3 circulating lanes.

Standards for deflection of vehicle paths through roundabouts were developed in the United Kingdom from the safety performance of a large number of roundabouts. At some of these sites, the size of the central island was reduced to provide greater circulating pavement widths and thus increase capacity. In doing so, deflection through these roundabouts was reduced. A study of 23 of 26 such sites showed a 91 percent increase in casualty accidents after deflection through the roundabouts was reduced. This increase, which was statistically significant at the 0.001 level, was attributed to higher vehicle speeds through the roundabouts and supports the design criteria outlined in this guide.

3.7 The Cost of Roundabouts

The cost of roundabout installation varies a great deal between sites depending on factors such as the area of pavement construction and other road works, the cost of land acquisition and the relocation of services. Roundabouts may be either more or less expensive than traffic signals depending on the particular site. There are many sites where traffic signals can be installed with little or no change to the existing pavement and curb lines, whereas this is rarely possible with a roundabout. For those situations traffic signals will generally be less costly to install than roundabouts. When completing a benefit/cost for a roundabout, the life cycle costs for the anticipated duration of the improvement should be considered.

Maintenance costs associated with roundabouts will normally include:

- Pavement Maintenance: The "scrubbing action" of heavy vehicles turning through a roundabout makes it necessary to carefully consider the type of surface treatment required and it may influence the frequency of resurfacing.
- Curb and gutter and drainage systems,
- Traffic signs,
- Pavement markings,
- Street lighting,
- Landscaping

In general there is little difference between the cost of maintaining these items at a roundabout compared with that at other forms of channelization of equivalent pavement area. However the additional cost of maintaining and operating traffic signals, which may be required in conjunction with other forms of channelization, is not required at a roundabout, except in the unusual situations where "metering" traffic signals are required.

3.8 Environmental Issues

Roundabouts can offer considerable scope for environmental enhancement and are sometimes favored over other forms of intersection treatment in environmentally sensitive areas.

The central island can be landscaped and planted provided:

- the treatment does not block any of the sight triangles (refer Section 4)
- any landscaping will yield to out-of-control vehicles and not be a hazard;
- the treatment does not constitute an unnecessary distraction to drivers.

Planting can be used to discourage pedestrians from crossing at undesirable locations.

Compared to traffic signals, roundabouts may operate with reduced queue lengths and shorter average delays. This results in:

- less air and noise pollution;
- lower fuel consumption;
- less parking restrictions;
- better access to private driveways.

In addition, the use of a roundabout eliminates potential traffic safety and disruption problems associated with the malfunction of traffic signals.

Roundabouts can be used on local streets to discourage high traffic speeds and the intrusion by very large vehicles. Provisions for emergency and service vehicles need to be considered in the design of these roundabouts.

3.9 Means of Improving the Performance of Roundabouts

3.9.1 Continuous (Slip) Lanes

Where there is a heavy right turn traffic movement, this may be either separated from the operation of the roundabout by providing a separate right turn slip lane or by providing an auxiliary lane for this traffic. In the latter case, the right turn entry conditions can be improved by positioning the splitter island past the entry lanes thereby shielding the right turn entry movement.

Where a separate right turn slip lane is used, right turning traffic can be excluded from the capacity and delay calculations. To be fully effective, the layout must ensure that the circulating traffic and the right turning traffic does not conflict. The exclusion of this right turning traffic will increase the capacity of the roundabout.

3.9.2 Flaring (tapering) of the Entries

Kimber (1980) has established equations for the performance of roundabouts (in the UK) with flared entries. The capacity is increased by about 20 percent if an entry is flared from two lanes to three lanes over a short tapering distance from the entry. However studies in the UK (Maycock and Hall 1984) showed that as the entry widths are increased, so do the accident rates between entering and circulating vehicles. The increase in capacity is not insignificant, but it is substantially less than can be achieved by adding another full length entry lane. It has been observed that drivers' perception and behavior at roundabouts is quite variable and at many locations drivers do not use multi-lane roundabouts to their maximum effectiveness. In Australia, the introduction of a short flaring on an approach generally does not realize increased benefits. However if designers wish to investigate the potential for flared approaches, they should consult the design equations given in Kimber (1980).

3.9.3 Grade Separation

The performance of any intersection can be improved with the grade separation of some of the conflicting traffic movements. On heavily travelled roads, roundabouts incorporating grade separated movements can offer significant benefits. It is usual for the through traffic movements of the more important road to be separated from the roundabout, e.g. as in a "bridged rotary" type interchange on a freeway. However in special situations, the traffic movements separated out from the roundabout can also be a right turn movement.

The analysis of a grade separated roundabout is the same as for other forms of roundabout except that the traffic movements that are removed from the conflict are not included in the calculation of the performance characteristics. It is obvious that grade separation substantially increases the capacity and reduces delay and accident potential of a roundabout.

3.9.4 Entry Metering

Roundabouts will not function efficiently if there are insufficient acceptable gaps in the circulating traffic stream. If there is one approach with a very heavy through or right turn traffic movement, which is not interrupted sufficiently by the circulating flow, then this stream will present few acceptable gaps to drivers at the next entry. The capacity of this next entry will be very low and the delay to this traffic excessive. Often this situation occurs only during peak flow periods and at these times the operation can be dramatically improved by artificially interrupting the high flow approach.

Entry metering is usually done by installing traffic signals to meter the flow as it approaches the roundabout. This has the effect of bunching the flow of traffic and introducing more of the longer duration gaps. It is important not to locate the signals too close to the entry as this may confuse the "right of way" requirements at the entry. While there has been concern expressed about the confusion of vehicle priority where signalized pedestrian crossings are located close to roundabouts, Thompson et al (1990) concluded that "there was no evidence to support the view that drivers who proceed through these traffic signals strongly associate them with the control of entry onto a roundabout".

Pedestrian operated traffic signals, set to operate on a regular cycle during the peak periods or activated by a detector placed in the approach suffering excessive delay, have been successfully used at some sites to meter traffic into a roundabout. Two aspect signals (red and yellow only) have also been used. Wherever such metering is used, it is important to provide signing at the signals to advise drivers that the flow is being metered. the "STOP HERE ON RED SIGNAL" sign is usually also provided.

Where a signalized pedestrian crossing is used for traffic metering, care needs to be taken with its location to ensure that sufficient space is provided between it and the exit from the roundabout to avoid traffic queuing back into the circulating roadway.

Metering can be applied to more than one entry at the same roundabout.

4 GEOMETRIC DESIGN OF ROUNDABOUTS

4.0 GENERAL

AASHTO guidelines should be followed for turning radii, superelevation, grades, etc. If they are not followed, justification must be documented and approved at the P.I. submittal.

4.1 GENERAL PRINCIPLES

4.1.1 FLARE DESIGN AT ENTRY

Flare is the widening of the approach road to increase the capacity of the roundabouts. The approach should never be widened such that there are more approach lanes than circulating lanes. The length of flare should be between 100 and 300 feet. See Figure 4.1.

4.1.2 ENTRY WIDTH

Entry width can vary depending on the design vehicle and approach roadway width. In general, the entry width should be between 11 feet and 15 feet per entry lane. The entry width should be less than or equal to the circulating width.

In the end, the entries should be designed to accommodate the design vehicle while ensuring adequate deflection.

The approach curve to the roundabout should be the same radius or smaller than the radius of the curved path that a vehicle would be expected to travel through the roundabout. It is better to give approaching drivers a clear indication of the severity of the curve they will have to negotiate, since the speed at which drivers negotiate is dependent on their perception of the sharpness of the first curve. The entry radii should be designed tangential to the central island.

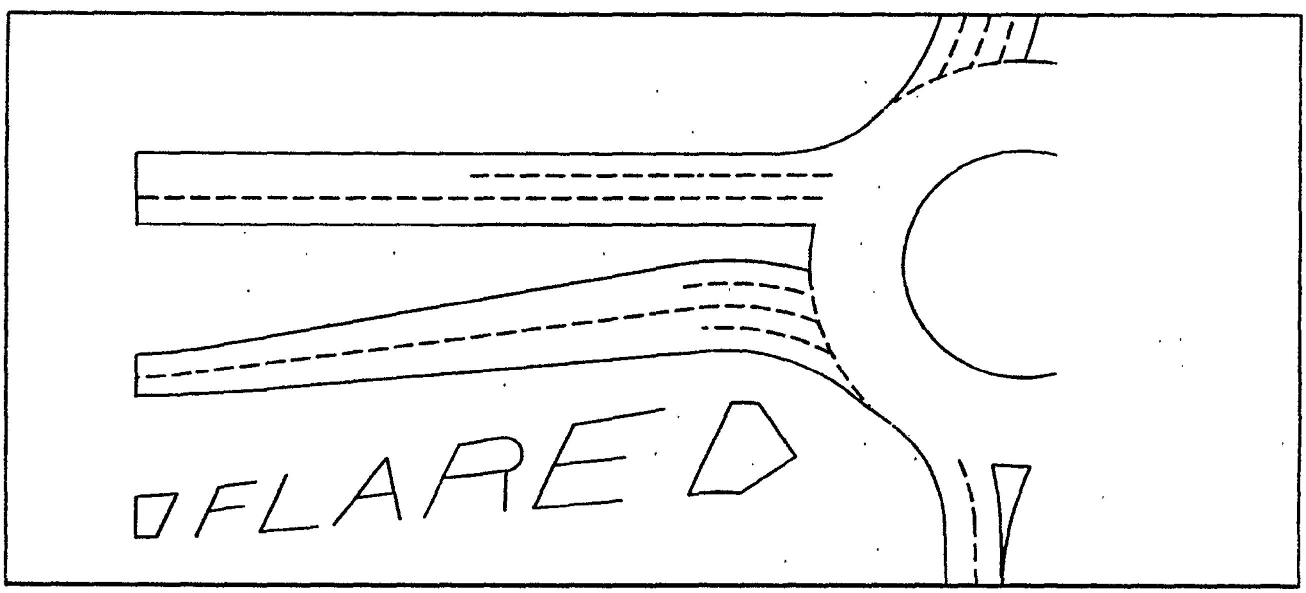


FIGURE 4.1 Flare design at entry

The entry radius should be a minimum of 50 feet for single lane roundabouts and 100 feet for multi-lane roundabouts. Small entry radii results in drivers reducing speed to a degree that drivers consider unreasonable or have difficulty in negotiating, or in drivers ignoring lane lines and cutting off vehicles in adjacent lanes. Figure 4.2 illustrates the components of entry design.

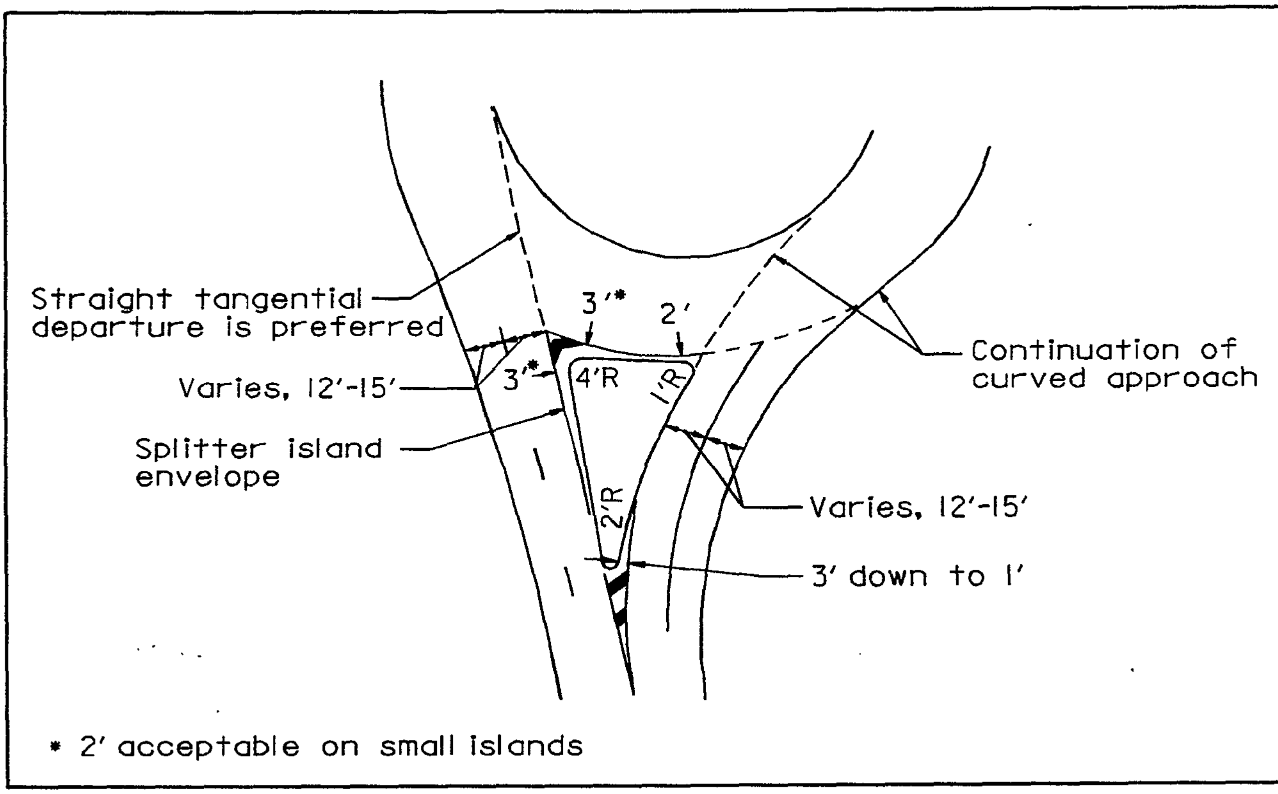


FIGURE 4.2 Typical Roundabout Entrance/Exit Conditions for Urban Areas

4.1.3 CIRCULATING WIDTH

The circulatory width should be constant and should be between 1.0 and 1.2 times the maximum entry width.

