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THE COBA 2021 USER MANUAL

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PART 6

JUNCTIONS IN COBA

Contents

Chapter

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1. WHEN TO MODEL JUNCTIONS

1.1   One of the benefits associated with the construction of a new road scheme can be the relief of the delay which vehicles experience at congested road junctions in the existing road network. The amount of delay which vehicles suffer when queuing at a junction is dependent on the capacity of that junction and the traffic flow level. On the existing network congestion delay occurs when junctions are operating close to or above their capacity for long periods. There will, of course, also be delay associated with junctions on the new road scheme.

1.2   Junctions are represented in the COBA network description by nodes joining links together. If any of these links represent roads on which the rural speed/flow relationships apply then the nodes should be 'classified' and modelled as junctions. This is because the rural speed/flow relationships in COBA take no account of the delays to traffic at junctions, and it follows that junctions on rural roads should be modelled explicitly. It is generally only necessary to model the junctions where there is a significant flow change but there are no fixed rules relating to the flow change that necessitates a junction being modelled. The user will need to make a judgement regarding the significance of each junction to the assessment of the scheme being studied.

1.3   For **urban** and **suburban roads** the speed prediction formulae within COBA **do** take account of delays at junctions. In general, therefore, most junctions in urban areas do not need to be modelled explicitly. The exception is the junction which is heavily congested (either now or at some time during the life of the scheme) to such an extent that the journey times through the network resulting from the application of the standard urban speed/flow relationships are incompatible with actual timed runs. This junction should therefore be modelled explicitly in conjunction with speed/flow information from timed runs.

1.4   COBA models all junctions in isolation, with all arrivals assumed to be random, no allowance is made for any junction interaction. Therefore, great care must be taken when modelling junctions in the same area of the network where the capacity of one junction controls the flow at another, either now or at some time in the future. If this occurs it may only be necessary to model the controlling junction. Double counting of delay must be avoided.

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2. JUNCTION CHOICE

2.1   The formulae in COBA for determining junction capacity and delays are based on work carried out by TRL. The COBA program makes an estimate of the delays to traffic flow due to the presence of junctions in the network and calculates associated user costs. The level of detail is sufficient to enable the program to compare the user costs of different junction types (for example, grade-separated *versus* at-grade solutions at a particular location, but it must not be used for detailed junction design. Junctions can be modelled in more detail using the programs PICADY, ARCADY and OSCADY which are available from the Transport Research Laboratory. COBA uses formulae compatible with those in these programs to calculate capacities and delays.

2.2   The programs ARCADY, PICADY, and OSCADY model capacities, queues and delays at roundabouts, major/minor and signalled junctions respectively. The TRANSYT program can be used to model signalised roundabouts. They incorporate the 'high definition' queuing theory (see Paragraph 7.3) and should be used for the detailed design of junctions.

2.3   It is suggested that the following approach should be used for junction choice, at an appropriate stage in scheme appraisal:

❏   design the junction alternatives using programs such as PICADY, ARCADY and OSCADY, to see which options are feasible from an operational viewpoint;

❏   test alternative junction types using COBA. This may be done using a reduced network taking one 'Do-Something' option as a base and the other options as alternatives ensuring that the fixed trip matrix assumption is not violated;

❏   identify the optimum junction type in economic terms: it is that which creates the maximum NPV when construction costs, time and accident costs are taken into account. Given the sensitivity of junction formulae and uncertainty of turning movements, small differences in NPV should not be given too much weight.

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3.   JUNCTION TYPES MODELLED

3.1   This section gives a brief description of each of the junction types modelled in COBA. The detailed formulae for calculating queuing delay and junction capacity are given in Part 6 Chapters 7 and 8 respectively and are applied on an arm specific basis.

Major/Minor Junctions

3.2   Major/Minor priority junctions are by far the most common form of junction control. They are most appropriately used where it is desirable to give priority to one route, usually that carrying the greater traffic volumes. The COBA formulae are based on the calibration of observed relationships between the geometric characteristics of junctions, saturation and turning flow levels and capacity. They apply primarily to three armed junctions but the results have been extended within the program to apply to four armed junctions.

3.3   The designs of major/minor junctions vary widely from one junction to another and it has been necessary to distinguish those features which have appreciable effect on capacity and those that have little or no effect. The parameters used in the COBA program are:

❍   the width of the major road (and kerbed central reserve if applicable);
❍   the lane width for non-priority streams;
❍   the visibility.

3.4   The delay to through traffic on the major route of a major/minor junction caused by right turning traffic from the major road blocking traffic behind it is not considered significant and is therefore not modelled by COBA.

Roundabouts

3.5   The capacity is predicted entry by entry (up to six) and is dependent on the geometry of the roundabout and demand flows on each arm. The geometric parameters fall into a distinct hierarchy with the entry width and length of flare having by far the most important effect on capacity and with the inscribed circle diameter having a lesser but still important effect. The angle and radius of entry are also relevant. The entry capacity is defined as the inflow from an entry when the demand flow is sufficiently high as to cause steady queuing on the approach. The inflow is dependent on the circulation flow across the entry, to which it has to give way. The entry capacity therefore decreases as the circulating flow increases and vice versa, consequently the capacity changes by flow group.

Signalised Roundabouts

3.6   Signalised roundabouts are becoming increasingly common, especially at grade separated roundabouts at motorway exits. A signalised roundabout is a system of one-way traffic circulating round a central island with at least one of the entries controlled by traffic signals. However COBA can only model junctions where all entries are controlled by signals. Cases where only one arm is signalised are probably best treated as conventional roundabouts. Intermediate cases with two or more entry arms could be considered as fully signalised as they would be likely to become so at some time during the assessment period. At some four arm at-grade roundabouts through traffic on the main road passes straight through the centre of the junction and minor road and turning traffic circulates. These `hamburger' roundabouts have a different co-ordination strategy from circulating roundabouts, they are designed for minimum delay to the main road traffic passing straight through the junction, and cannot be modelled using COBA.

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3.7   The method of assessing delays at signalised roundabouts in COBA was developed from the TRANSYT program, see TRRL Research Report RR 274 - the use of TRANSYT at Signalised Roundabouts. It is assumed that the system operates on a common cycle time irrespective of how it is controlled on-street. By an iterative procedure COBA determines a suitable cycle time and green times for each approach for each flow group for each year of the appraisal. Then using queuing delay formulae predicts mean delay per vehicle, including geometric delay, for each entry.

3.8   The circulating link system necessary to model signalised roundabouts is set up automatically by COBA. The circulating carriageway typically has between two and four lanes. Each link carries traffic originating from a particular entry arm and hence the shared stop-line models the arrival of different platoons of traffic at the stop-line at different times in the cycle. All nodes are assumed to be two-stage and there is no double-cycling or double-greening. Intergreen periods are fixed at five seconds although at some sites this may need to be increased for operational reasons. Excess queue weighting on the circulating links applies only if the distance between stop-lines is less than 100m. The COBA model uses a relatively low weighting but takes account of excess queuing once the link is predicted to be half full on average, rather than the TRANSYT recommended two-third full.

3.9   Modelling signalised roundabouts in COBA can significantly increase the program run time and the user may prefer to have built separate COBA models for individual junctions until the data is known to be correct.

Traffic Signals

3.10   Calculations of capacity and delay are performed to reflect OSCADY theory and to use algorithms as published in TRRL RR105.

3.11   Designs of signalised junctions vary considerably and COBA can model junctions with three to six arms and a maximum of nine stages. The user inputs the permissible movements from a lane and staging details. The only geometric details affecting the capacity of signals is the width of the stop line and approach gradient. The program assumes that traffic will act in a reasonable way and attempt to equalise congestion between lanes, it then performs an iterative process to reach the optimal signal settings for each flow group in each year of the evaluation.

3.12   Research has shown that traffic signals which are computer controlled to TRL's MOVA strategy experience, on average, 13% less delay compared to normal vehicle actuated signals. The user can indicate signals operating under MOVA control and the program will factor the calculated delays accordingly.

Gates and Other Regular Obstructions

3.13   It is possible to model gates and other regular obstructions to traffic such as level crossings and swing bridges. If the cycle time is reasonably constant pelican crossings may also be modelled. The essential feature of these types of obstruction is that traffic control is of a stop/start nature making them similar to traffic signals. For this reason, COBA makes use of the data relating to the number and average duration of closures to determine a mean 'cycle time' and an 'effective green time per cycle' and then models gates as signals.

Merges at Grade Separated Junctions

3.14   When the total traffic demand on the main line and slip road at a grade separated junction exceed the capacity of the downstream link flow breakdown can occur with resultant delays to both the main line and merging traffic queues. The delays to the total traffic stream can be modelled in COBA.

