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THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF THE ENVIRONMENT
FOR NORTHERN IRELAND

Junctions and Accesses : Determination of Size of Roundabouts and Major / Minor Junctions

Summary: This Advice Note gives recommendations for determination of Design Reference Flows for roundabouts and major / minor junctions using available traffic figures. Layouts of different size based on these Reference Flows are then considered in relation to engineering, economic, environmental and other factors. Examples of how final decisions are reached are given.

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VOLUME 6 ROAD GEOMETRY
SECTION 2 JUNCTIONS

TA 23/81

JUNCTIONS AND ACCESSES: DETERMINATION OF SIZE OF ROUNDABOUTS AND MAJOR/MINOR JUNCTIONS

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1. INTRODUCTION

1.1 This Advice Note gives guidance on the use of available traffic figures in determining the size of roundabouts and major/minor junctions following the issue of the Traffic Appraisal Manual (TAM) (Ref 1) and major research results (Refs 2,3 and 4). Procedures, based on the TAM, outlined to derive "Design Reference Flows". These are used in arriving at trial designs, and caveats which need to be applied are described. The economic aspects of each of the trial designs are then considered, along with operational assessments which use either the computerised methods of Ref 3 or, for manual calculations, the formulae in Appendices 1 and 2. Engineering, environmental, operational and other aspects of the junction type needing to be taken into account are listed. Examples of the use of various factors in reaching a decision are given in Appendix 3.

1.2 A new approach to handling roundabout traffic flows is described at section 6, and the following documentation dealing with flow calculations is superseded by this Advice Note:-

TE Design Note No.1 (with the exception of visibility and entry spacing requirements which are temporarily retained)

Technical Memorandum H2/75

Paragraphs 16,17,18,47,54,67 and 73.

Sub-paragraphs 49a,49b,49e and that part of paragraph 49 on page 9.

Table 1 and the reference to it in paragraph 8.

Figures 3 and 4.

The whole of Appendix A with the exception of Fig 3.

1.3 Roundabout layout requirements in Technical Memorandum H2/75 and TE Design No.1 will remain, pending the issue of further advice. The one exception of this is layout advice regarding circulatory carriageway width (see paragraph 6.5.1).

1.4 Technical Memorandum H11/76 (including Amendment No.1) and Departmental Advice Note TA 5/80 are now completely superseded. A new approach to handling major/minor junction traffic flows is described at section 7 of this documents.

1.5 Major/minor layout recommendations are fully detailed in Ref 5.

2. SCOPE

2.1 This Advice Note is concerned with determining the size of roundabouts and major/minor junctions. It does not give guidance on the choice between a roundabout or major/minor junction for a particular site (that guidance will be given in Ref 6). Guidance, however, is given for choosing between different sizes of roundabout and between the various types of major/minor junctions.

2.2 There will be cases in assessing major/minor junctions, especially in rural areas, where even single lane dualling will be unable to cope with the traffic flows. Such cases are beyond the scope of this documents and require the use of the MIDAS program (Ref 7) which should also be used to compare roundabouts with compatible forms of grade separated junctions.

2.3 Manual or computerised methods may be used and a different approach to each is identified, but since complex and time consuming calculations are involved the latter method is preferred. However, if manual calculations are unavoidable the use of a desk calculator is recommended, since repetitive calculations are involved.

2.4 The depth to which analysis is taken should be related to the extent of any changes which might result. There is no point in carrying our extensive analysis if only trivial changes are likely to occur.

3. DEFINITIONS

For the purpose of this document the following definitions apply:-

3.1 Design Reference Flows

In order to generate alternative junction options for assessment frameworks, trial traffic flows are needed for detailed design. These are called "Design Reference Flows". Full details of their derivation are given in section 4.

3.2 Capacity

In this Advice Note "Entry Capacity" of "Capacity" at roundabouts is defined as the maximum inflow from an entry to a roundabout when the traffic flow at that entry is sufficient to cause continuous queuing in its approach road. Major/minor junction flows, some of which are intermittent turning movements, are considered to be at "Capacity" when there is continuous queuing feeding a particular turning movement. Not all movements need be at "Capacity" for the junction to be considered at "Capacity".