The circulating roadway should generally be circular in plan. Oval shaped roundabouts are acceptable (and preferred on roadways with wide medians or with unusual geometry) as long as deceptively tight bends are avoided. See Figure 4.3 for an example of an oval shaped roundabout.

It is a good design to avoid short lengths of reverse curve between entry and exits. It is difficult to achieve this on three-legged roundabouts or on roundabouts with skewed entries.

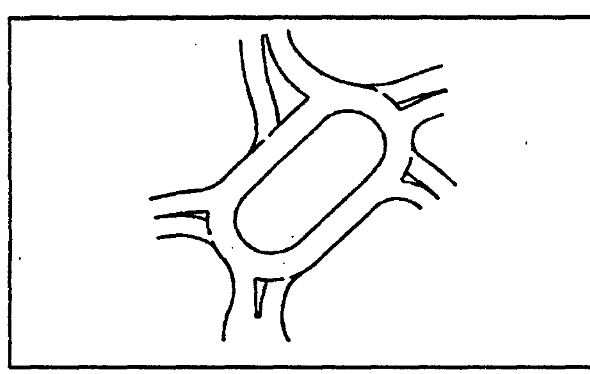


FIGURE 4.3 Oval roundabout

4.1.4 INSCRIBED CIRCLE DIAMETER

The size of the roundabout is a compromise between making it small enough to provide adequate deflection while making it large enough to provide for the appropriate design vehicles.

Maryland SHA has determined that the smallest inscribed circle diameter for a single lane roundabout is 100-feet on a state highway based on a WB-50 design vehicle. Roundabouts on smaller subdivision roads may be smaller depending on the maximum design vehicle. In all cases, the layout should be verified using the appropriate design vehicle template. See Figure 4.4.

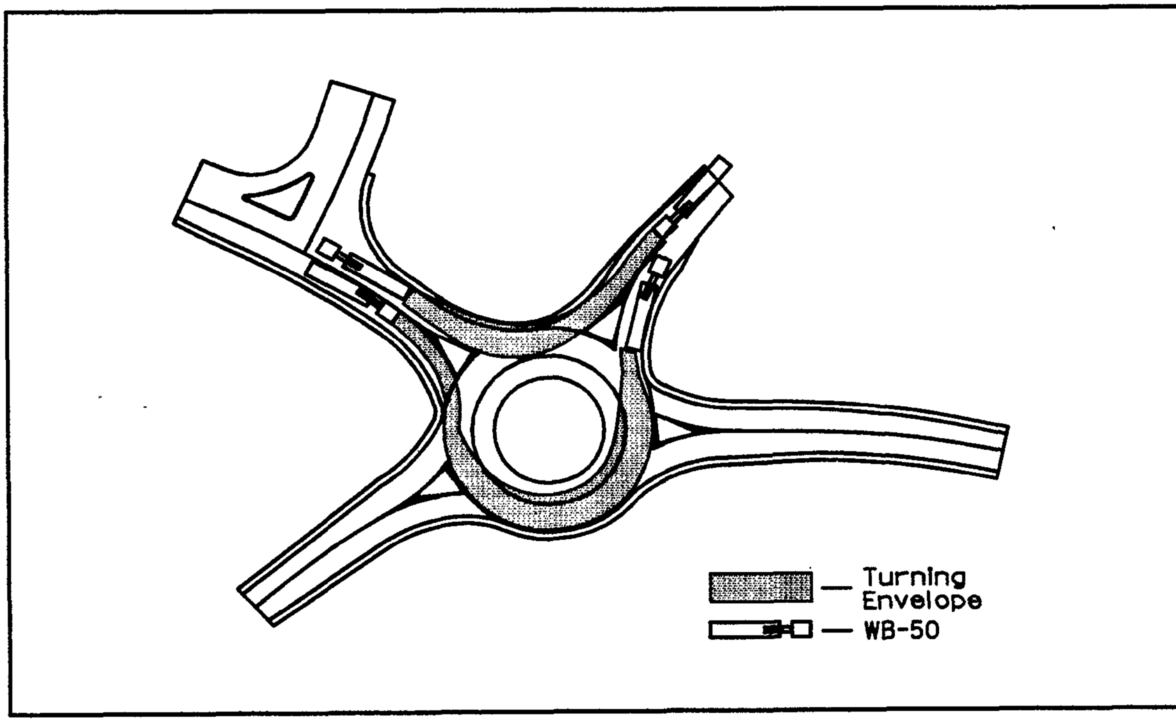


FIGURE 4.4 Turning templates for over-dimensional vehicles

4.1.5 EXITS

The exit from a roundabout should be as easy to negotiate as possible. Whereas entries are designed to slow vehicles, exiting vehicles should be able to accelerate out of the circulating roadway. Therefore, the exit radii should generally be greater than entry radii. Straight paths are preferred, if possible.

4.1.6 SPLITTER ISLANDS

Splitter islands should be provided on all roundabouts. They provide shelter for pedestrians, guide traffic into the roundabout, and deter left turns from dangerous short-cuts through the roundabout.

On arterial road roundabouts, the splitter islands should be of sufficient size to shelter a pedestrian and be a reasonable target to be seen by approaching traffic.

A long length of curve on the approach island allows drivers to more easily recognize the degree of curvature ahead. This is particularly so on divided roads and when approach speeds are high. However, care should be taken not to provide unnecessarily large radius curves as this could encourage high speeds through the roundabout.

The entry and exit curves from a roundabout form the spitter island envelope. Pavement markings and a raised island should be constructed within the spitter island envelope as shown in Figure 4.2.

In high speed areas the splitter island should be relatively long (200 feet ±) to give early warning to drivers that they are approaching an intersection and must slow down. Preferably the splitter island and its approach pavement markings should extend back to a point where drivers would be expected to normally start to reduce their speed. The lateral restriction and funnelling provided by the splitter island encourages speed reduction as vehicles approach the entry point. Curbs should be placed on the right-hand side for at least half the length of the splitter island to strengthen the funnelling effect. See Figure 4.5.

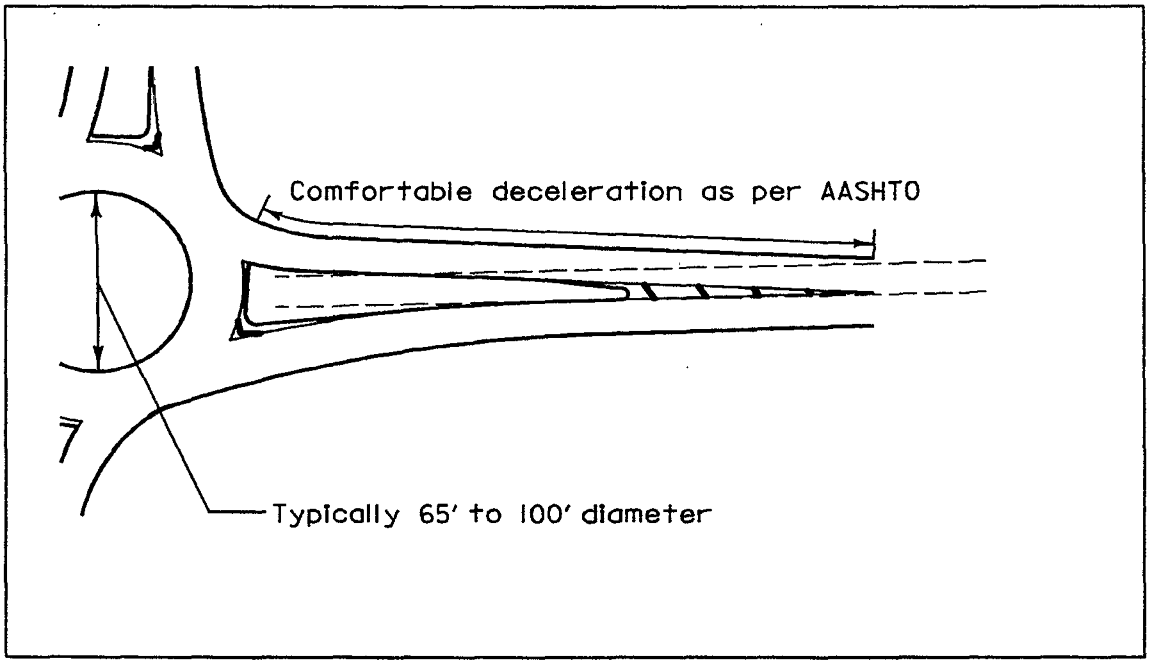


FIGURE 4.5 Typical Rural Roundabout Design (with High Speed Approach Roads)

4.1.7 DEFLECTION

Adequate deflection of the vehicle entering a roundabout is the most important factor influencing their safe operation. Roundabouts should be designed so that the speed of all vehicles is restricted to less than 30 MPH within the roundabout. This is done by adjusting the geometry of the entry and by ensuring that "through" vehicle paths are significantly deflected by one or more of the following means:

- The alignment of the entry and the shape, size and position of approach splitter islands (see Figure 4.6);
- Provision of a suitable size and position of central island;
- Introduction of a staggered or non-parallel alignment between any entrance and exit.

Deflection at Roundabouts with one Circulating Lane

The maximum desired "Design Speed" is obtained if no vehicle path (assumed 7 feet wide) has a radius greater than 430 feet. This radius of curvature corresponds approximately to a vehicle speed of 30 MPH assuming a sideways force of 0.2 g. The required deflection for a single lane roundabout is shown in Figure 4.7.

Deflection at Roundabouts with two or three Circulating Lanes

For multi-lane roundabouts (two or three circulating lanes), it is generally difficult to achieve the full deflection recommended above for single lane roundabouts. Where this is the case, it is acceptable for the deflection to be measured using a vehicle path illustrated in Figure 4.8. This differs from that used at single lane roundabouts in that the fastest (maximum radius) vehicle path is assumed to start in the right entry lane, cut across the circulating lanes and pass no closer than 5 feet to the central island before exiting the roundabout in the right lane.

Deflection at Roundabouts for various design speeds

For most state highway applications, design of the entries for 25-30 MPH deflection is acceptable. However, on minor state roads, county roads, and local roads, the designer may wish to create a slower entry condition. The following table illustrates the deflection curve for different entry conditions.

Design Speed (MPH)	Deflection Curve (FT)
12	60
15	100
20	180
25	290
30	430

Table 4.1 Deflection Curve Radii

4.1.8 SIGHT DISTANCE

Several sight distance criteria should be applied to the combination of vertical and horizontal geometrics at roundabouts. Those criteria which greatly influence the safety performance of a roundabout and also affect the positioning of signs and landscaping etc., are illustrated in Figure 4.9.

Criterion 1

The alignment on the approach should be such that the driver has a good view of both the splitter island, the central island and desirably the circulating carriageway. Adequate approach stopping sight distance should be provided, to the "Yield" lines and, as an absolute minimum, to the nose of the splitter island.