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ACDY only Node

3.15   ACDY nodes are used to model, for example, user specified geometric delay at a roundabout or points in the network such as sharp bends where the speed/flow relationships do not apply. Although not necessarily a junction ACDY nodes are included in the junction data input records. The specified delay is applied to all vehicles and is constant over all flow groups.

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4. THE CONCEPT OF 'MAXIMUM DELAY'

4.1 The concept of maximum delay has an important bearing on junction evaluation in COBA. The form of the COBA junction delay formulae implies that, as the capacity of the junction is reached, delays increase very rapidly. COBA includes, as default, a maximum delay at junctions of 300 seconds for peak flow group types (note that although up to six flow groups can be modelled in COBA there are only four flow group types - see Part 7, Chapter 7). If journey time evidence warrants a local adjustment for any individual junction the user may change the value for the peak group upwards or downwards (maximum 900 seconds). The maximum delay is attributed to all vehicles but on an arm by arm or stream by stream basis. Maximum delays for the non-peak flow groups are calculated as a proportion of the peak flow group maximum delay; see Table 4/1.

Flow Group Type	Proportion of Maximum Delay	Default Maximum Delay
Type 1 (Off-peak)	0.4	120 secs
Type 2 (Adjacent to Peak)	0.6	180 secs
Type 3 (Peak)	1.0	300 secs

Table 4/1 - Maximum Delays by Flow Group

4.2 Without a maximum delay cutoff, the formulae would predict much higher delays in the future at heavily congested junctions than have been generally observed at existing congested junctions. Such high delays are considered unrealistic bearing in mind that it applies to all traffic in the flow group. The 300 second maximum delay for the peak flow group presumes that some vehicles will queue for considerably longer than this, some for a shorter period, with the average for all vehicles over the entire peak period being 300 seconds. In practice some drivers will act to prevent such long delays occurring by travelling by a different route or travelling at a different time. The maximum delay substitutes for the detailed modelling and evaluation of such behaviour for which COBA is not designed.

4.3 Where junctions are explicitly modelled and COBA junction delay benefits are an important element, the realism of the magnitude of the junction delay benefits should be examined by considering the following:

❏ 'Do Minimum' improvements (see Part 1 Chapter 2) or, where 'Do-Something' delays are large, 'Do-Something' junction optimisation. Small changes in the coding of junction layouts can sometimes yield significant changes in junction delay costs.

❏ Comparison of COBA and measured journey times (see Part 5 Chapter 10), for example, where junctions interact. In practice, it is more common for local maximum average delays to be less than 300 seconds rather than more.

❏ Explicit modelling of critical junctions outside COBA. In exceptional cases, the COBA user may wish to consider whether in-depth analysis of a critical junction is necessary.

A particular problem with calibrating maximum delays using timed runs is that it is impossible to calibrate the future, for example, a junction may be coded at 300 seconds maximum delay on the basis of present journey time evidence, but may give rise to longer delays in the future with traffic growth. This may be tested using more sophisticated junction modelling techniques as found in congested assignment packages explicitly models local diversionary reassignment; advice from the Overseeing Organisation should be sought in such circumstances.

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5.   GEOMETRIC DELAY

5.1   Geometric delay is considered as the delay suffered *in the absence of queues*, as a result of the need for vehicles to slow down, negotiate a junction and accelerate back to normal speed. (It is also known as ACDY delay.)

Geometric Delay at Major/Minor Junctions

5.2   Vehicles travelling straight through the junction on the major arms do not suffer any significant geometric delay, but all other vehicle movements are subject to delay dependent upon the turning movements. It has been shown that geometric delays are as given in Table 5/1.

Vehicle Category	Delay per Movement (seconds/vehicle)					
	Major left	Major ahead	Major right	Minor left	Minor ahead	Minor right
Cars, LGV	5.7	0.0	6.5	7.8	12.2	10.6
OGV, PSV	7.8	0.0	8.6	9.9	14.3	12.7

Table 5/1: Geometric Delays at Major/Minor Junctions

5.3   Speed and visibility are also factors which increase the delay to vehicles.

   ❍   Geometric delays vary with entry and exit link speeds. They are significantly higher at intersections with link speeds greater than 64 kph (generally rural roads without speed limits) for which COBA adds 2 seconds delay per vehicle, than those with link speed less than 64 kph (generally suburban roads with speed limits).

   ❍   Delays at major/minor junctions which do not meet the visibility requirements Standard TD42 (DMRB 6.2.6) are significantly higher than those at junctions which do. For this substandard visibility COBA adds 1.4 seconds per vehicle.

Geometric Delay at Roundabouts - Conventional and Signalised

5.4   COBA calculates geometric delay for light vehicles at all types of roundabout, including grade separated, using the formula:

$$\text{Delay per light vehicle} = \frac{dbc}{js} + 0.23V - 5.62 - 0.12D + 0.000367V.D \text{ seconds}$$

Where js =   estimated vehicle speed on roundabout of $0.96\sqrt{D} + 2.03$ metres/sec
         D =   inscribed circle diameter in metres
         dbc =   distance travelled on roundabout ($= \alpha \pi (D-7)$ metres)
         α =   proportion of roundabout travelled. It is assumed that arms are equally spaced around roundabout. (For a right turn at a 4 arm roundabout $\alpha = 0.75$.)
         V =   the average of the approach speed on entry link and the departure speed on exit link, in kph, and will vary by flow group. Entry and Exit speeds are calculated as the weighted mean of light and heavy vehicle speeds.

5.5   Geometric delay per vehicle for heavy vehicles is calculated at being 15% greater than for light vehicles.

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5.6   The user may choose **not** to have geometric delays calculated by COBA; optionally he may then directly enter geometric delay data using an ACDY only node (see paragraph 5.8)

Geometric Delay at Traffic Signals

5.7   The program assumes no geometric delay but if considered significant it must be input by the user An estimate of geometric delay can be made using formulae detailed in TRRL Research report 105 or by use of a suitable program such as OSCADY.

ACDY Only Node

5.8   ACDY only nodes are used to take account of geometric delay; for example when user specified geometric delay is required at a roundabout, at other points in the network (such as bends) where no *queuing delay* occurs but *geometric delay does* due to the need for drivers to reduce speed. ACDY delay is input as average delay per vehicle and is not disaggregated into flow groups or vehicle types.

Merges at Grade Separated Junctions

5.9   Even under light traffic flow conditions there will be a slight geometric delay at all merges at grade separated junctions but these are not modelled in COBA. Under heavy flow conditions the formula used to calculate the total delay subsumes the geometric delay.

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6.   TURNING FLOWS AT JUNCTIONS

6.1   For the calculation of link transit costs COBA makes the simplifying assumption that the flows on two way links are the same in each direction. However at junctions a similar assumption cannot be made as unequal flows can have a significant impact on capacity and hence delays. For example, the entry capacity of a particular stream of traffic at a roundabout is dependent on the circulating flow which will be different under balanced or tidal conditions. Hence the delay to the stream of traffic will also vary and if the volume of traffic being delayed is large, then the calculation of delay can be significantly in error from a balanced flow assumption when imbalanced conditions actually occur. In addition to different morning and evening peak period tidality COBA can also reflect the daily imbalance of flows that can occur on approaches to some junctions, this is modelled in COBA by the use of an 'inflow proportion'.

6.2   Although flows on links can be defined by vehicle mix group, junction turning movement details apply equally to all vehicle categories.

6.3   Figure 6/1 shows an example of a four-arm junction at Node 5 in a network which connects links 16, 2, 33 and 758
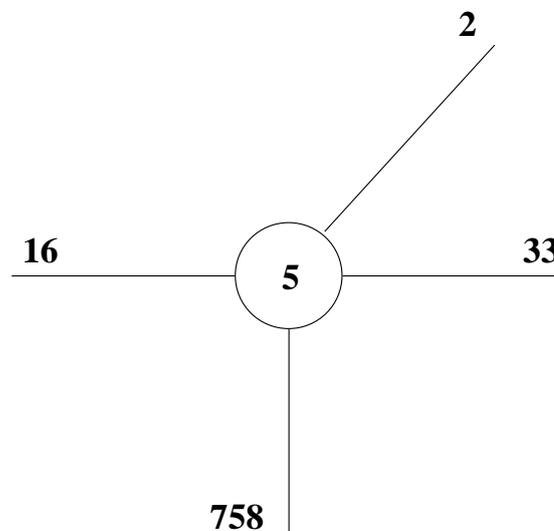

Figure 6/1: Example of a Four-Arm Junction

6.4   It is conventional (and implicit in COBA data input) to list or tabulate the links in clockwise sequence both along the rows and down the columns of the matrix. One-way links, both entry and exit, need to be entered.

6.5   Turning flows may be input either as flow matrices or as proportion matrices. Earlier versions of COBA required that these input turning matrices were symmetrical; this is no longer the case. The turning flow matrix below may be entered as it stands, or converted as described into the turning proportion matrix that follows it, and then entered in that form. KEY 082 is the appropriate record type.