4. FROM TRAFFIC FIGURES TO DESIGN REFERENCE FLOWS

4.1 General

4.1.1 It is recommended that section 13.5 (Junction Appraisal) and sections 13.6 (Preparation of Traffic Figures for use with our Departmental Publications) of the TAM are read by those preparing traffic figures, along with section 6.2 of the COBA Manual (Ref 8).

4.1.2 Two sets of traffic flows are required to design and evaluate junction options, namely Design Reference Flows and Annual Average Daily Traffic (AADT) flows.

4.2 Design Reference Flows

4.2.1 Design Reference Flows are the hourly traffic flow rates used in undertaking the detailed design of alternative practical junction layouts at a site. These alternative designs are then assessed in terms of their economic, environmental and operational impacts and choice between the competing options is made.

4.2.2 The range of Design Reference Flows used for the generation of options at a site should be wide enough to embrace junction designs which will be optimal economically or operationally, but no so wide as to produce junction designs which would clearly be an under- or over-provision.

4.2.3 There are exceptions to this rule such as a site which has severe land take constraints or a scheme which, although an under-provision, still represents a worthwhile improvement on the existing situation. It is likely that junction designs providing sufficient capacity for the traffic flows which are expected to occur 10 to 15 years after the scheme is implemented will be economically and operationally acceptable. (The economic viability of a scheme is assessed over a 30 year period with discounting placing the emphasis on the early years: traffic growth beyond the late 1990's is expected to be much less than in the intervening period.)

4.2.4 In choosing a peak hourly flow to represent the Design Reference Flow, the function of the road in the network (recreational, urban or inter-urban) must be taken into account. It is most unlikely that a junction designed to carry the very highest peak hourly traffic flows in a future year will prove economically viable. These very highest hourly flows will be many times greater than the Annual Average Hourly Traffic flow (AAHT = AADT/24).

4.2.5. The highest hourly flow that would typically generate viable junction options on recreational roads, where the traffic flows are much greater during the high season than at other times of the year, might be the 200th highest hour, some congestion and delay being almost inevitable during the exceptionally high peak. In urban areas where there is very little seasonal variation, designs which cater for the 30th highest hourly flow are likely to be justified. Four inter-urban roads, designs which cater for the 50th highest hour are likely to be most acceptable.

4.2.6. However, the use of Design Reference Flows is not a substitute for assessment. The purpose of these flows is to allow the detailed design of alternative feasible junctions to take place, the performance of which can then be assessed.

4.2.7 Design Reference Flows should also encompass the range of turning movements than can be expected at a site. Junction options must not be assessed on a single set of turning movements only.

4.3 Annual Average Daily Traffic Flows

Estimates of the AADT flows on the approach roads to a junction are required for economic evaluation, using COBA 9, or for use in MIDAS, and possibly for the environmental appraisal of junction options.

4.4 Derivation of Traffic Flows

The manner of the derivation of both Design Reference and AADT flows will depend upon whether the junction already exists or where it is a new junction, and is summarised in Fig A on page 11.

4.4.1 The Improvement o Existing Junctions: At existing junctions, turning movements, peak hour and daily traffic flows can be measured directly. These measurements will contain appreciable statistical error (see TAM 6.3.4, 6.10 and Appendix D11, example 3), but they may be used as the basis for the derivation of Design Reference Flows provided the statistical error is recognised, and as long as significant diversion away from the existing junction is not suspended at the existing level of congestion. The derivation of traffic flow data for the design and evaluation of junction options would be:-

Example A

A manual classified turning count should be carried out at the junction to establish both link flow volumes and turning proportions. In an urban area, or on an inter-urban road the count should encompass an average peak during a normal weekday (Monday to Thursday) in a neutral month (April, May, September or October). On a recreational road the timing of the count is more difficult. The intention should be to observe a level of traffic flow which heavily loads the junction without actually causing congestion, but not one which is so very high as to occur only a few times each year. A count taken on an average weekday (Monday to Thursday) during the high season may well be appropriate and practical. Design Reference Flows may be estimated by increasing the observed hourly inflows for the morning and evening peak hours by the factors given in the National Road Traffic Forecasts (NRTF), or by reference to TAM Appendix 12.2, to produce a range of high and low future year flows. The turning proportions and tidal flow patterns observed in the base year may be used as the basis for the development of possible future year situations by rounding, to allow for the accuracy of the base year count and the uncertainty of forecasting. For example, an observed right hand turning proportion of 18% might be considered as a 10% to 30% range in the future.