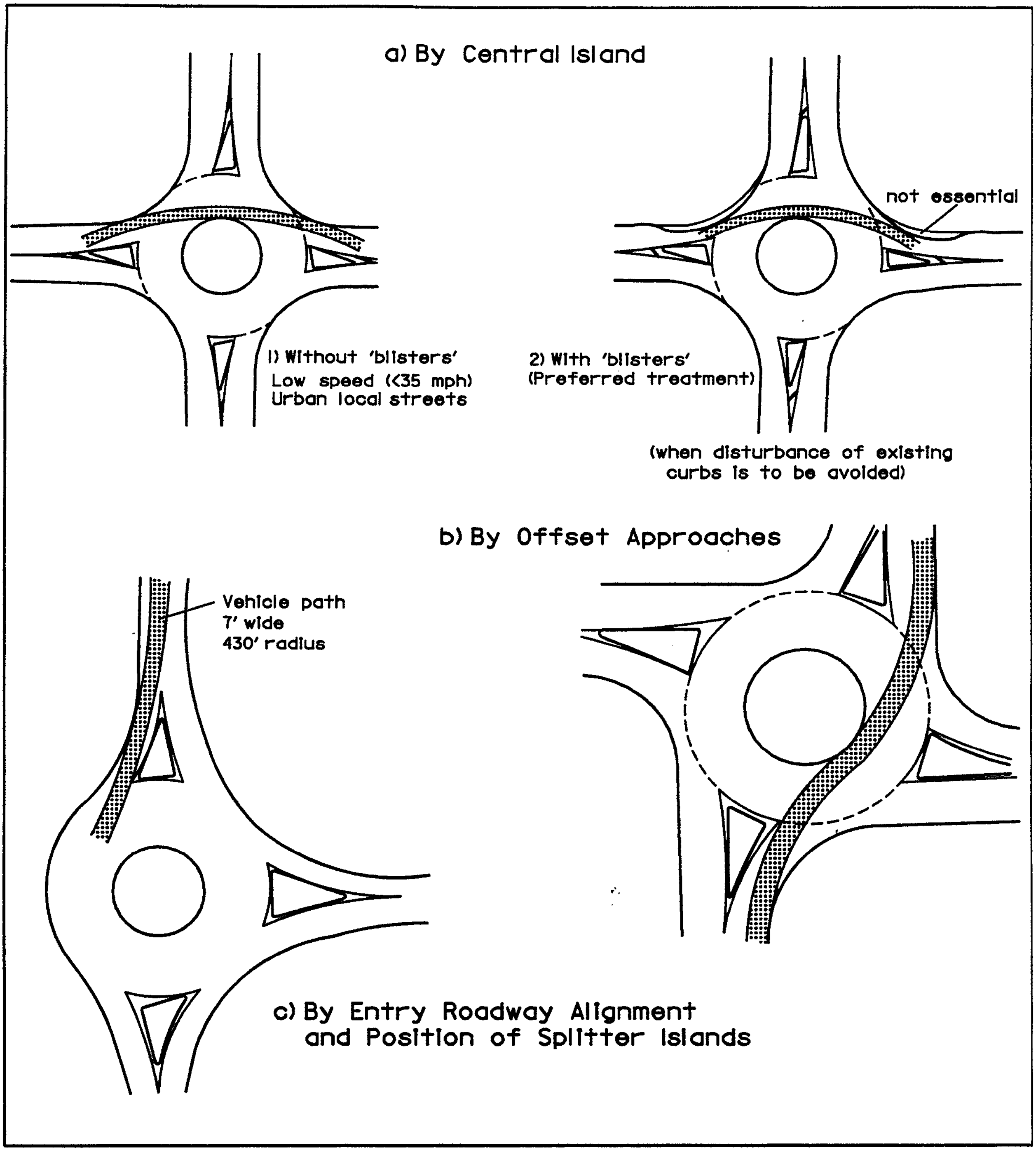


FIGURE 4.6 Alternative methods for providing vehicle deflection (not to scale)

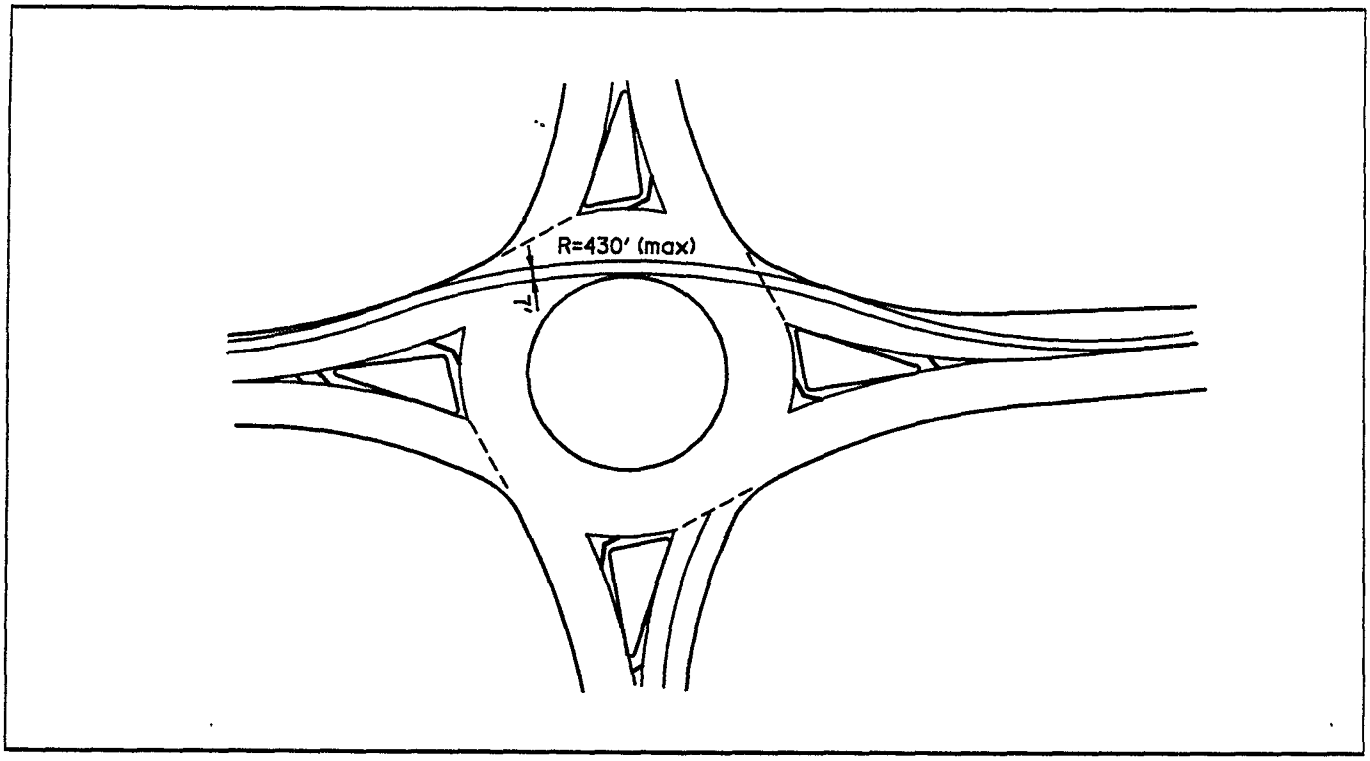


FIGURE 4.7 Illustration of the deflection criteria for a single lane roundabout

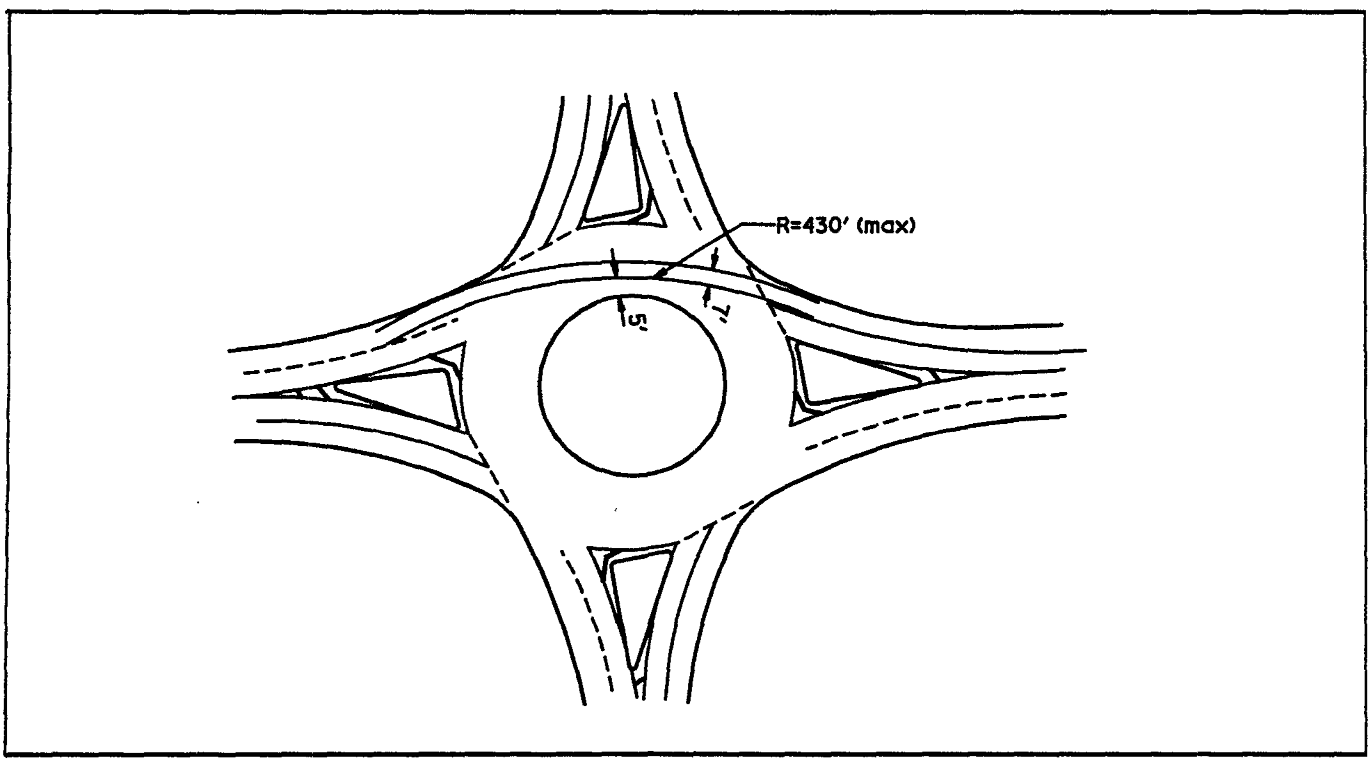


FIGURE 4.8 Illustration of the deflection criteria for a multi-lane roundabout

Table 4.1 indicates the required approach sight distances.

Approach Speed (MPH)	Stopping Distance* (ft)
25	98
31	131
37	180
43	230
50	344
56	426
62	525
68	623
75	754
* measured 4.0 ft to zero	

TABLE 4.2 Approach Sight Distance (ASD)

Criterion 2

A driver, stationary at the "yield" line, should have a clear line of sight to approaching traffic entering the roundabout from an approach immediately to the left, for at least a distance representing the travel time equal to the critical acceptance gap. A critical gap value of 5 s, giving a distance of 225 feet, (based on an entry speed of 30 MPH), would be typical for arterial road roundabouts operating with low circulating flows. At sites with higher circulating flows or in local streets, criterion 2 sight distance could be based on a critical gap of 4 s.

The criterion 2 sight distance should also be checked in respect to vehicles in the circulating roadway having entered from other approaches. The speed of these vehicles can be expected to be considerably less than 30 MPH and the corresponding sight distance to them (e.g. across the central island) should also be based on a critical gap of 4 s to 5 s. This represents a distance much less than 225 ft because of the low circulating speed of these vehicles. This is illustrated in Figure 4.9.

Criterion 3

It is also desirable that drivers approaching the roundabout are able to see other entering vehicles well before they reach the "yield" line. The 125 ft - 225 ft sight triangle shown in Figure 4.9 allows an approaching driver, slowed to 30 MPH, time to stop and avoid a vehicle driving through the roundabout at 30 MPH. It is desirable that this sight triangle be achieved, although in urban areas it may not always be possible. At roundabouts, the speed of vehicles is more controlled in the circulating roadway than on the approaches and if Criterion 3 sight distance is available to an approaching driver then any circulating driver in this zone would also be able to see an approaching vehicle.

Note that within the zones subject to Criteria 2 and 3, it is acceptable to allow momentary sight line obstructions such as poles, sign posts and narrow tree trunks.

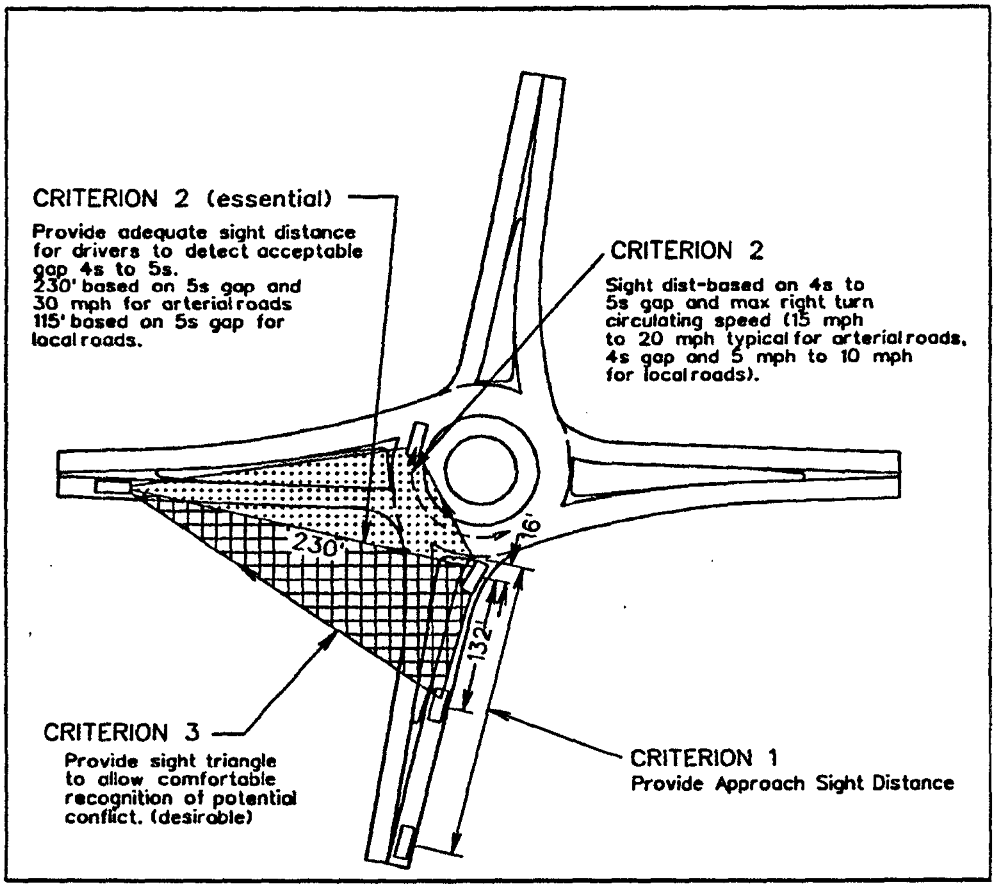


FIGURE 4.9 Sight distance requirements

4.1.9 OTHER VISIBILITY CONSIDERATIONS

At any roundabout, designers must provide the sight distance quantified and described above. A driver must also be provided with sufficient visibility to readily assess the driving task. The sight distance required for this is not precisely quantified and only general guidance can be given.