6.6   Table 6/1 is an example of a turning flow matrix. It will rarely be symmetrical, either in reality or as output from a traffic model.

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From/towards	16	2	33	758	Total Inflow
16	0	25	830	27	882
2	24	0	21	110	155
33	751	20	0	19	790
758	26	110	20	0	156
Total outflow	801	155	871	156	1983

Table 6/1: Example of a Turning Flows Matrix

6.7   The corresponding turning proportion matrix, given in Table 6/2, expresses each movement as a thousandths proportion of the inflow, that is, the number of vehicles making a turn per thousand entering the junction on the arm. This must be expressed as a whole number for submission to COBA (the presence of a decimal point will cause a program error).

From/towards	16	2	33	758	Total
16	0	028	941	031	1000
2	155	0	135	710	1000
33	950	025	0	024	1000
758	167	705	128	0	1000

Table 6/2: Example of a Turning Proportions Matrix

6.8   When entering a turning proportions matrix the rows should sum to 1000; if the values entered total between 990 and 1010 the program will scale all values accordingly. There is no significance to the column totals. If the flow on any of the links changes, either in a reassignment year or for a scheme, then a revised turning matrix must be entered. U-turns can be modelled at roundabouts, but not at signals or major/minor junctions.

6.9   If a turning flow matrix is entered then any imbalance of flows that occurs on any, or all, of the entry arms is automatically defined; the total turning flow on each arm does not have to equal the total flow as entered on the link flow data. When a turning proportion matrix is entered then COBA automatically assumes that flows on all two-way links are balanced. This assumption can be overwritten by defining the inflow proportion of each particular entry link. The inflow proportion is defined as the percentage of the two-way link flow that approaches the junction over a twenty-four hour period. Whichever method is used to define turning movements COBA will undertake a balancing procedure to ensure that total junction inflow and outflow are equal.

6.10   Different morning and evening peak period tidality can be modelled in COBA by means of tidality factors. The tidality factor is defined as the percentage of the total two-way flow approaching the junction in the peak. The use of tidality factors overwrites any inflow proportion that has been entered for the link, consequently for links with unbalanced flows it is possible that both 'a.m.' and 'p.m.' tidality factors may be greater/less than 50%.

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6.11   Whenever tidality is modelled COBA will undertake a balancing procedure to ensure that the total junction inflow equals the outflow in each peak period modelled and thus the factors used in the calculation of delays may vary slightly from those input.

6.12   In summary the COBA program undertakes the following operations to produce the required turning flow matrices necessary to carry out junction delay calculations, the process is also shown diagrammatically in Figure 6/2:

      i)   if a turning flow matrix is input; produce a turning proportions matrix and junction inflow proportions matrix; **or,**

      ii)   if a turning proportions matrix and an inflow proportions matrix are input, carry out minor balancing operations;

      iii)   produce off-peak flow matrices for each year by the combination of link flows, input proportions matrix and turning proportions matrix;

      iv)   if tidality is modelled read a.m. and p.m. peak tidality factors and balance inflows and outflows accordingly to produce peak period turning proportions matrix; **and,**

      v)   produce peak period turning flow matrices for each year by the combination of link flows, peak inflow proportions matrices and turning proportions matrix.
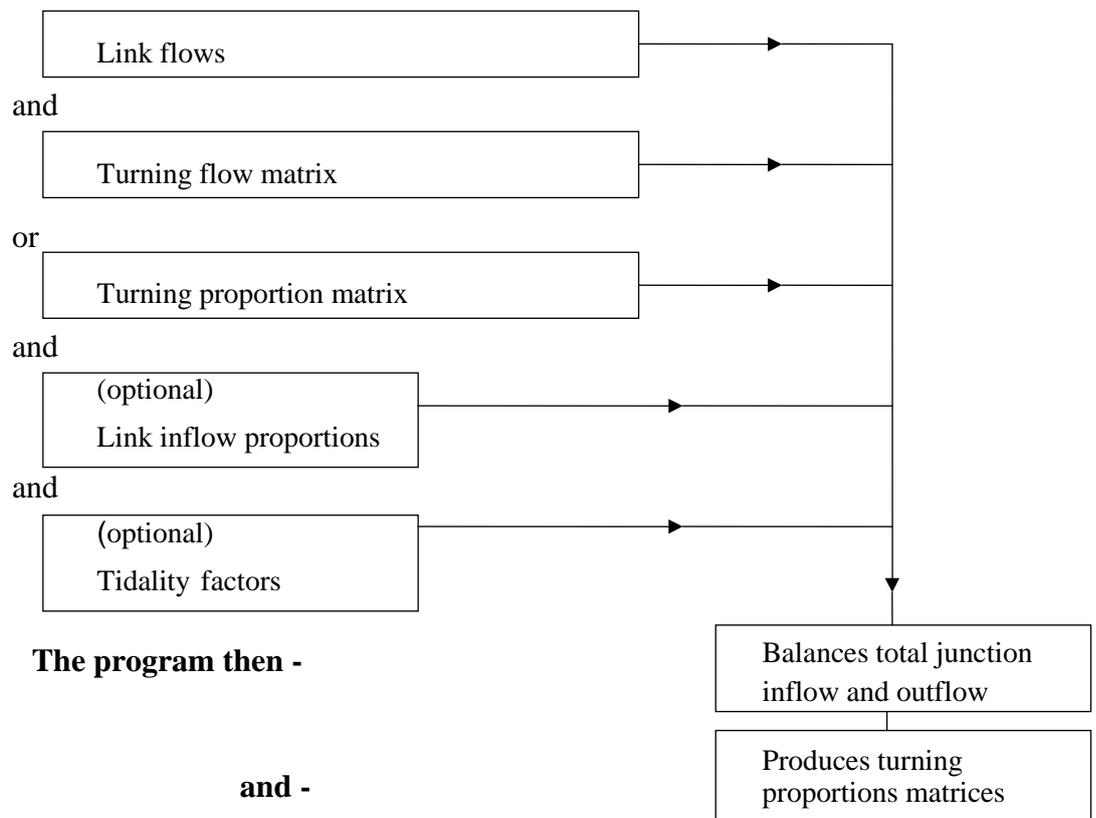
User enters data for -

Figure 6/2: Production of Turning Proportion Matrices

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6.13   The user should be aware of the following characteristics of COBA:

     ❍   any imbalance of flows on connecting links is applied to all flow groups;

     ❍   tidality occurs only in the peak flow groups (flow group types 3 and 4);

     ❍   flow imbalance and tidality does not affect the calculation of link transit costs or the junction accident model;

     ❍   when tidality is modelled COBA automatically redefines the flow group structure to provide two peak flow groups each containing 250 hours with the same volume of traffic and vehicle proportions in each. If the user overwrites the default flow group structure the peak flow group must also be manually divided in a similar way if tidality is being modelled;

     ❍   COBA uses the average daily turning movement defined by the user for all vehicle types combined, even though link flows for separate vehicle mix groups may be input;

     ❍   the program will undertake some basic checks but it is up to the user to ensure that the data has been input correctly. By permitting an imbalance of flows on connecting links it is not possible to define rigorous consistency checks.

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7.   QUEUING DELAY

7.1   Queuing delay can be considered as that delay resulting from the presence of other vehicles. Traffic at road junctions can usually be divided into a number of separately identifiable streams each with a capacity and a demand flow. Whether or not queuing occurs in a given stream depends on the ratio of these two items, known as the 'Traffic Intensity'. The capacity has been defined in Part 6 Chapter 3 for the different junction types. The demand flow is inherently variable, beside random fluctuations in the arrivals at a junction the average level of flow varies during the day. At peak times the traffic demand in a stream may approach or even exceed the capacity available to it, whereas in off peak periods there is usually a substantial margin of spare capacity. This varying demand is represented in the COBA flow group structure (see Part 4, Chapter 7). For COBA purposes it is necessary to employ flow/delay relationships which reflect these varying flow group conditions.

7.2   Traditionally flow/delay relationships are derived from a steady state theory which provides good predictions when there is spare capacity. (These conditions usually apply in COBA off peak and adjacent to peak flow groups - COBA flow group numbers 1 and 2). Steady state queuing theory, however, predicts infinite queues and delays when the demand flow reaches the capacity available to it and therefore cannot be applied to determine delay at high traffic intensities which can occur in COBA peak flow groups. Instead, time dependent queuing theory is used with a maximum delay cut off (see Part 6 Chapter 4). The exception is for grade separated merges where a simple linear flow/delay relationship was found to represent observed delays.