AADT flows on the approach links may be estimated from the short period turning counts using the factors given in TAM Appendix D14, or local factors where these are suitable and available. The accuracy of the national factors is quoted in TAM Appendix D14 (in terms of their coefficients of variation). The accuracy of any local factors used should also be known. Once again, growth factors given in the NRTF or in TAM Appendix 12.2 should be used to estimate the range of future year traffic flows.

4.4.2 The Appraisal of New Junctions (future provisions): Where a junction is to be provided on a new road, or where the traffic flows through an existing junction are expected to change significantly in the future due to other network changes or the relief of congestion at the existing junction, future year traffic flows for both the design and evaluation of junction options must be derived by the use of a traffic model. Traffic models are capable of producing reasonable estimates of 12,16, or 24 hour link flows in a future year, but any direct estimates of peak hourly tidal flows and turning proportions produced by a traffic model are likely to be very uncertain. Estimates suitable for use in the generation and evaluation of junction options can be produced, however, in the following ways:-

Example B

AADT flows on the approach roads to a junction may be derived by factoring the 12,16 or 24 hour link flow information output from the model (for a particular year taken from the time period upon which the model is based) to represent 24 hour AADT flows using the factors in TAM Appendix D14.

Design Reference Flows are derived from these AADT flow estimates. Firstly, the AAHT flow is calculated (AAHT = AADT/24). The AAHT on each link is then factored to represent an appropriate highest hourly flow (see paragraph 4.2.5) using the factors given in TAM Appendix D14. Tidal flows must then be considered (a 60:40 split is often appropriate), and turning proportions estimated so that junction options may be designed. The turning proportions output by the traffic model should be manipulated to cover a range of turning movements thought likely to occur in practice. For example, a traffic model prediction of 152 vehicles turning left, with 1708 vehicles proceeding straight ahead and 572 vehicles turning right might be interpreted as a heavy straight on movement, with a substantial proportion of right turning traffic and a small left turning movement. This could in turn lead to the use of the following turning proportions for the generation of feasible junction options:-

left turn	zero to 10% of approach flow
straight ahead	60% to 80% of approach flow
right turn	40% to 10% of approach flow

4.5 Allowing for Short Term Variation in Flow

4.5.1 Design options will be tested against the Design Reference Flows, the derivation of which has just been described. However, traffic does not usually arrive at a junction at a constant or uniform rate. For short periods the arrival rate will be equivalent to a flow which is higher than the Design Reference Flow, at other times the rate will be equivalent to a lower flow. This short term variation in flow may be taken into account when applying the programs ARCADY and PICADY (see section 5) by using a "flow profile".

4.5.2 For existing junctions, the flow profile may be input to the program in 15 minute intervals, based upon the existing observed arrival pattern where this has been recorded. For new junctions, Ref 3 gives typical flow profiles which can be generated by the program in 5 minute.

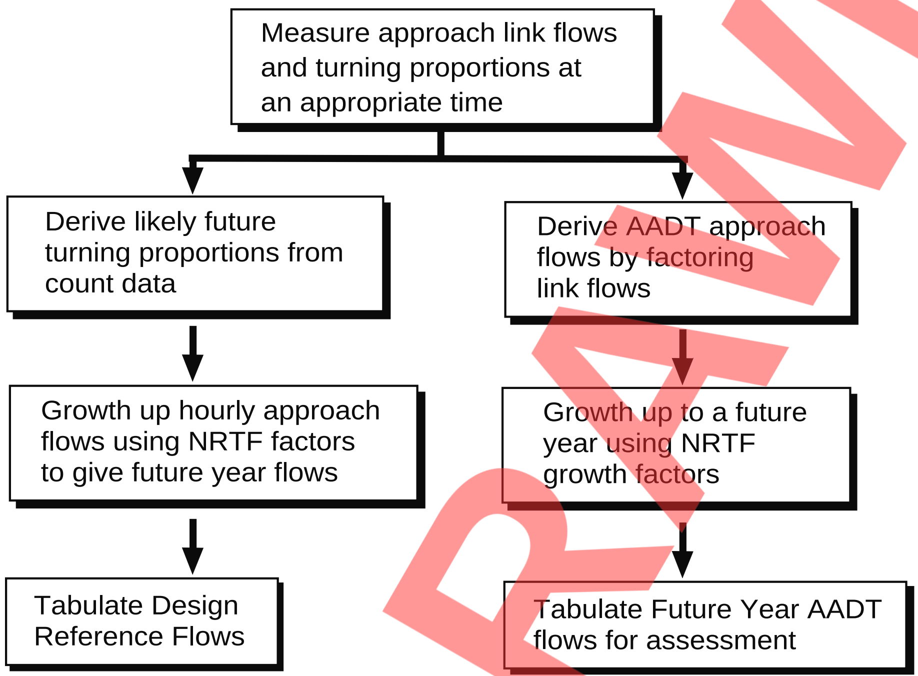
4.5.3 When calculating manually, the short term variation in traffic flow may be allowed for by using an hourly flow 1.125 times the Design Reference Flow, which produces a similar Reference Flow/Capacity (RFC) ratio to the maximum value produced by the program.