To enhance the prominence of the roundabout, the curbs on both the splitter island and central island may be light colored or painted white. To improve driver recognition, the central island may be mounded and/or reflectorized chevron pavers may be used, provided the overall height does not obstruct visibility or hide the drivers view of the overall layout.

As with other types of channelization, it is better to position a roundabout in a sag vertical curve rather than on a crest. Unlike other cross intersections, roundabouts require all drivers to change their path and speed, thus it would be important to avoid locating roundabouts just over a crest where the layout is obscured from the view of approaching vehicles.

At grade separated roundabouts, particularly where there may be a structure in the central island or a bridge railing which might obstruct a drivers' visibility, care must be taken to ensure that the sight distance requirements are met. Any guard fences used to protect piers and structures may also interfere with visibility.

Where there is a light rail crossing incorporated in to 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 tracks.

Light rail 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 recognized.

4.1.10 SUPERELEVATION AND DRAINAGE

Normal curve superelevation through the roundabout is generally not necessary as speeds are constrained and drivers tolerate higher values of the sideways force and utilize higher values of the coefficient of sideways friction when travelling through an intersection.

Above all, it is important that the layout of the roundabout be clearly visible to approaching drivers and this is best achieved by sloping the crossfall away from the central island. This generally means accepting negative superelevation for left turning and through vehicles in the circulating carriageway, but avoids depressing the central island thereby reducing its visibility to approaching traffic.

As a general design practice, a minimum pavement crossfall of 0.025 to 0.03 ft/ft should be adopted for the circulating carriageway. A crossfall as low as 0.02 ft/ft has been found adequate to allow for pavement drainage and would also provide additional driver comfort.

Designing superelevation to slope away from the central island often simplifies the detailed design of pavement levels and avoids inlets around the central island.

Exceptions to this approach include:

- Roundabouts on a generally sloping topography, in which case the crossfall should approximately match the slope across the whole of the roundabout. At these roundabouts the crossfall may vary around the circulating carriageway but it should stay within the range of ±0.04 ft/ft. Locating a roundabout on grades greater than 3 to 4 percent should be avoided. Where the general slope of the land is greater than 0.04 ft/ft, it will be necessary to "bench" the area for a roundabout, modifying the grade to not exceed a maximum negative crossfall of 0.03 ft/ft.

- Large roundabouts where vehicles will travel on the circulating roadway for some distance. In such cases, a crown following the center line of the circulating roadway may be satisfactory or it may be positively superelevated by sloping the roadway toward the central island. This improves driver comfort but tends to increase vehicle speed within the roundabout and reduces the visibility of the circulating roadway and the central island.

4.1.11 WIDE MEDIANS AND STREETS OF UNEQUAL WIDTH

Particular problems in roundabout design occur at locations where one intersecting street is considerably wider than the other and/or where a wide median exists. This situation can occur with local, collector or arterial streets or, as is often the case, where the intersecting streets are not of the same functional classification. Very often a roundabout will not be the appropriate type of treatment. However, where the volume of traffic on the narrower street is greater or equal to that on the wider street and if there are heavy left turn flows, a roundabout could be suitable.

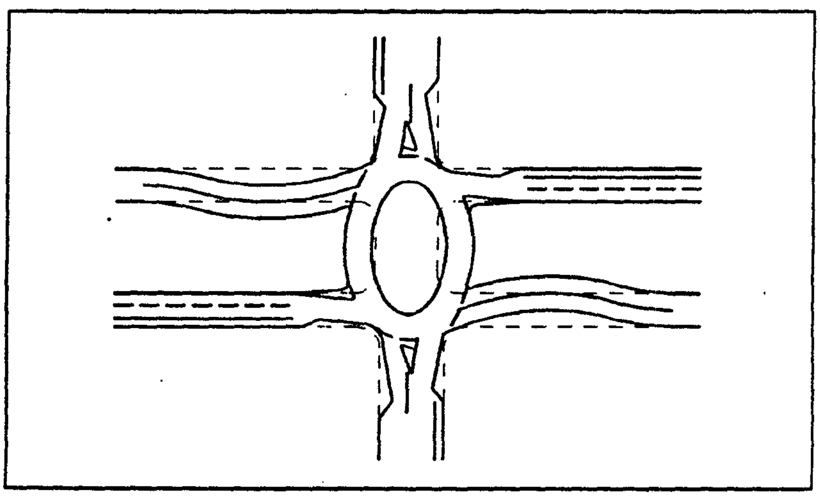


FIGURE 4.10 Roundabout on a road with a very wide median

Where a roundabout is proposed, special care should be taken to ensure that the design is in accordance with the guidelines listed in Sections 4.2.1 to 4.2.8. In particular, sufficient deflection for through traffic entering the roundabout is most important. Generally, a low cost solution which does not require improvements encroaching onto existing medians will not be possible. Figure 4.10 is an example of a roundabout designed to adequate standards for an in undivided road crossing a divided road with a wide median.

In these situations the central island is not circular and will involve different circulating speeds for different sections of the circulating roadway. Left turning drivers entering from the narrow road in Figure 4.10 will find that the radius of their turning path decreases and becomes more difficult. This may create a higher accident risk.

4.1.12 WIDE UNDIVIDED STREETS AND "T" INTERSECTIONS

Where a roundabout is to be constructed at an existing "T" junction, it is generally necessary to build out the curbline to provide deflection of the traffic movement across the top of the "T" opposite the terminating road. This practice has also been adopted at certain cross road intersections where one cross street is wider than the other and/or where there is space for more than one lane of traffic on a particular approach.

Where curblines are to be built out on approaches to roundabouts, special care should be taken to ensure that adequate delineation is provided, particularly in instances where there are no parked vehicles on the approach.

4.2 LOCAL STREET ROUNDABOUTS

The major differences in the geometric treatment of local street roundabouts, compared with roundabouts on state routes, arise from the differing design aims, the generally narrower street widths, the lower traffic speeds applicable, and the smaller class of vehicle using the facility. Typical vehicles to be designed for include cars and occasional single unit trucks.

Roundabouts are usually installed in the local street system as a traffic management device to improve safety and amenity by controlling vehicle speed and, due to their restrictive geometry, to create a deterrent for large vehicles and high traffic flows. The traffic demand in local streets is usually low and capacity and delay calculations are not required. In most if not all cases, only one entry lane (on each approach) and one circulating lane is provided.

The geometric design principles for local street roundabouts differ slightly from those used for rural and arterial road roundabouts. The control of vehicle speed on entry and through the roundabout remains an important objective as with roundabouts on arterial roads. However, in the layout of local street roundabouts, the pavement space provided for vehicle maneuvers usually comprises areas normally required for cars and light commercial vehicles and specially paved encroachment areas (which may be slightly raised), to cater for the few larger vehicles which may need to use the site. These larger vehicles may be required to drive over traversable islands or rumble strip treatments in the splitter islands to negotiate the roundabout.

5.0 LANDSCAPING

5.1 GENERAL

A roundabout creates new design opportunities within an intersection:

- Roadside Planting.
- Planting in the central island.
- Creative pavements in truck aprons, splitter island, circling lanes and pedestrian crosswalks.

However certain goals should be kept in mind as the landscaping design of a roundabout is developed.

- Improve the aesthetics of the area.
- Avoid introducing hazards to the intersection.
- Avoid obscuring the form of the roundabout layout to the driver.
- Maintain required stopping and turning sight distances (see Figure 4.9).
- Clearly indicate to the driver that they cannot pass straight through the intersection.
- Minimize oncoming headlight glare.
- Discourage pedestrian traffic through the central island.

5.2 ROADSIDE PLANTING

Plant material can be added to the approach roadways to help in the funnelling effect. The funnelling effect is discussed in Section 4.1.6 on arterial road roundabouts. The lateral restriction and funnelling provided by the splitter island encourages speed reduction. Plant material on the right an left side of the approaches reinforces this effect. This illustrated in Figure 5.1.

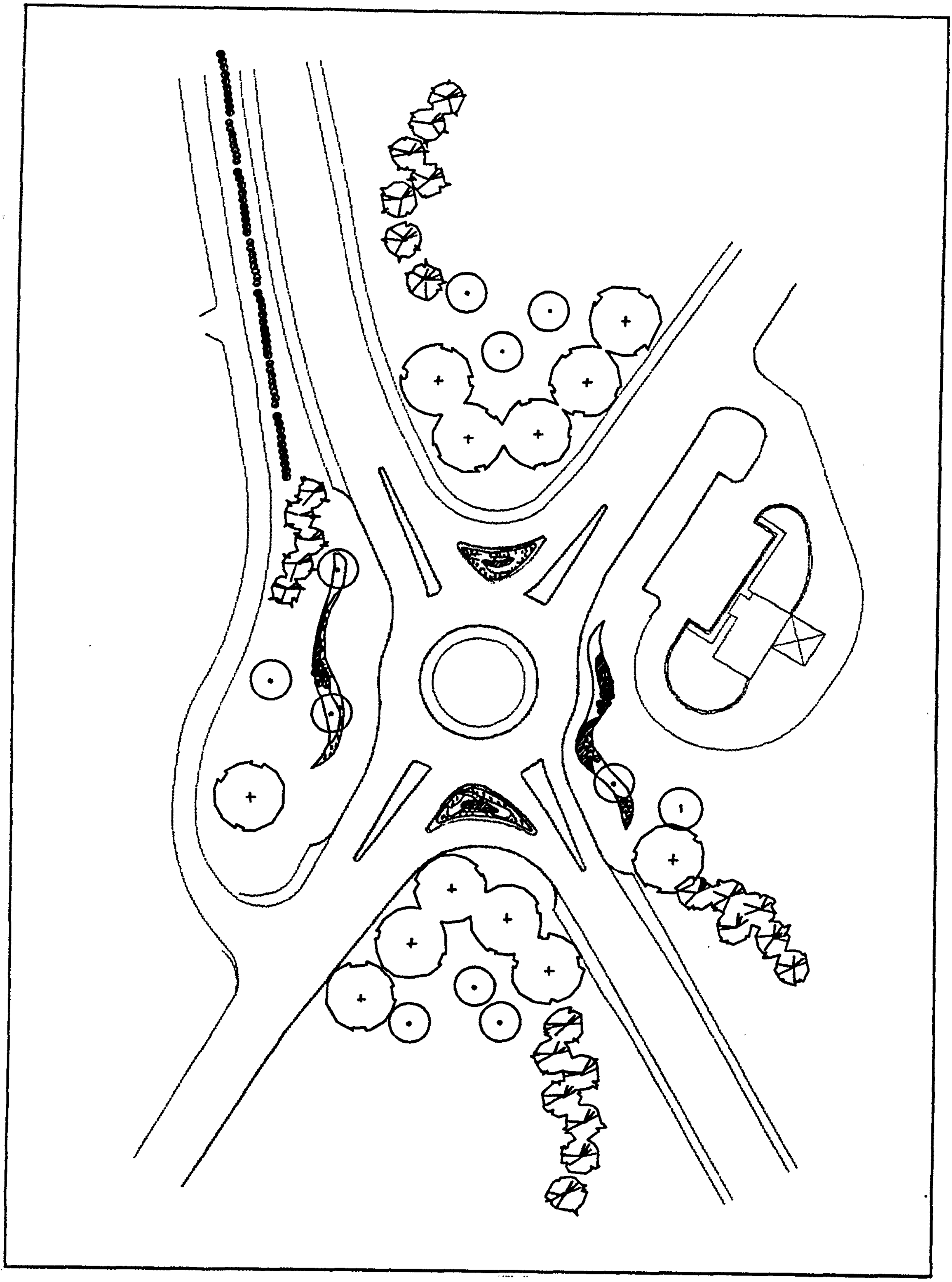


FIGURE 5.1 Example of the use of landscaping to reinforce the funnelling effect at the entrance to roundabouts

5.2 LANDSCAPE DESIGN FEATURES

As previously mentioned, the introduction of the roundabout to the state highway system creates new opportunities for landscaping.

In addition to the central island planting, plant material can be added to the approach roadways to help in the funnelling effect. The funnelling effect is discussed in Section 4.1.6 on arterial road roundabouts. The lateral restriction and funnelling provided by the splitter island encourages speed reduction. Plant material on the right and left side of the approaches reinforces this effect. This is illustrated in Figure 5.1.

Where truck aprons are used (As shown in section 4.1.4) and in the splitter islands, creative pavement colors, textures and markings can be used to blend the roundabout in with the existing surrounding area. This is illustrated in Figures 5.2 and 5.3.

Finally, when planting in an old road bed, as for the central island, it is necessary to excavate the old road bed out (to a depth two feet below the original road surface) and backfill with approved topsoil to provide for adequate growing conditions.