7.3   In its most refined state (high definition) this theory makes possible the modelling of the growth and decay of queues of vehicles in very small time intervals. For COBA purposes, it is sufficient to employ flow/delay relationships which give only average delay associated with average flows over the peak flow group hours (a low definition approach). Assumptions have been made about the shape of the traffic demand profile, the approximate length of peaks and as a result the average intensity and capacity. This is illustrated in Figure 7/1. In junction delay formulae the length of each peak is called the 'block time' (T). For COBA purposes the length of each peak has been taken as 1 hour.

7.4   The flow/delay relationships used in the program are based on a coordinate transformation technique which smoothes the steady state relationship for vehicle delays into the over capacity 'deterministic' results obtained by integrating demand minus capacity. The relationships depend on the flows and capacities during the peak and adjacent off peak periods, and on the peak duration. Figure 7/2 illustrates schematically the form of the curve. The value of the maximum delay cut off includes geometric delay where appropriate.

7.5   The peak period flow conditions (flow group type 3) generally correspond to the traditional morning and evening commuting peaks (the COBA default peak hour is flow group 4 with a duration of 500 hours). However in recreational areas the peaks are more likely to occur in the summer months at weekends. Where reliable local data indicates for a particular scheme that the peak flow conditions occur for a larger/smaller proportion of the year or that the length of each peak period is greater than the standard COBA values, it is possible to redefine the flow group structure (see Part 4, paragraph 7.3) and the block time, either separately or simultaneously. The user should note that the relationship between the number of hours of the year experiencing peak flow conditions and the length of individual peaks implies that a particular number of peaks occur. Increasing the number of hours in the peak flow group without changing the block time would result in more peaks. Increasing the number of peak hours would not necessarily result in an increase in the modelled delays because increasing the number of peak hours generally reduces the average traffic intensity of the hours modelled. The relationship of the peak period and the maximum delay cut off (see Part 6 Chapter 4) in the assessment of total delay also needs to be considered.
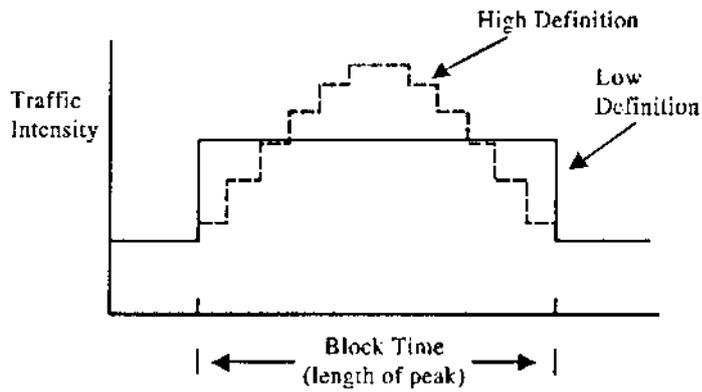
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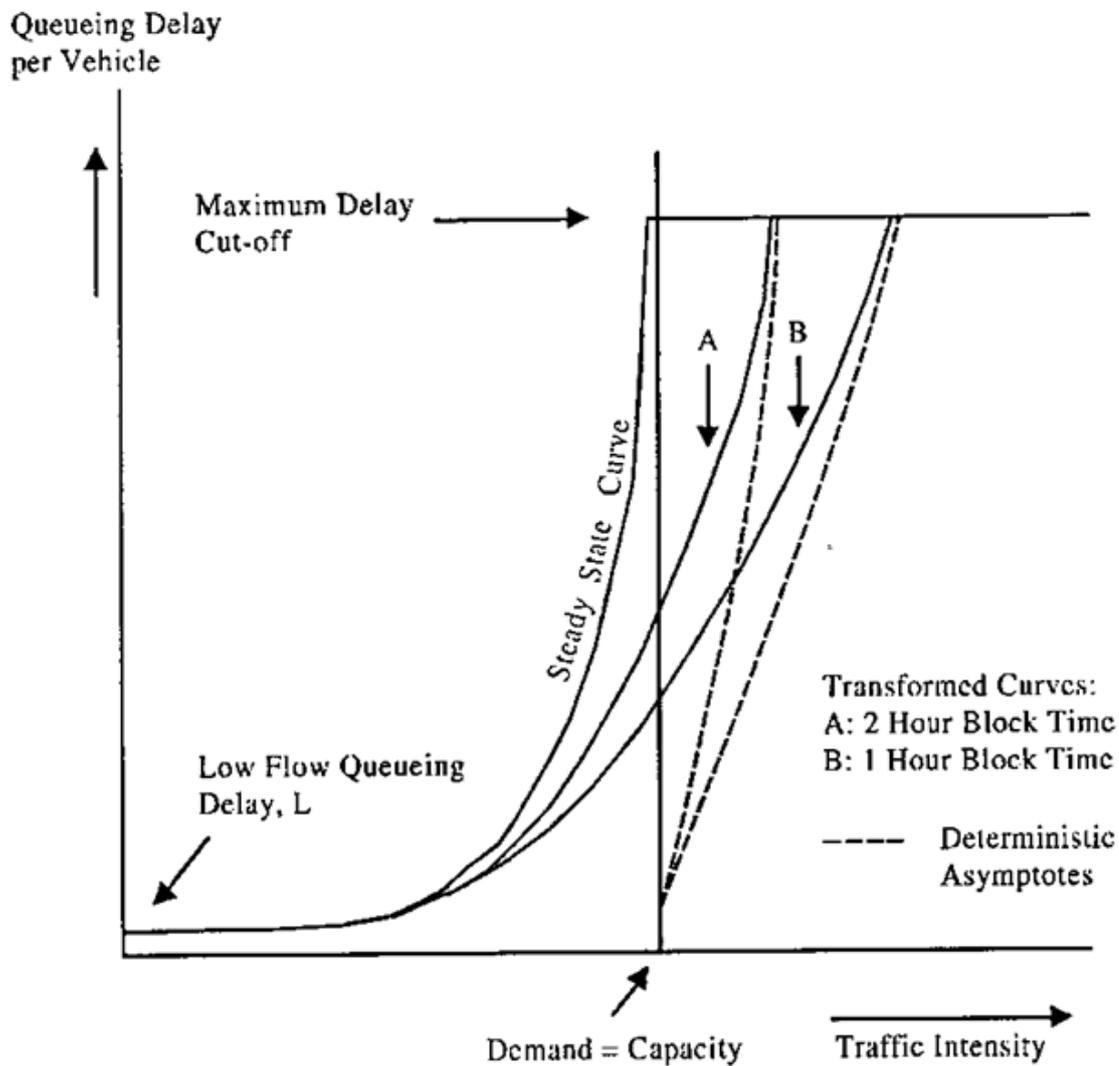**Figure 7/1: High and Low Definition Peak Models**

Figure 7/2: Queuing Delay Curves

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Queuing Delay Formulae (except Grade Separated Merges)

7.6   The formulae used in COBA to calculate the delay for either the non peak flow groups (steady state queuing) or the peak period flow groups (time dependent queuing) are given below.

7.7   For steady state queuing: Delay $= \dfrac{C\,\rho_o}{\mu_o(1-\rho_o)} + L$

where ρ_o is the traffic intensity for non-peak

C is the randomness factor

μ_o is the capacity for the non-peak

L is the low flow queuing delay

For roundabouts and major/minor junctions $L = \dfrac{1}{\mu_o}$ For signals $L = \dfrac{CT(1-\lambda)^2}{2(1-\lambda\rho_o)}$

where   CT is the cycle time, and λ is $\dfrac{\text{green time}}{\text{cycle time}}$

7.8   For time dependent queuing: Delay, $D = \dfrac{\sqrt{F^2+G}}{2} - \dfrac{F}{2} + e + L$

where $F = \dfrac{1}{\mu_o - q_o}\left[\tfrac{1}{2}T(\mu-q)\left(1-\dfrac{h}{q}\right) + 2C\left\{1-h\left(\dfrac{1}{q}+\dfrac{1}{\mu}\right)\right\}\right] + e$

$G = \dfrac{2T}{\mu_o - q_o}\left[\dfrac{2Cq}{\mu} - (\mu-q)e\right]\left(1-\dfrac{h}{q}\right)$

$h = \mu - \mu_o + q_o$

$e = \dfrac{2C\,q_o}{\mu_o(\mu_o - q_o)}$

L is the low flow queuing delay
q is the demand flow at peak periods
q_o is the demand flow for adjacent to peak
μ is the capacity at peak periods
μ_o is the capacity for adjacent to peak
T is the Block Time representing length of peak
C is the Randomness factor, a constant depending on arrival and service patterns

7.9   At give way junctions (major/minor and roundabouts) it is reasonable to assume that both vehicle arrivals to the rear of the queue and departures across the give way line are approximately random and therefore C can be taken as 1.0. Also, as before (see Part 6 paragraph 7.7) L, the low flow queuing delay $= 1/\mu$.