4.5.4 At a roundabout, the short term variation in flow will affect not only the entry flows but also the circulating flows.

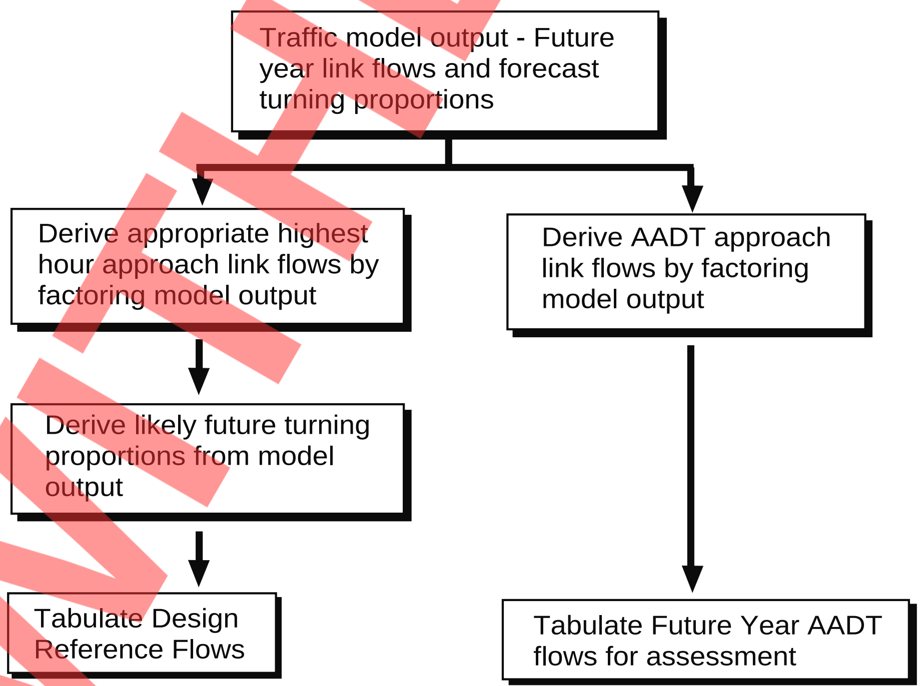
4.5.5 At major/minor junctions, the short term variation in flow on both the major and minor arms should be considered.

FROM TRAFFIC FIGURES TO DESIGN REFERENCE FLOWS

Existing Junctions (paragraph 4.4.1)



Future Provision (paragraph 4.4.2)



5. CAPACITY FORMULAE AND METHODOLOGY

5.1 General

5.1.1 The new formula for capacity calculation for roundabouts (Ref 2) evaluates the conflict between the traffic entering the roundabout and the circulating flow already on the roundabout. In most cases iterative calculations are necessary to compute the various flows gaining entry against the different circulating flows. The computer program ARCADY (Ref 3) has been developed to carry out these calculations and evaluate queues and delays.

5.1.2 Similarly, the new formulae for capacity calculation for major/minor junctions (Ref 4) evaluate the conflict between various traffic streams. The computer program PICADY (Ref 3) can carry out these calculations and evaluate queues and delays.