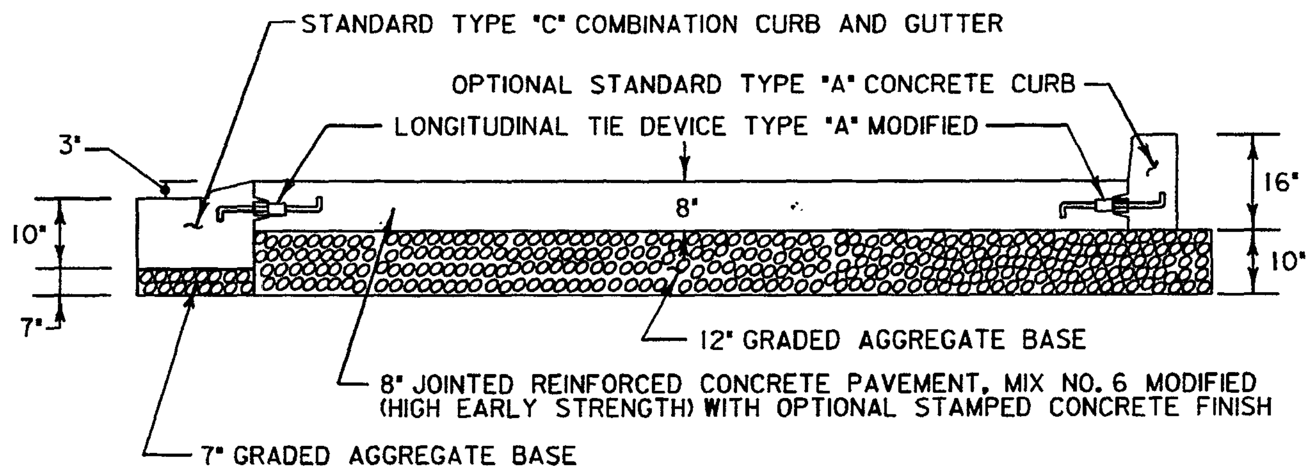


FIGURE 5.2 Typical Section of the Truck Apron

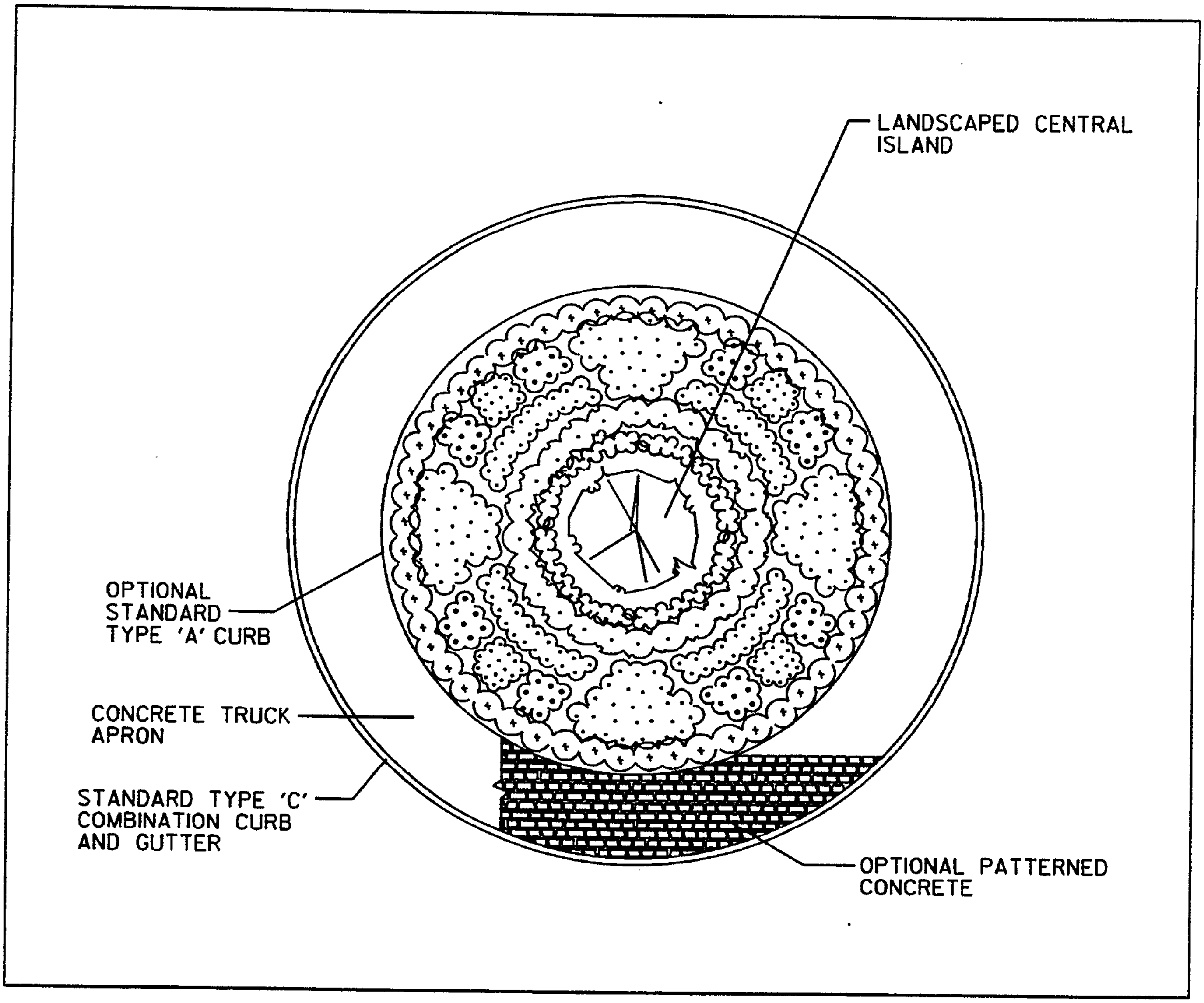


FIGURE 5.3 Plan of Central Island

6.0 SIGNING AND PAVEMENT MARKING

6.1 SIGNING

The general concept for Roundabout signing is similar to signing any other new geometric feature along a highway. Proper advance warning, directional guidance and regulatory control are required to avoid driver expectancy related problems. The guidelines set forth in the Manual on Uniform Traffic Control Devices for Streets and Highways, the State of Maryland - Standard Highway Signs booklet, and the various memorandums issued by the Maryland Department of Transportation - State Highway Administration govern the design and placement of signs along all roads in the state. Standards developed by the individual counties that supplement the state guidelines may also apply to the design and placement of signs along roads within those jurisdictions.

Roundabout signing varies based upon the type of roadways intersecting at the roundabout. The division is made between Highways and Local roads, as explained below.

- **HIGHWAY** - State highways and County collector roads
- **LOCAL ROAD** - Other County roads, commercial and residential

6.11 HIGHWAY

Approaches:

1. Junction Assemblies should be used.
2. "ROUNDABOUT AHEAD" Warning signs with "YIELD AHEAD" plates should be used.
3. Destination Guide signs should be used. For higher speed multi-lane approaches (> 45 MPH) Diagrammatic Guide signs should be considered.
4. "YIELD AHEAD" (W3-2A) signs in combination with Advisory Speed (W13-1) plates should be used.
5. Where possible, the designer should attempt to reuse any appropriate existing signs.
6. Other guide signs, such as Advance Route Marker Turn Assemblies may be used as described in the MUTCD and SHA sign guidelines.

Intersections:

1. "YIELD" (R1-2) signs in combination with "TO TRAFFIC ON LEFT" (RX-X) plates should be used.
2. "ONE WAY" (R6-1R) signs in combination with obstruction markers (W15-2) should be used.

3. Exit Guide signs should be used.

The locations of these signs are shown in Figure 6.1.

In high speed, rural areas extraordinary warning signs may be used. These signs are to be used only if approved by the Director, Office of Traffic & Safety. In cases where high speeds are expected, and the normal signage and geometric features are expected to have less than desired effect on vehicle speeds, the following measures may be considered:

- Addition of Hazard Identification Beacons to approach signing (Diagrammatic, Roundabout Ahead).
- Speed Display Signs (actuated by speed detectors).

6.12 LOCAL ROADS

Each approach should include:

1. "ROUNDABOUT AHEAD" Warning sign with "YIELD AHEAD" plate.

Each approach may include:

2. Destination Guide sign.

The intersection should include:

1. "YIELD" (R1-2) sign in combination with "TO TRAFFIC ON LEFT" (RX-X) plate.

The intersection may include:

2. "ONE WAY" (R6-1R) sign in combination with obstruction marker (W15-2).

3. Exit Guide sign with "DO NOT ENTER" (R5-1) sign mounted on back.

The locations of these signs are shown in Figure 6.2.