7.10   Signalised junctions differ in that the departures across the stop line are clearly non random. Because of cyclic operation traffic from each approach is released in dense platoons with substantial gaps between. Within each platoon vehicles are spaced almost regularly. For this case C is therefore not equal to unity and the most appropriate value for C has been taken as 0.60. L, the low flow queuing delay is the same as that for the steady state (see Part 6 paragraph 7.7).

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Queuing Delay Formula for Grade Separated Merges

7.11   Research into merge delays showed that the measurement of actual average delays from on-site observations was exceedingly difficult. Day to day variations in delay were apparent under apparently similar flow conditions. Delays varied considerably with time even within a single peak period. The research was unable to differentiate between delays to the main line and merging streams. Consequently COBA includes a relationship which applies delays (over and above speed/flow effects) to all vehicles passing through the merge area. This is a simple straight line function of the form:

$$\text{Delay} = 227\,(\text{Capacity Ratio} - 0.75\,)\text{ seconds per vehicle}$$

where the capacity ratio is defined as the total upstream demand divided by the capacity of the downstream link.

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8.   FORMULAE FOR JUNCTION CAPACITY

8.1   Junction capacity is calculated for each traffic stream entering the junction and is therefore an `entry capacity'. The capacity formulae for the various junction types are given below. Details of how to determine the various geometric parameters are given in Chapter 9.

Major/Minor Junction Capacity

8.2   The COBA methodology is based on that developed for PICADY (see TRRL LR 941 - PICADY: a computer program to model capacities, queues and delays at major/minor junctions). In general for 3 and 4 arm major/minor junctions, the entry capacity of a stream has the following form:

Capacity   Q   =   X (C - Y x Flow Factor)

         Where X   =   is a term representing geometric characteristics of the layout (see TRRL LR 941 for further details),

                     C   is the saturation flow, and

                              =   745 for the left turn out of and the right turn into the minor road (pcu/hr) or,

                              =   627 + 14 times central reservation width (m) for the right turn into the major road (pcu/hr)

                     Y   =   1 - 0.0345 x major road width (m). It determines the overall effect of the flow factor

         Flow Factor   comprises the flows that inhibit the movement in question weighted according to the effect of each. The flows are in pcu/hr.

8.3   Research has shown that even with extremely high traffic flows on the major road some traffic will always force a way into the traffic stream. Therefore there is a minimum capacity value of 60 pcu/hour for the minor road, this is divided equally between the number of approach lanes.

Roundabout Capacity

8.4   The COBA methodology is based on that developed for ARCADY (see TRRL LR 940 - ARCADY: a computer program to model capacities, queues and delays at roundabouts). The capacity (Q_e) of each arm of the roundabout in pcu/h is given by the equation:

$$Q_e = k (F - f_c Q_c)$$

Where the terms are defined as:

$$k = 1 - 0.00347 (\phi - 30) - 0.978 \left(\left(\frac{1}{r} \right) - 0.05 \right)$$

$$F = 303 \, x_2$$

Q_c = the circulating flow across the entry in pcu/h

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$$f_c = 0.210 \, t_D \, (1 + 0.2 \, x_2)$$

$$t_D = 1 + 0.5 / (1 + \exp(\frac{(D - 60)}{10}))$$

$$x_2 = v + (e - v) / (1 + 2S)$$

$$S = 1.6 \, (e - v) / l'$$

For grade separated roundabouts (Junction Type code 4 Key 077), the entry capacity, $Q_{e'}$, of a slip road into the roundabout from a grade separated route is calculated from a modified form of the above equation. Where capacity, in pcus, is given by:

$$Q_{e'} = k \, (1.1 \, F - 1.4 \, f_c \, Q_c)$$

8.5   This relationship is not the most recently derived one for grade separated roundabouts (as reported in TRRL Research Report 142) which has been incorporated into ARCADY. This is because the latest formula may give unrealistic results at the low circulating flows which may be found within COBA during certain periods of the day. ARCADY should of course be used for junction design (see Part 6 paragraph 2.3) when peak hour flows will be used, and the user can decide on the realism of the results. Although the formula used in COBA is not the most recent it remains perfectly good, for assessment purposes. RR142 states that "the practical implications of the relationship (that is, the latest formula) as compared to existing predictive methods are not great".

8.6   Geometric items e, v, l', D, r, are in metres and ϕ is in degrees. They are defined below and details of how they are measured are given in Part 6 Chapter 9. The following list gives the equivalent COBA data record headers (KEY 077).

v	= AWID	-	Approach road half width
e	= EWID	-	Entry width measured along the normal to the outer kerb line
r	= ERAD	-	Entry radius measured as minimum radius of curvature of the nearside kerb at entry
ϕ	= FI	-	Entry angle (geometric proxy for the conflict angle between entering and circulating streams)
l'	= FLEN	-	Average effective length over which the flare is developed
D	= DIAM	-	Inscribed circle diameter

Capacity of Traffic Signals

8.7   The COBA methodology is based on that developed for OSCADY (see TRRL RR 105 - OSCADY: a computer program to model capacities, queues and delays at isolated traffic signal junctions). Saturation flows in passenger car units per hour are calculated by movement within a lane. They are dependent upon the geometric parameters stop line width and gradient. Allowances for unopposed movements, opposed movements with hooking and opposed movements without hooking are made, using formulae detailed in TRRL Research report 67.

8.8   The cycle and effective green times are derived using a Webster and Cobbe type algorithm. Calculations are based on the inflows and saturation flows on each lane, and the user can input lost time per cycle, where generally lost time (L) is equal to the sum of each intergreen (IG) minus 1 second per intergreen that is:

$$L = \sum\sum (IG-1)$$

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8.9   Practical capacity is determined for each lane using the formula:

$$0.9\mu = S.\ g/c$$

where μ is the theoretical capacity
S is the saturation flow
g is the green time
c is the cycle time

Note: the cycle time and green times are calculated separately for each year and flow group, and cycle time increases up to a limit of 120 seconds as the flow increases.

Capacity of Merges at Grade Separated Junctions

8.10   Research has shown that the best estimate of the capacity of dual three lane motorways is:

$$2330/(1+0.015P)\ \text{vehicles per hour per lane for motorways.}$$

taking a value ten per cent less gives a figure of :

$$2100/(1+0.015P)\ \text{vehicles per hour per lane for all-purpose dual carriageways.}$$

Where P is the proportion of heavy vehicles expressed as a percentage. These capacities are the defaults held within the program used in the calculation of delays at merges. However, local conditions can affect the capacities and the user has the facility to input local values if reliable data is available.

Capacity of Gates and other Regular Obstructions

8.11   COBA makes use of the data relating to the number and average duration of closures to determine a mean 'cycle time' and 'an effective green time per cycle' and then models gates as signals.

8.12   Capacity and delay per vehicle are calculated on an arm specific basis. The cycle time is given by:

$$c = 3600/n\ \text{seconds}$$

where n is the number of closures per hour in each flow group.

The effective green time per cycle, g, is given by:

$$g = (c - t)\ \text{Seconds}$$

where t is the average duration per closure, in seconds.

The proportion of cycle time that is effectively green, λ, is given by:

$$\lambda = g/c$$

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8.13   COBA calculates the saturation flow, S, for each arm from the number of lanes and width per lane (w) entered by the user. This calculation is similar to that performed for traffic signals but at gates it is intended that all traffic should proceed straight ahead, so no correction for turning proportions is made. Also no correction is made for any gradient at the stop line. The formula default used to calculate saturation flow is:

$$S = 2080 + 100 \, (w\text{-}3.25) \, \text{pcu hr/lane},$$
but the user can specify the saturation flow in pcus in which case this formula will be superseded.

The capacity, λ, is given by:

$$\lambda = S. \, g \, / \, c$$

Where S, g and c are as defined above.

8.14   At gates, the form of the low flow delay is as for a signalised junction. However, at gates, the cycle time c is not flow group dependent unless the number and duration of closures is defined separately for each flow group.

Capacity of Signalised Roundabouts

8.15   The COBA model has been developed for fully signalised roundabouts with between three and six nodes, which operate on a common cycle time. It is assumed that all nodes can be modelled as two-stage junctions irrespective of how they are controlled on-street.

8.16   The capacity of a signalised link is determined using the formula:

$$\text{Capacity} = S. \, g \, / \, c$$

Where   S is the saturation flow of the link,
             g is its effective green time, and
             c is the cycle time of the junction.

8.17   Signal timings are selected such that entry arms are more saturated than the circulating carriageway which therefore has short queues, the capacity of each arm to the roundabout is determined entirely by the entry arm and not by the downstream links.

8.18   The saturation flow for an entry arm (excluding any flare) is calculated according to the formulae in TRL Research Report RR 67 from the relationship:

$$\text{LANES x (1940 - (42 x FLAG x GRAD) - 325) + WIDTH x 100 \ \ pcu/hour}$$

Where   LANES   is the number of lanes in the approach (excluding flare),
             FLAG   is 0 for downhill approaches and 1 for uphill approaches,
             GRAD   gradient of approach over last 60m if uphill (%), and
             WIDTH   is the stop line width (excluding flare) in metres.