5.2 Variation

5.2.1 It must be stressed that the calculated capacities, queues and delays are AVERAGE values of very broad distributions. The formulae used are based on multiple regression analyses from observations from a large number of sites. Actual values can vary about the average due to:-

Site to site variation

Day to day variation

5.2.2 The site to site variation has been estimated, and is covered by the procedures in sections 6 and 7. As far as day to day variation is concerned, this will manifest itself in practice as variations in the queue lengths and delays at any given time in the peak period. The formulae merely calculate the average values over many days.

5.3 Range

The procedures in sections 6 & 7 are based on determining RFC ratios. Unlike past methodology, however, designers should not strive to obtain a unique value. A range of situations must be considered and the advantages and disadvantages of each one assessed.

5.4 Generation of Options

5.4.1 Fig B below may be useful to designers when considering options for a site. For single carriageway roads it shows approximately the various levels of T-junction which may be applicable for different combinations of flows. The information takes into account geometric and traffic delays, entry and turning stream capacities, and accident costs.

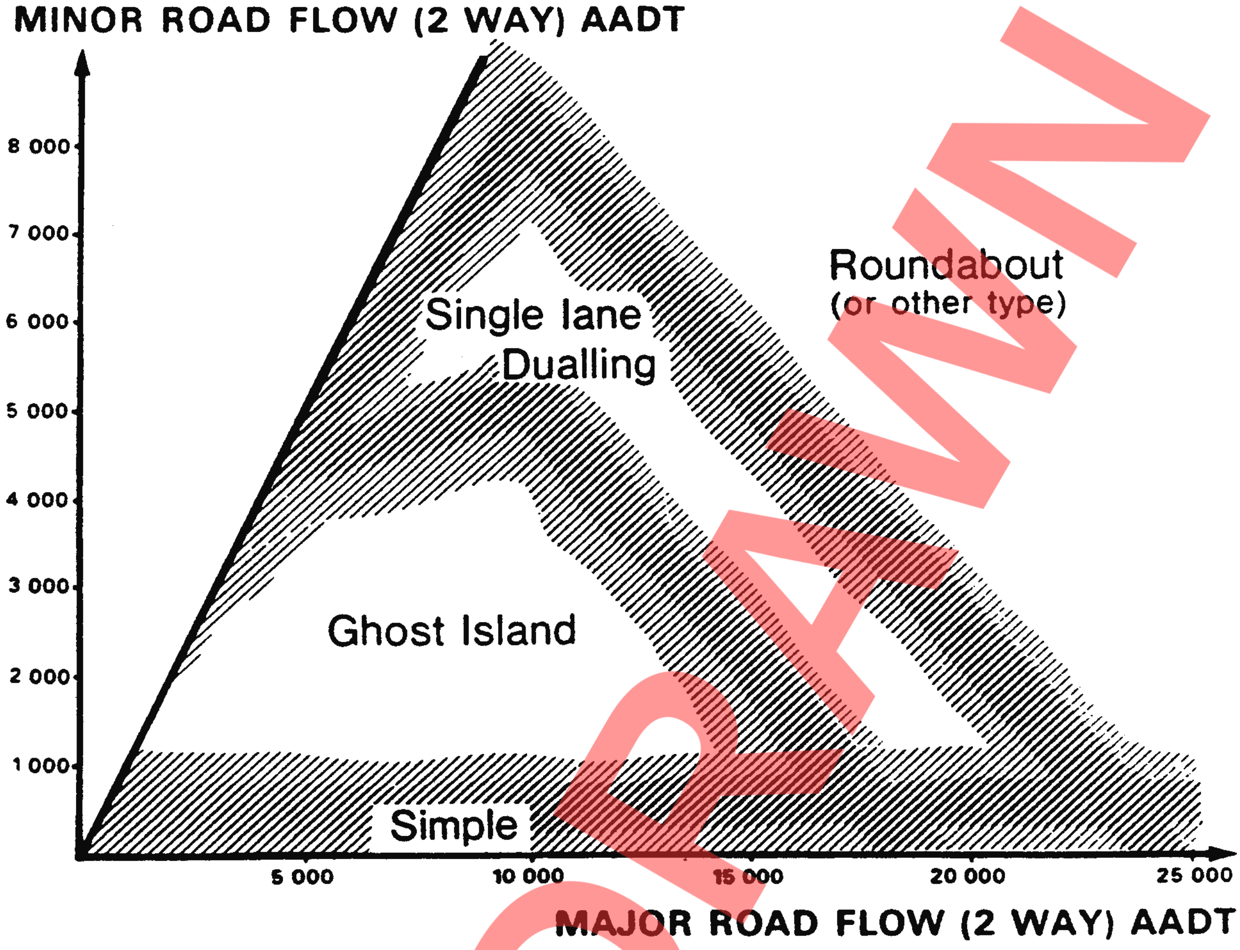


Fig B

5.4.2 For dual two lane carriageways, major/minor junctions are unlikely to be cost effective where the minor road flow is expected to exceed about 3,000 vehicles AADT 2-way.