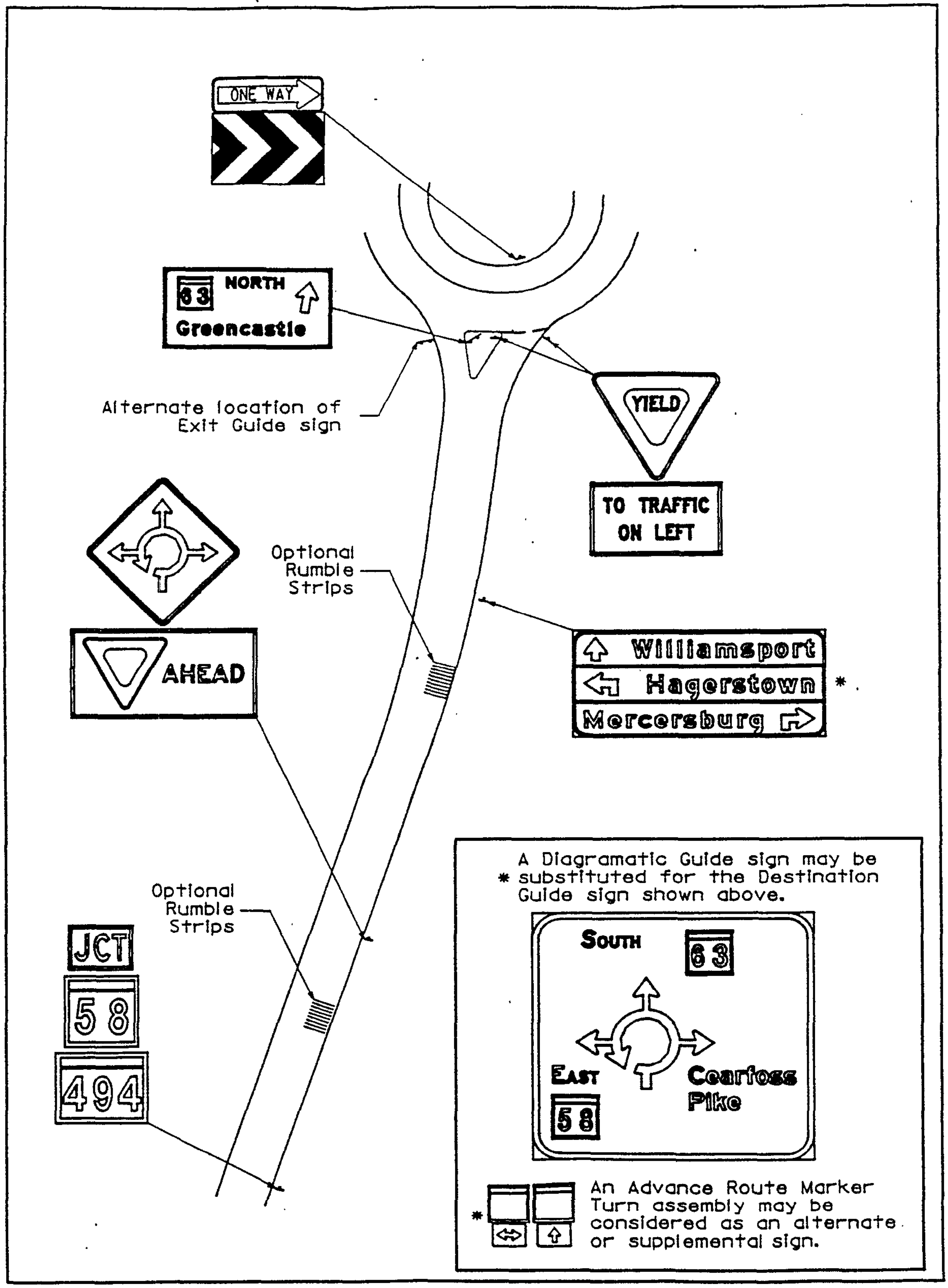


FIGURE 6.1 Typical signing for a state route roundabout

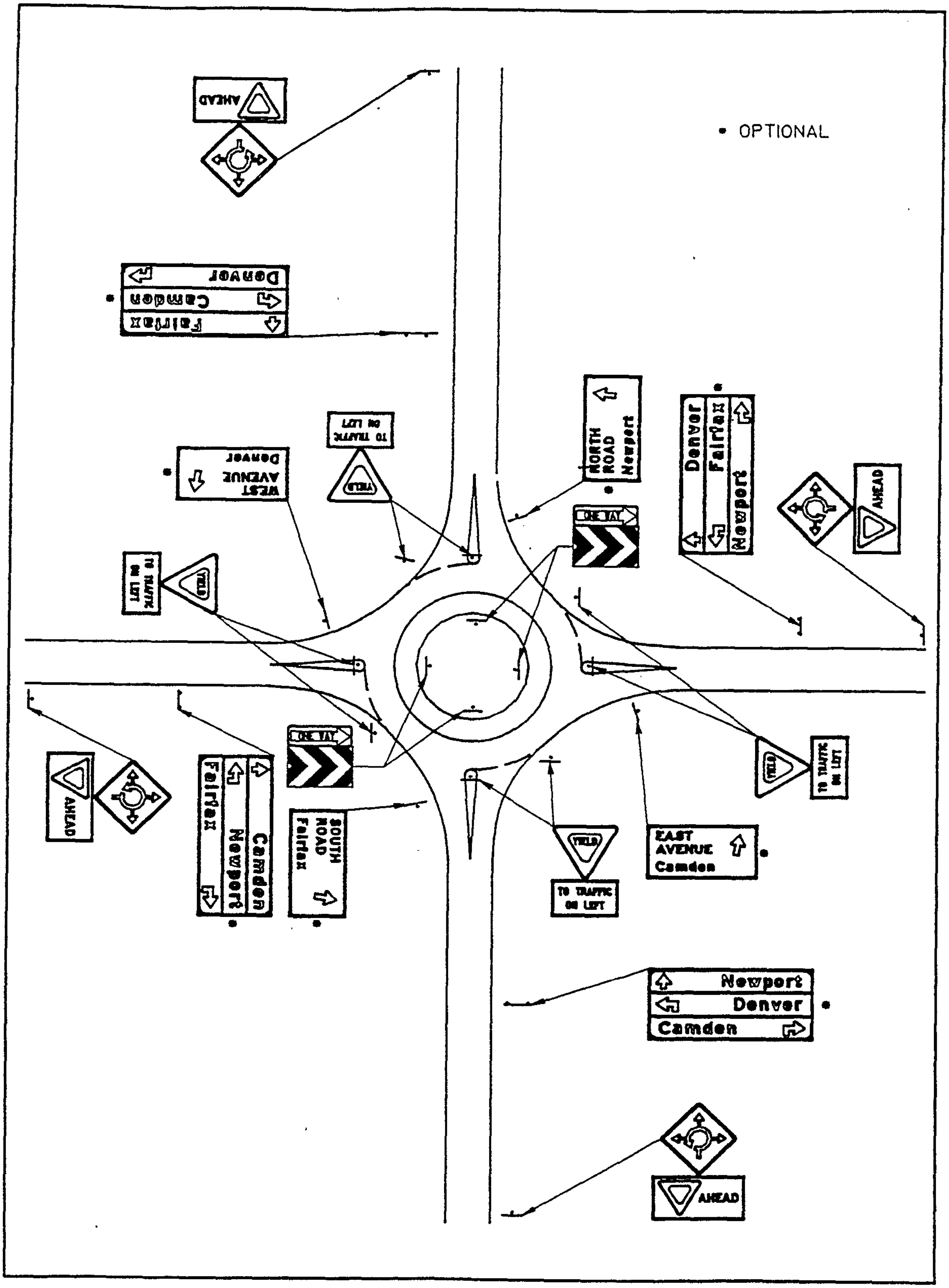


FIGURE 6.2 Typical signing for a local road roundabout

6.2 PAVEMENT MARKINGS

Pavement markings for a roundabout intersection consist of the yield line, hatch markings in the splitter island envelope, raised reflective pavement markings and typical edge and lane line striping as shown in Figure 6.3.

The pavement markings at the entrance to the roundabout consist of an 8" to 16" wide stripe with 3-foot segments and 3-foot gaps. A yield line placed 8' to 10' in advance of the entrance to the roundabout is shown in Figure 6.3. There shall be no painted lines across the exits from roundabouts.

While there is not conclusive evidence as to whether there should be lane lines delineating the circulating lanes within the roundabout, it is felt that such pavement markings may confuse rather than help drivers in the performance of their task of negotiating the roundabout. With multi-lane roundabouts, pavement markings may also mislead drivers into thinking that vehicles must exit exclusively from the outer lane of the roundabout. The use (or lack of use) of pavement markings will be made on a case by case basis.

The pavement marking may be emphasized by the placement of raised reflective pavement markings as shown in Figure 6.3. Thermoplastic markings may be used to increase the pavement marking visibility.

Rumble strips may be utilized to reduce approach speeds and to call the drivers' attention to warning or destination guide signs. They are especially useful on high speed rural approaches. An example of an application of rumble strips is shown in Figure 6.1.

The rumble strips consist of 10 foot crosswise stripes of 4 inch permanent preformed pavement marking tape. Double applications of the 4 inch permanent preformed pavement marking tape are typically required to produce the desired effect (i.e. one layer of the tape is applied to the roadway and the second layer is applied directly on top of the first layer). The spacing of the strips varies directly with the approach speed.

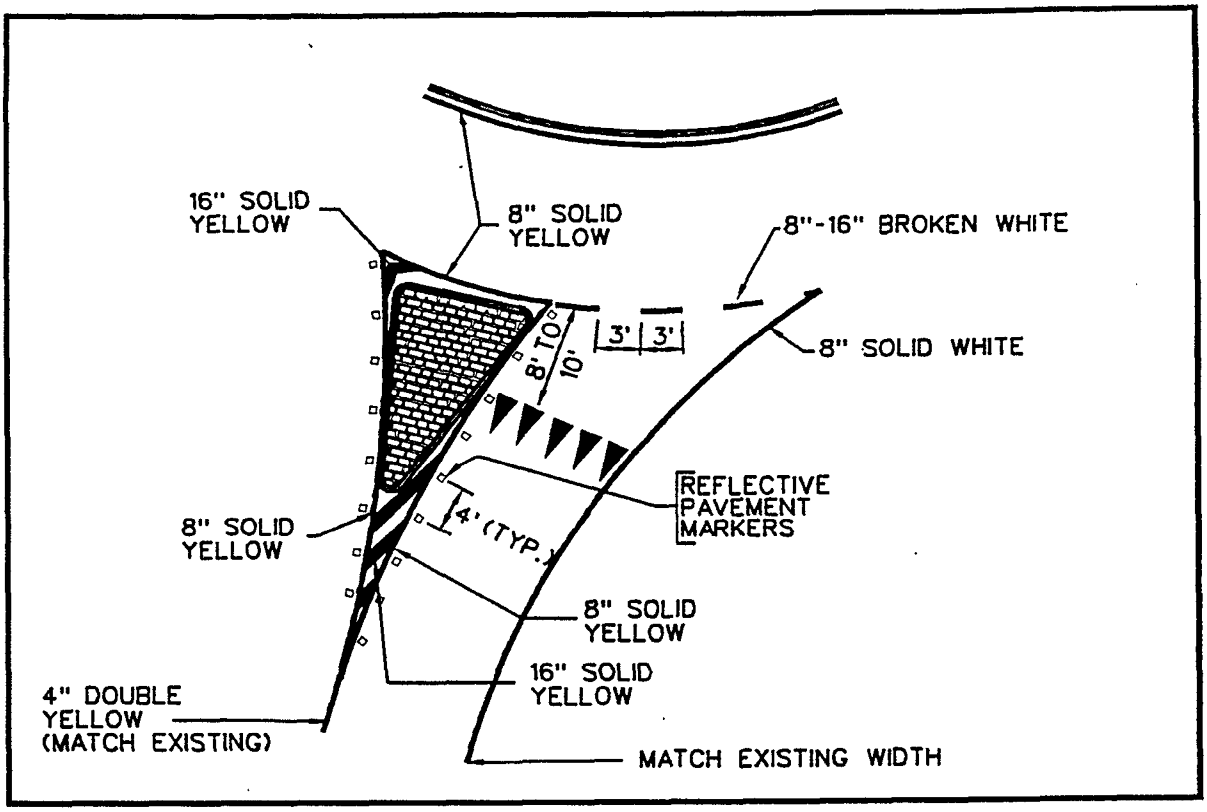


FIGURE 6.3 Typical pavement markings for roundabouts

7.0 LIGHTING

The satisfactory operation of a roundabout relies heavily on the ability of drivers to enter into, and separate safely and efficiently from a circulating traffic stream. To do this, it is important that the driver must perceive the general layout of the intersection in sufficient time.

It is not possible to provide detailed recommendations on lighting layout for roundabouts because of the great variety of possible geometric layouts. It is important, however, to recognize certain desirable features:

A. Lights should be located so that they provide good illumination on the approach nose of splitter islands, the conflict area where traffic enters the circulating stream, and at places where traffic streams separate at points of exit.

B. Particular attention should be given to the lighting of the pedestrian crossing areas if applicable.

C. Lighting poles should not be placed within splitter islands, on the central island directly opposite an entry roadway, or on the right-hand perimeter immediately downstream of an entry point.

8.0 PEDESTRIAN AND BICYCLE CONSIDERATIONS
8.1 PEDESTRIANS

In the planning and design of roundabouts, special thought should be given to the movement of pedestrians. In general, research indicates fewer pedestrian accidents at roundabout intersections when compared to signalized and unsignalized intersections. This is so for two reasons. First, the speed of all vehicles is slower at a roundabout intersection. Second, pedestrians use the splitter island as a refuge area. In so doing, the pedestrian only crosses one stream of traffic at a time. The normal pedestrian crossing location should be at the yield line as shown in Figure 8.1 (a).

Pedestrian crossing lines should not be painted on the entrances and exits of roundabouts as they may give pedestrians a false sense of security. Pedestrians should be encouraged to identify and accept gaps in traffic and to cross when it is safe to do so.

The normal placement of pedestrian crossings at roundabouts should be 20-25 feet from the yield line. Crosswalk striping should not be used because the driver may confuse the crosswalk limit lines with the yield lines. The pedestrian crossing could be reinforced with handicapped ramps and/or colored and patterned concrete.

Consideration should be given to providing priority crossings for pedestrians where pedestrian volumes are very high, where there is a high proportion of young, elderly or infirm citizens wanting to cross the road, or where pedestrians are experiencing particular difficulty in crossing and are being delayed excessively. It is desirable that these crossings be placed at least 75 feet downstream of the exit from the roundabout (and possibly be augmented by pedestrian signals). This will reduce the probability that vehicles delayed at the pedestrian crossing will queue back into the roundabout and gridlock the whole intersection. The locations of these pedestrian crossings are shown in Figure 8.1 (b).

8.2 CYCLISTS

In most circumstances, roundabouts provide satisfactorily for cyclists, although it has been found that multi-lane roundabouts are more stressful to cyclists than single roundabouts owing to the greater chance of conflicts between vehicles and cyclists. It has been found that generally, cyclists use roundabouts in a similar way to motor vehicles. Special provisions for cyclists are not normally required.

To provide a satisfactory level of safety for cyclists at roundabouts, particular attention will need to be taken in the layout design to:

- ensure that adequate deflection and speed control is achieved on entry and through the roundabout.
- avoid larger than necessary roundabout (inscribed) diameter, thus reducing travel speed through the roundabout.
- avoid excessive entry widths and alignments which can also increase vehicle entry speeds.

- ensure that sight lines are not obstructed by landscaping, traffic signs or poles which may even momentarily obscure a cyclist.

- provide adequate lighting.

Normally, special bicycle lanes will not be required as the cyclist would be able to proceed through the roundabout in the travel lane. If high volumes of bicycle traffic exist, a special bicycle/pedestrian facility could be constructed as shown in Figure 8.2.