Curvature is ignored and no account is taken of the proportion of turning traffic

8.19   In order to estimate the saturation flow of the flare on an entry arm it is necessary to calculate the number of vehicles that can be accommodated in the flare by assuming that each pcu occupies 5.75 metres. Then, providing the clearance time is less than the effective green time then the additional saturation flow is calculated by assuming a cycle time. The estimate is then reviewed at the next iteration of the optimisation procedure. If the approach has been widened over such a length that the flare will not be cleared during the effective green time then the lanes in the flare should be included in the approach width.

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8.20   The saturation flow for the circulating stop lines is calculated from the relationship given in paragraph 8.18 and reduced by 10% as it has been found that the capacity of circulatory arms are usually lower than that for entry arms. It is assumed that approaches to circulating stop lines are not flared and have zero gradient. In circumstances where these assumptions are not valid the user is able to input saturation flows directly.

8.21   The number of vehicles which can be accommodated on a circulatory link is calculated from:

$$\text{Number (pcu)} = \frac{\text{link length (m)}}{5.75} \times \frac{\text{saturation flow (pcu/hour)}}{1850}$$

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9.   GEOMETRIC PARAMETERS

9.1   The junction capacity formulae detailed in Chapter 8 require the user to input geometric features of each junction modelled. The determination of these is given below.

Major/Minor Junctions - Geometric Parameters

9.2   The measurement of the geometric features used in the geometric term X, given in Part 6 paragraph 8.2, is illustrated in the following figures, the COBA input record is KEY 078.

9.3   <u>Major road width (CWID).</u> The four parts of Figure 9/1 show the main components of major road width.
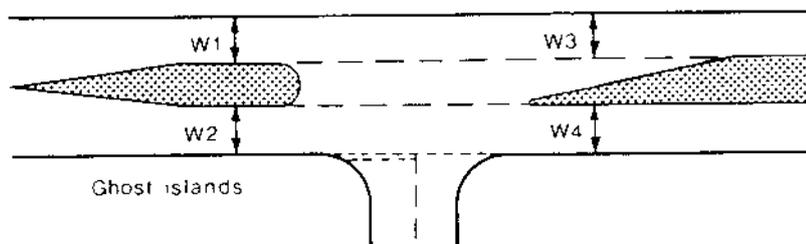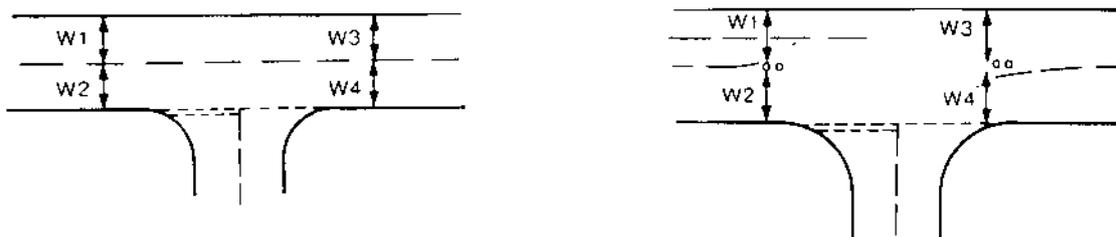
the total carriageway width: TWID = ½(W1 + W2 + W3 + W4)
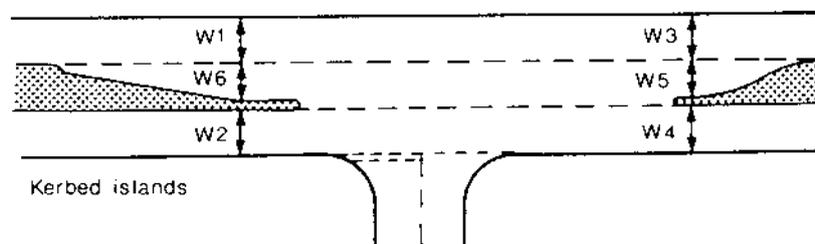
at dual carriageway sites with kerbed central reserve the width of the central reserve:

CWID = ½ (W5 + W6)

Figure 9/1: Major Road Width

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9.4   Lane width for non priority streams, w Where there are clear lane markings the width is measured directly. The average of measurements taken at 5 metre intervals over a distance of 20 metre upstream from the give way point is used. Any measurement exceeding 5 metres is reduced to 5 metres before the average is taken. Where lane markings are unclear (or absent), measurements are taken as illustrated in Fig 9/2 and the lane width calculated according to:

$$w = (a + b + c + d + e) / 5$$

9.5   Where the width of minor road is insufficient to accommodate separate lanes for left and right turning vehicles measurements a to e are made across the shared lane from the kerb line to the centre line of the minor road.
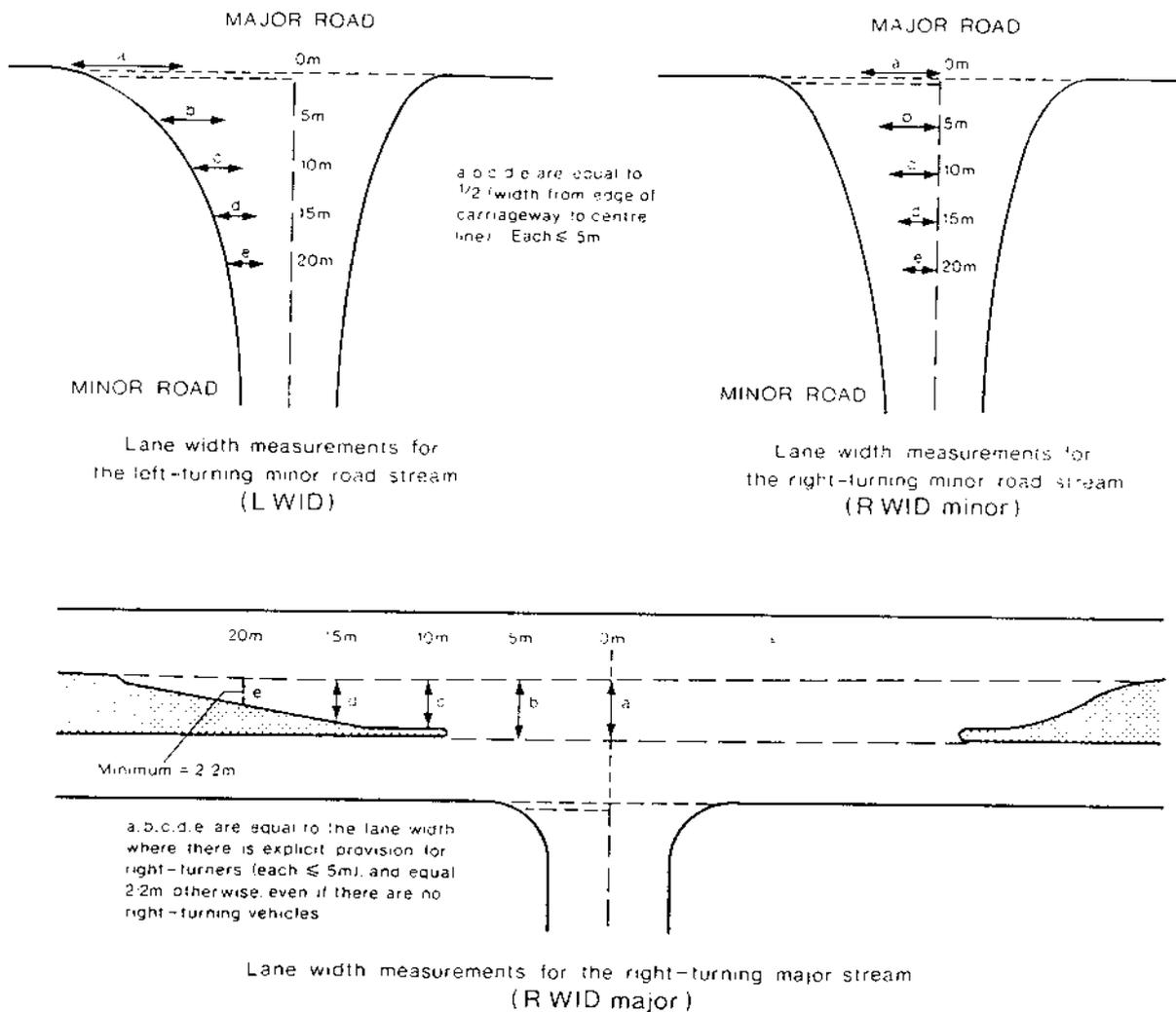
Figure 9/2: Lane Width for Non-Priority Streams

9.6   Visibility distances for the minor road streams, Vl and Vr minor

These are measured from points 10m back from the give way line on lines bisecting each lane. Visibility to

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the left Vl is measured from the offside lane to a line bisecting the far major road carriageway. Visibility to the right, Vr, is an average of the measurements made from each lane to a line bisecting the near major road carriageway. See Fig 9/3. All measurements are made at a height of 1.05m above the carriageway surface.
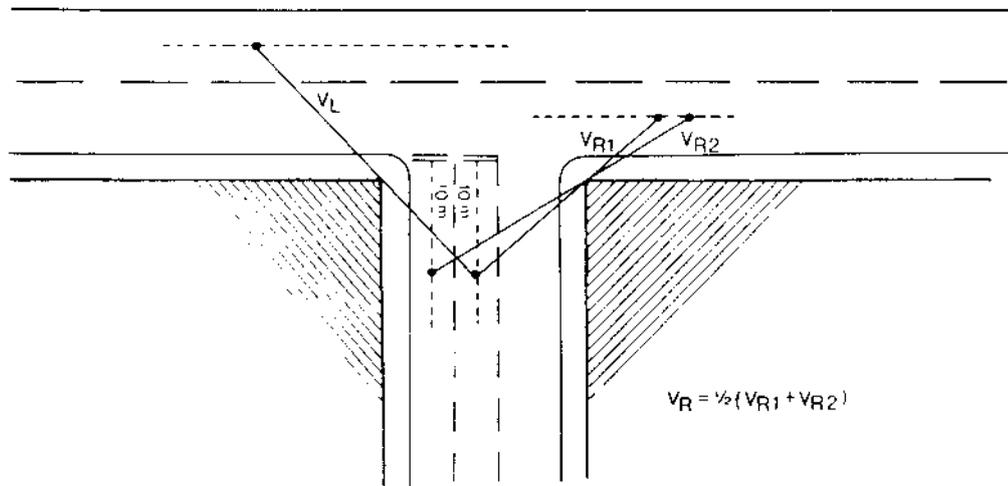

Figure 9/3: Measurement of Visibility Distances V_L and V_R

9.7   <u>Visibility distance for the major road right turning stream, Vr major</u> One visibility measurement is made, from the mid point of the right turning lane, or the position assumed by vehicles waiting to turn right, towards the oncoming major road traffic, at a height of 1.05 metres.

9.8   <u>Advice Note TD42</u> The user must also indicate whether or not the visibility requirements detailed in paragraph 8.2 of TD42, are met or not, as this affects geometric delay (see Part 6 paragraph 5.3)

Roundabouts - Geometric Parameters

9.9   The measurement of the geometric features required for the program to calculate capacities and delays (see Part 6 paragraph 8.4) is described in the following figures. The appropriate COBA input record is KEY 077.

9.10   <u>The approach half width v (AWID)</u> is measured at a point in the approach upstream from any entry flare, from the median line to the nearside kerb along a normal. See Figure 9/4.

9.11   <u>The entry width, e (EWID)</u> is measured from the point A along the normal to the nearside kerb in Figure 9/4.

9.12   <u>The entry radius, r (ERAD),</u> is measured as the minimum radius of curvature of the nearside kerb line at entry. See paragraph 8.4. For some designs the arc of minimum radius may extend into the following exit, but this is not important provided that a half or more of the arc length is within the entry region.
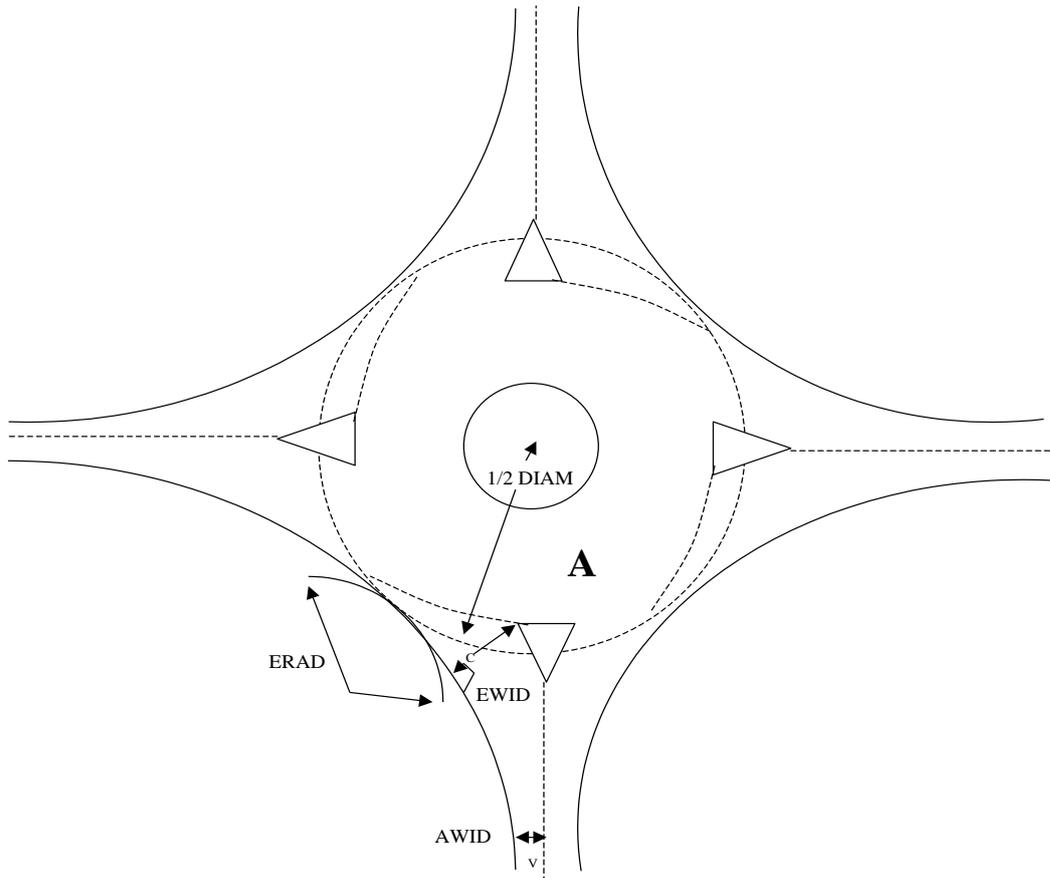
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Figure 9/4: Geometric Parameters for Roundabouts

9.13   The entry angle, φ (FI), serves as a geometric proxy for the conflict angle between entering and circulating streams. Three constructions are used; the first two apply to well defined conventional roundabouts and the third to all other types. For conventional roundabouts (ie those with identifiably parallel sided weaving sections) the construction is illustrated in Figs 9/5 and 9/6.
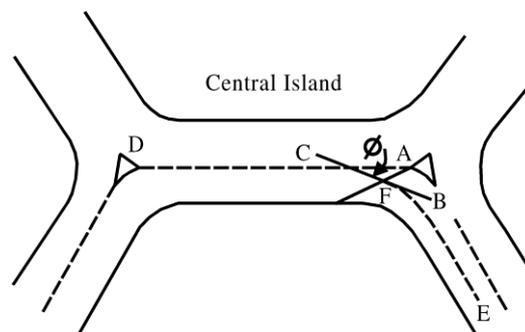

Figure 9/5: Entry Angle on Straight Weaving Sections

9.14   Fig 9/5 refers to roundabouts with approximately straight weaving sections in which the line parallel to the weaving section is AD where the point A is as in Fig 9/4 and D is the point nearest to A on the median island (or marking) of the following entry. Fig 9/6 shows the equivalent construction for roundabouts with curved weaving sections (for which the line AD is clearly not parallel with the weaving section). A' D' replaces AD as the line parallel to the weaving section.
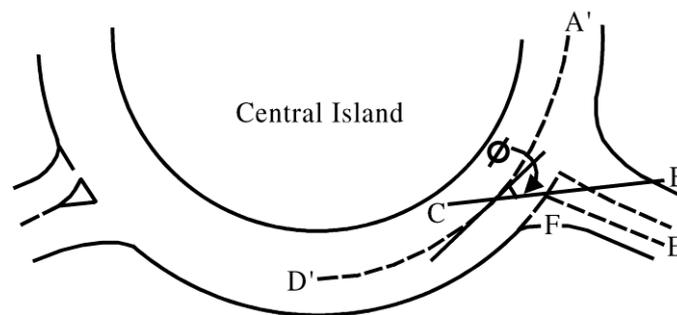
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Figure 9/6: Entry Angle on Curved Weaving Sections

9.15   In both cases the line BC is a tangent to the line, EF, which is midway between the nearside kerb line and nearside edge of any median island, at the point where it intersects the give way line is measured as the angle between the lines BC and AD in Fig 9/5, and as the angle between BC and the tangent to A'D' at the point of intersection in Fig 9/6.