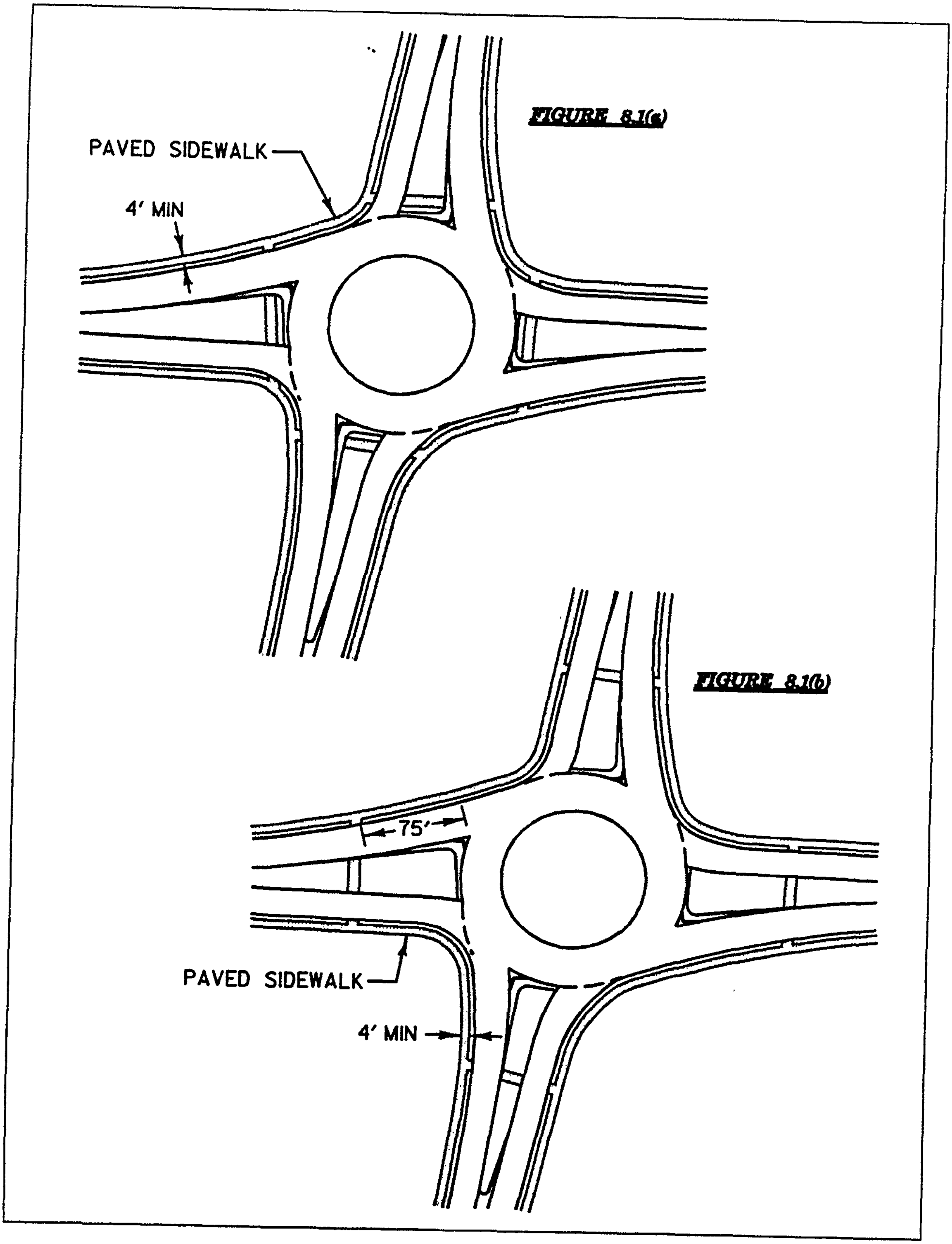


FIGURE 8.1 Examples of Pedestrian Crossings

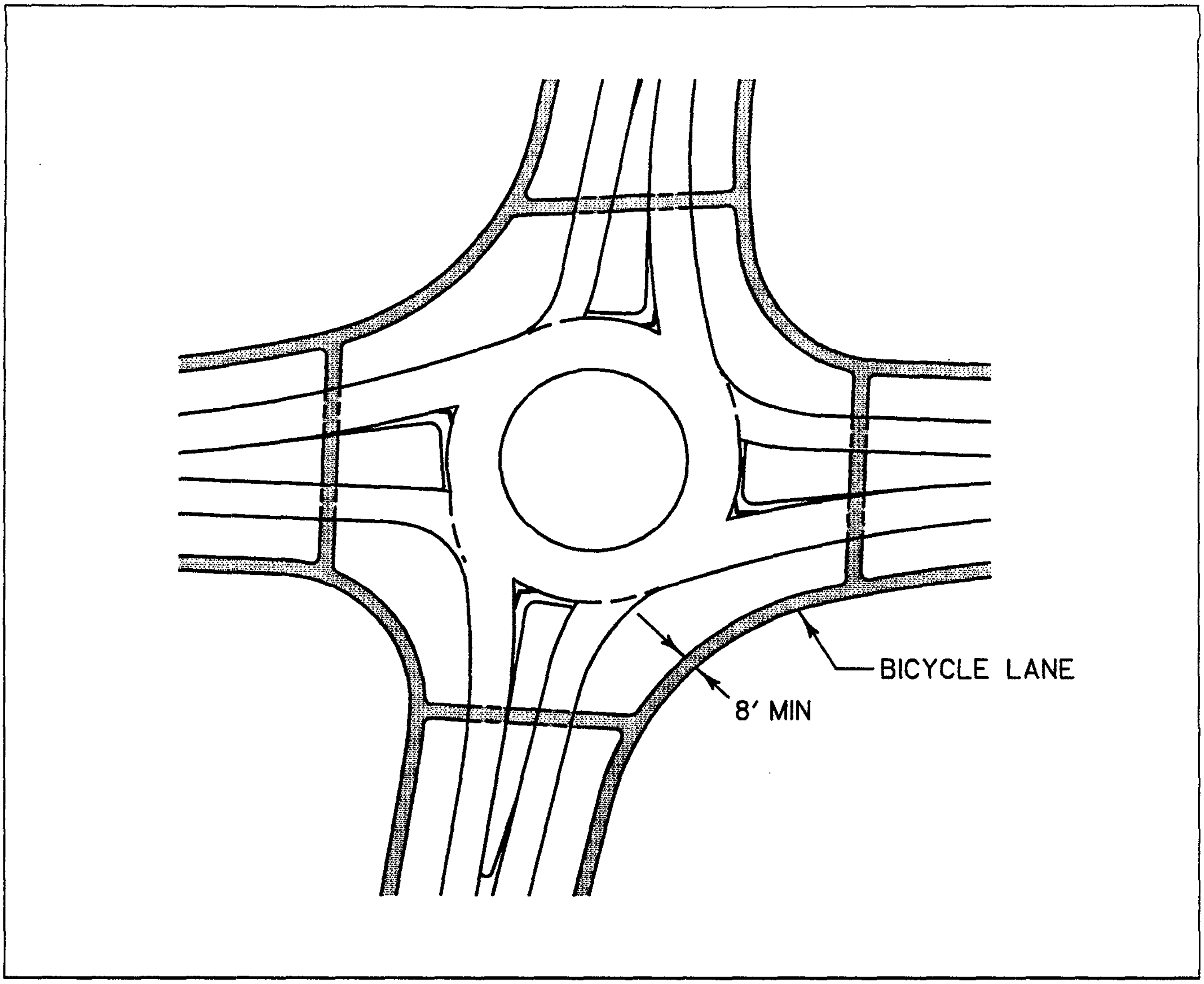


FIGURE 8.2 Example of a special bicycle facility

9.0 WORK ZONE TRAFFIC CONTROL

During the construction of a roundabout it is essential that the intended travel path be clearly identified. This may be accomplished through pavement markings, signing, delineation, and guidance from police and/or construction personnel depending on the size and complexity of the roundabout. Care should be taken to minimize the channelizing devices so that the motorist has a clear indication of the required travel path. Each installation should be evaluated separately as a definitive guideline for the installation of roundabouts is beyond the scope of this policy.

9.1 PAVEMENT MARKINGS

The pavement markings during construction should be the same layout and dimension as those used for the final installation. Because of the confusion of a work area and the change in traffic patterns, additional pavement markings may be used to clearly show the intended direction of travel. In some cases when pavement markings cannot be placed, channelizing devices should be used to establish the travel path.

Temporary raised pavement markers (TRPM's) should be used to supplement pavement markings. Spacing and positioning of the TRPM's should be such as to clearly delineate the intended travel path but not be in such quantities as to detract from the pavement markings. (See Figure 9.1)

9.2 SIGNING

The signing during construction shall consist of all necessary signing for the efficient movement of traffic through the work area as described in Figure 9.2, pre-construction signing advising motorists of the planned construction, i.e. "CONSTRUCTION OF A ROUNDABOUT TO BEGIN...", and any regulatory and warning signs necessary for the movement of traffic outside of the immediate work area.

9.3 LIGHTING

Permanent lighting, as described in section 7.0, should be used to light the work area. If lighting will not be used, delineation, as described in section 9.1, should be used.

9.4 CONSTRUCTION STAGING

As is the case with any construction job, before any work can begin, all traffic control devices should be installed as indicated in the traffic control plan or recommended typical. This signing shall remain in place as long as it applies and then removed when the message no longer applies to the condition.

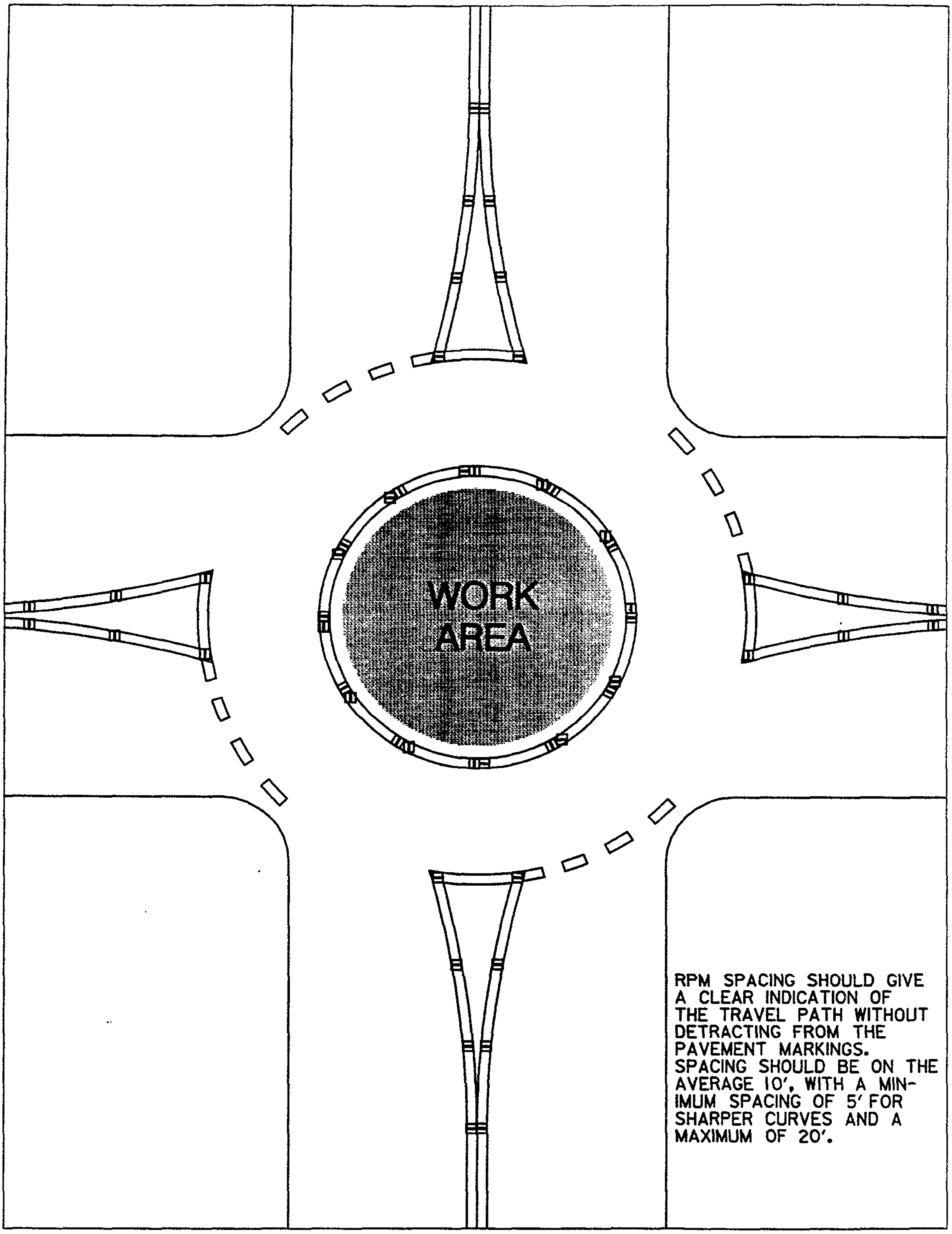
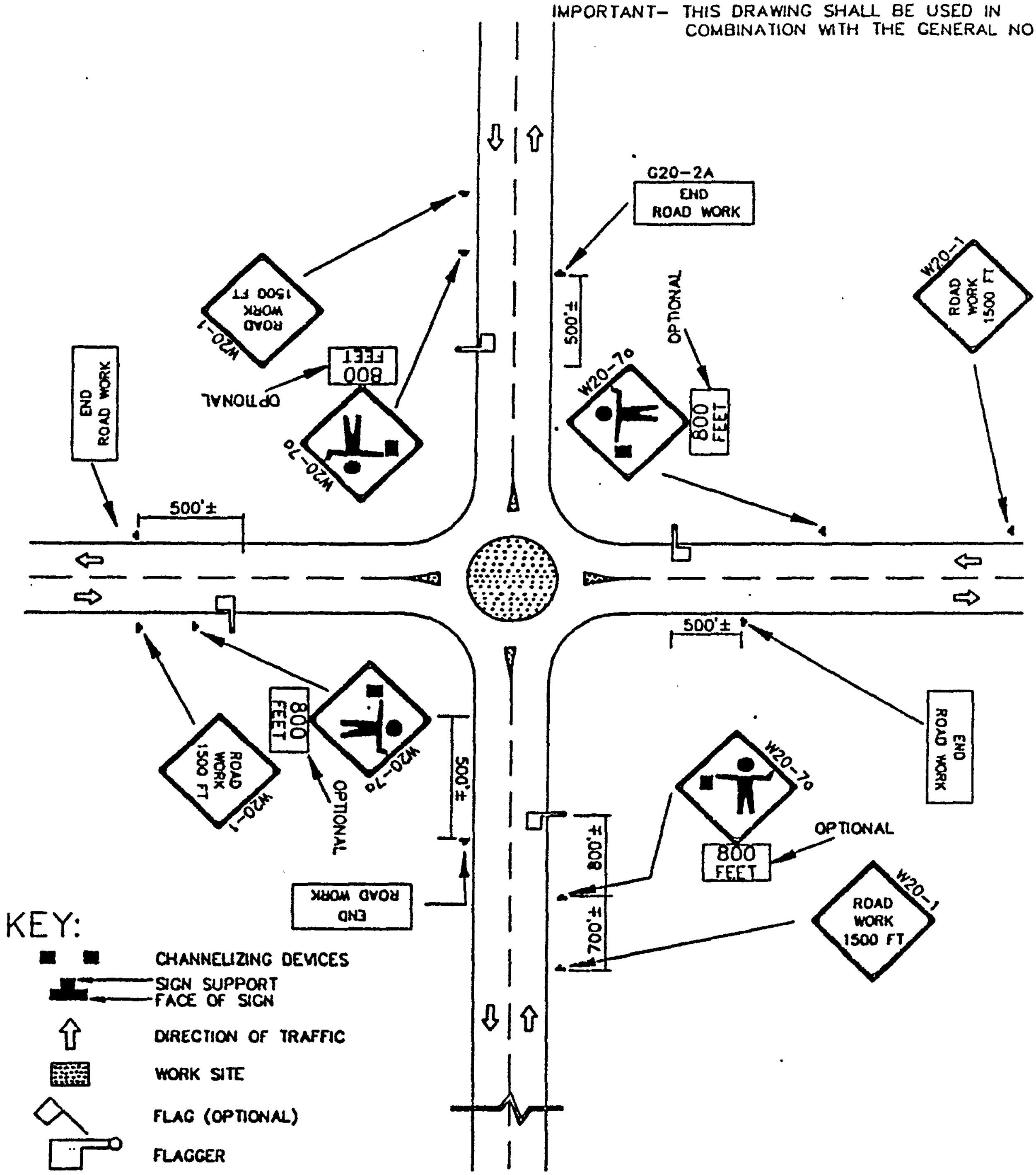


FIGURE 9.1 Roundabout workarea pavement markings

WORK ZONE TRAFFIC CONTROL TYPICAL

IMPORTANT– THIS DRAWING SHALL BE USED IN COMBINATION WITH THE GENERAL NOTES.