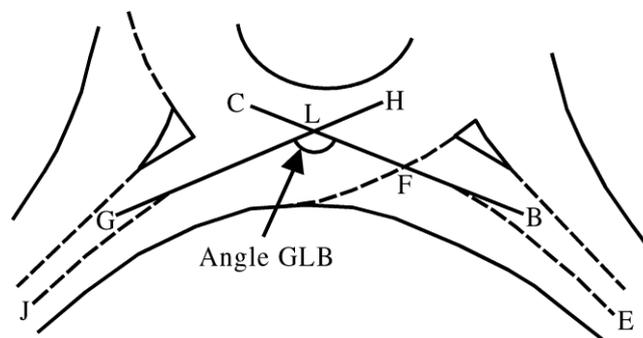9.16   On roundabouts with short weaving areas the construction is as in Fig 9/7.

Figure 9/7: Entry Angle on Short Weaving Areas

9.17   Here, the line BC is as in Fig 9/5 and the line GH is the tangent to the line, JK, which is in the following exit midway between the nearside kerb and nearside edge of any median island, at the point it joins the outer boundary of the roundabout circulation. BC and GH intersect at L is then defined by:

$$\emptyset = 90 - \tfrac{1}{2} \text{ (angle GLB)}$$

9.18   This applies if the right hand side is positive. But $= \emptyset = 0$ if the right hand side is zero or negative. GLB is the angle measured on the `outside' of the roundabout, that is on the side facing away from the central island.

9.19   The practical difference between this and the previous constructions is that in Figs 9/5 and 9/6 Ø is independent of the angle at which the following exit joins the roundabout whereas in Fig 9/7 Ø takes account of this angle. The reason is that for roundabouts with appreciable separation between entry and following exit (conventional roundabouts) the direction of circulating traffic depends on the alignment of the weaving section and is largely independent of the geometry of the following exit, but when the separation is smaller (as for off side priority roundabouts) circulating traffic which leaves at the following exit traces a path determined in part by the angle at which that exit joins the roundabout. The conflict angle reflects this difference.

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9.20   The flare length (FLEN). This is defined as half the length over which the flare is developed. The line CF' is parallel to BG and distance ½(e-v) from it. Usually CF' is therefore curved and its length is measured along the curve. The construction is illustrated in Fig 9/8.
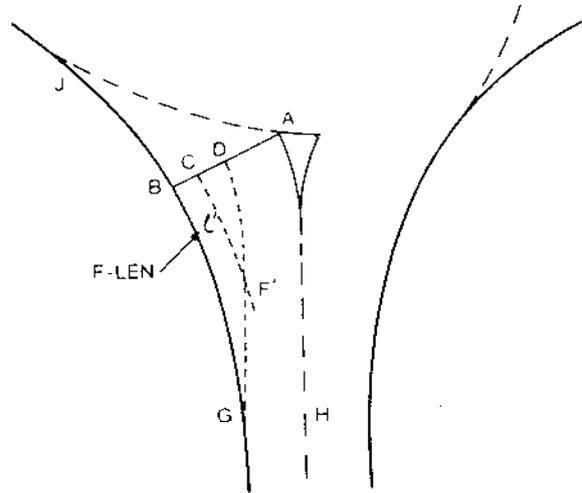

Figure 9/8: Flare Lengths for Roundabouts

9.21   The inscribed circle diameter (DIAM) is the diameter of the largest circle that can be inscribed within the junction outline. See Fig 9/4. In cases where the outline is asymmetric, the local value in the region of the entry considered is taken. The extreme case arises for a `double' offside priority roundabout at a 'scissors' crossroads. Fig 9/9 illustrates the determination of D in such cases.
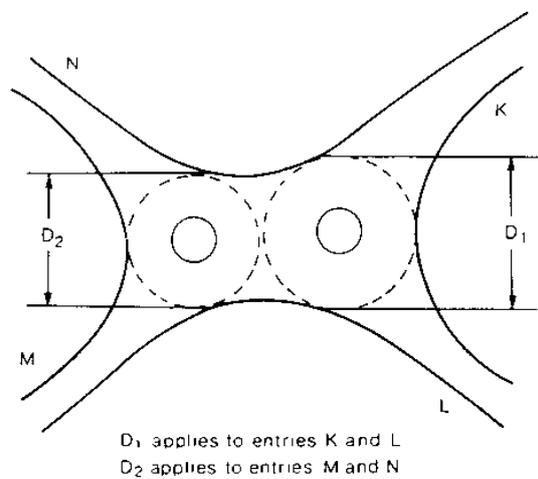

D_1 applies to entries K and L
D_2 applies to entries M and N

Figure 9/9: Inscribed Circle Diameter for Roundabouts

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Traffic Signals - Geometric Parameters

9.22   To calculate the capacity of the individual stop lines at traffic signals the only geometric parameters required by the program are the average lane width and uphill gradient for each movement. The appropriate COBA input record is KEY 080.

Merges - Geometric Parameters

9.23   To calculate the capacity of the merge area the only parameter required is the number of lanes. The appropriate input record is KEY 074.

Gates - Geometric Parameters

9.24   To calculate the capacity of the individual stop lines at traffic signals the only geometric parameter required by the program is the average lane width. The appropriate COBA input record is KEY 079.

Signalised Roundabouts - Geometric Parameters

9.25   The measurement of the geometric features required by the program to calculate capacities and delays is described below and shown on Figure 9/10. The appropriate COBA input record is KEY 081.
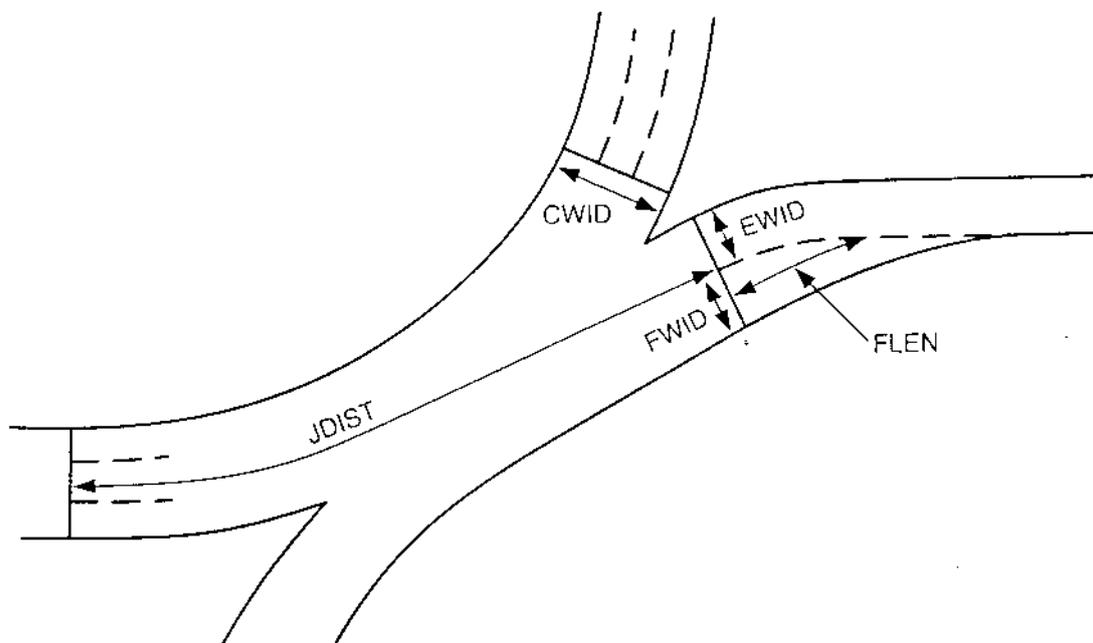

Figure 9/10: Geometric Parameters for Signalised Roundabouts

9.26   The entry width (EWID) is the width of the approach measured normal to the offside kerb and is entered to the program as the average width per lane. It should exclude any flare width (FWID) which is entered separately.

9.27   The flare width (FWID) is the width of the flare at the stop line measured normal to the kerb and is entered to the program as the average width per lane. It should exclude the entry width (EWID) which is entered separately.

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9.28   The flare length (FLEN) is defined as for conventional roundabouts as half the length over which the flare is developed (see paragraph 9.20). Flares less than 5m in length should be ignored.

9.29   Distance from entry link stop-line to next downstream stop-line (JDIST) is the distance travelled on the ground and not the straight line distance.

9.30   The circulating width (CWID) is the width of the approach to the circulating stopline measured normal to the offside kerb and is entered to the program as the average width per lane. No allowance is made for any gradient that may exist.

9.31   The inscribed circle diameter (DIAM) is the diameter of the largest circle that can be inscribed within the junction outline (see Fig 9/4). This parameter is used for the calculation of geometric delay and in cases where the outline is asymmetric an average value should be used.

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