SPECIFICATION	CATEGORY CODE ITEMS	Maryland Department of Transportation STATE HIGHWAY ADMINISTRATION ROUNDABOUT FLAGGING OPERATION GREATER THAN 40 MPH/OVER 24 HRS.
APPROVED ______ DIRECTOR, OFFICE OF TRAFFIC AND SAFETY		
	APPROVAL • SHA REVISIONS	APPROVAL • FEDERAL HIGHWAY ADMINISTRATION
	APPROVAL	APPROVAL
	REVISED	REVISED
	REVISED	REVISED
	REVISED	REVISED
		STANDARD NO.

FIGURE 9.2 Work Zone Traffic Control

Prior to the work which would change the traffic patterns to that of a roundabout, certain peripheral items may be completed. This would include permanent signing (covered), lighting, and some pavement markings. These items, if installed prior to the construction of the central island and splitter islands would expedite the opening of the roundabout and provide additional safety (lighting) during construction.

When work has commenced on the installation of the roundabout, it is desirable that it be completed as soon as possible to minimize 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 linemarking should be done in one day.

If it is necessary to leave a roundabout in an uncompleted state overnight, the splitter islands should be constructed before the central island. Any portion of the roundabout that is not completed should be outlined with pavement markings and delineation in such a way as to clearly outline the intended travel path. (see Figure 9.3).

In general, the order of construction should be as follows:

1. Install and cover proposed signing.
2. Construct outside widening if applicable.
3. Reconstruct approaches if applicable.
4. Construct Splitter Islands and delineate central island. At this point the signs should be uncovered and the intersection should operate as a roundabout.
5. Finish construction of the central island.

9.5 Public Education

It is important to educate the public anytime there is a change in traffic patterns. It is especially important for a roundabout because a roundabout will be new to most motorists. The following are some suggestions to help alleviate initial driver confusion.

1. Public meetings prior to construction.
2. News releases/handouts detailing what the motorist can expect before, during, and after construction.
3. Variable Message signs during construction.
4. Travellers Advisory Radio immediately prior to and during construction to disseminate information on "How to drive" etc.

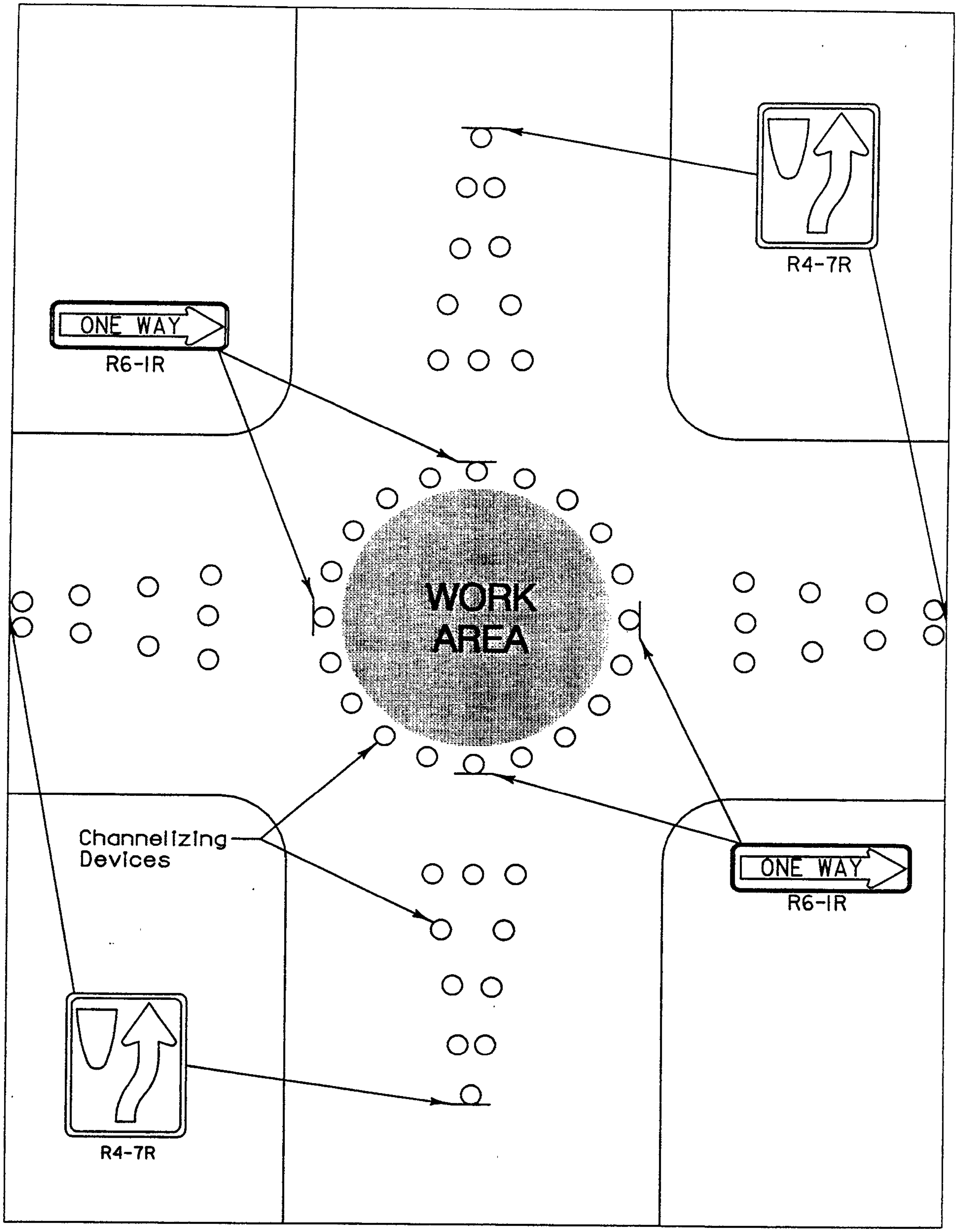


FIGURE 9.3 Roundabout workarea delineation

APPENDIX

BENEFIT/COST ANALYSIS

1. Accident data is collected for a three year period. This allows a large amount of data to be collected and insures an high probability that collision types related to the design and operational features of the location are represented proportionately to their occurrence.

2. Traffic volume figures are developed for each year studied for the purpose of computing accident rates by type of collision. Based upon the three year average accident rate by each collision type and using linear regression to project the traffic volume to the improvement completion year we can estimate what the projected accident frequency by collision type will be for the improvement completion year.

3. The proposed improvements are based upon the accident experience that has manifested itself during the three year study period. After an improvement is selected, the effectiveness of the improvement is measured by estimating the extent it will reduce a specific type of accident. Since roundabouts are a relatively new traffic control device, the accident reduction factors are assumed and will be monitored upon completion of Before/After studies when a statistically significant number of roundabouts have been installed. Primarily roundabouts have been found to reduce angle and left turn accidents. The accident reduction factor for each type of accident is as follows:

Angle	:	0.5
Left Turn	:	1.0

4. The effectiveness quotients are summed to determine the total number of accidents reduced. The composite figure is then multiplied by the average accident cost for each collision. This figure is what we identify as the First Year's Benefit (FYB). The first year's benefit derived from the improvement cannot in itself be used to evaluate the merits of the improvement. This cost does not reflect the increases in the benefit brought about by an increase in the accidents reduced and interest saved over the service life of the improvement.

5. The cost of the improvement should reflect all those costs that will have to be paid out. This includes interest on money spent derived by the use of a Capital Recovery Factor (CRF) based upon the current interest rate and the service life of the project, and the maintenance costs paid in addition to normal maintenance. The capital recovery factors to be used for this analysis are as follows:

Project Life	CRF
10	0.1490
15	0.1168
20	0.1019
25	0.0937

In addition, those costs returned to the motorist should be considered. For instance, if the equipment at a particular site was removed and replaced with new, the old equipment's salvage value should be considered and reduced from the initial cost of the improvement. Therefore, the initial cost of the improvement multiplied by the capital recovery factor plus the maintenance cost minus the salvage value multiplied by the sinking fund factor gives us an Equivalent Uniform Annual Cost (EUAC).

$$EUAC = \text{initial cost} \times CRF + \text{maintenance cost} - (\text{salvage value} \times SFF)$$

6. The first year's benefit, as stated, is an inadequate measure of benefit. This recognizes only the benefit expected to be derived during the first year. In reality we would expect an annual benefit derived by the cumulative total of accidents being reduced during the entire service life of the improvement. This also takes into account those accidents reduced that would have occurred as a result of increased traffic volumes. The benefit is actually a measure of cost that would have been paid out by the motorist in accident occurrence, but has been foregone by the improvement. This same cost, if it has been paid, would also collect interest. Since the cost has been eliminated, the interest paid on this cost is also eliminated and therefore able to be invested productively elsewhere. Thus taking into consideration the First Year's Benefit, the interest saved over the entire service life of the improvement, the Capital Recovery Factor and normal growth in traffic volumes, we calculate the Equivalent Uniform Annual Benefit (EUAB).

   The EUAB is calculated as follows:

$$EUAB = (CRF)\,\frac{FYB}{1+i}\;\frac{\left(\frac{1+j}{1+i}\right)^{N}-1}{\left(\frac{1+j}{1+i}\right)-1}$$

Where:	EUAB	=	Equivalent Uniform Annual Benefit
	CRF	=	Capital Recovery Factor
	FYB	=	First Years Benefit
	i	=	Average Annual Interest Rate
	j	=	Annual Growth
	N	=	Service Life

7. The Benefit/Cost ratio is a rather simple concept. The EUAB is divided by the EUAC, the resulting number indicates how much return (benefit) can be expected for each dollar invested (cost). If the number exceeds one, the project can be considered cost effective. On a very simplistic basis, the higher the B/C ratio, the better the investment. A sample spread sheet is attached for the users benefit.

MANNER OF COLLISION	PROJECTED ACCIDENT EXPERIENCE	ACCIDENT REDUCTION FACTOR	AVERAGE ACCIDENT COST	FORECASTED REDUCTION	FIRST YEARS BENEFIT
ANGLE	2.3	0.5	10200	1.2	11730
LEFT TURN	11.6	1	10700	11.6	124120
			TOTAL	12.8	135850

	SERVICE LIFE	CRF	INITIAL COST	EUAC
JNDABOUT	10	0.149	300000	44700

j=	(ANN % INC IN TRAFFIC)	0.03	
i=	(INTEREST RATE)	0.08	EXP= 8.15
N=	(SERVICE LIFE)	10	

FORECASTED REDUCTION	**12.8**	**EUAC**	**44700**
AVERAGE ACCIDENT COST	**10653.13**	**EUAB**	**152749**
FIRST YEARS BENEFIT	**135850**		
		BENEFIT COST	**3.42**

RST-ROSCHE ENGINEERS, INC.
SCOTT ADAM ROAD, SUITE 103
COCKEYSVILLE, MD 21030
TEL: 410-683-1683

MARYLAND STATE HIGHWAY ADMINISTRATION

SAFETY IMPROVEMENT WORKSHEET

MD 170/ MD 174 ANNE ARUNDEL COUNTY

MANNER OF COLLISION	1990 ACC #	RATE	1991 ACC #	RATE	1992 ACC #	RATE	RATE	PROJ.
ANGLE	3	0.22	2	0.14	1	0.07	0.14	2.3
REAR END								
FIXED OBJECT								
OPP. DIRECTION								
SIDESWIPE								
LEFT TURN	13	0.97	8	0.55	9	0.62	0.71	11.6
PEDESTRIAN								
PARKED VEHICLE								
OTH COLLISION								
NIGHTTIME ACC.								
WET SURF ACC								
	ADT	36700	ADT	40000	ADT	39600		
	AVM	13395500	AVM	14600000	AVM	14493600	42489100	
				TRAFFIC FORECAST 1994				16359233

IMPROVEMENT: ROUNDABOUT

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