6. ROUNDABOUT DESIGN ASPECTS

6.1 General

The equation for the prediction of entry flow into a roundabout as a function of the circulating flow and entry geometry is applicable to all types of single at-grade roundabout whether mini or normal types (see Appendix 1). Having developed a range of Reference Flows as described in section 4, a designer should use the equation to produce trial designs for assessment. The method of handling these figures will depend on whether manual or computerised methods are used. It is not realistic to calculate queue lengths and delays manually, on the other hand this is a normal part of the computer output.

6.2 The Reference Flow/Capacity Ratio

The RFC is an indicator of the likely performance of a junction under a future year traffic loading. It should be calculated or computed for each trial design. Due to the site to site variation mentioned in paragraph 5.2 there is a standard error of prediction of the entry capacity by the formula of + or - 15% for any site. Thus if any entry RFC ratio of about 85% occurs queuing will theoretically be avoided in the chosen design year peak hour in 5 out of 6 cases (schemes). Similarly, if an entry RFC ratio of 70% occurs queuing will theoretically be avoided in 39 out of 40 cases (schemes). The general use of designs with a RFC ratio of about 85% is likely to result in a level of provision which will be economically justified. There will be cases, however, where the adoption of a lower figure will be justified: for example, where the cost of a higher level of provision is low in both economic and environmental terms, or where space for enlargement is unlikely to be available in the future at a reasonable cost and thus the cost of being wrong becomes unreasonably high. On the other hand, if there are cost or environmental implications in providing higher capacity, for instance in urban areas, then even the 85% ratio may be unsuitable and a higher ratio, with consequent queuing, will have to be accepted (to an extent assessed by the reduction of economic or environmental impact).

6.3 Manual Calculation

The RFC ratios should be calculated using the formula given in Appendix 1. The Design Reference Flows should be multiplied by 1.125 as described in paragraph 4.5.3.

6.4 Computer Computation

The computer program ARCADY (Ref 3) should be used. The appraisal can be based on either a RFC ratio of 85% or, in certain cases, a higher or a lower ratio in the same way as described in paragraph 6.2. In computing this, a time segment length of not less than 5 minutes should be used to build up the follow pattern during the peak. The program prints out the RFC ratios (Labelled Demand/Capacity in the output), queue lengths and delays at each entry for each time segment. An inspection can therefore be made, for each arm in turn, of the queue length and delay if the RFC ratio reaches 85% (or 70%). Circumstances will vary, and it may often not be possible to provide the same RFC ratio at all approaches, but the aim should be to achieve a reasonably balanced design in this respect. On the other hand, ratios higher than 85% could be used at some less important entries if exceptionally low ratios are unavoidable at other, though the possibility of excessive queuing at any entry should be avoided.

6.5 Other Factors

Other factors which need to be considered are as follows:-

6.5.1 Layout Factors: The trial design should be calibrated where necessary to obtain operational efficiency by adjusting the entry widths and the effective length of flares. Whilst Appendix 1 gives the range of the parameters for which the predictive equation is valid, the following list gives the normal practical limits of those parameters in new design.

e	Entry width	4.0 - 15.0m
v	Approach half width	2.0 - 7.3m
l^l	Effective length of flare	1.0 - 100.0m
r	Entry radius	6.0 - 100.0m
ϕ	Entry angle	10 - 60 degrees
D	Inscibed circle diameter	15 - 100m

The circulatory carriageway width around the roundabout should be constant at about 1.0 to 1.2 times the greatest entry width, subject to a maximum of 15m. Full details of geometric layout aspects will be given in Ref 9, but in the meantime reference should be made to paragraphs 1.2and 1.3 herein.

6.5.2 Safety: Roundabouts are normally the safest form of at-grade junction over a wide range of entry flows and approach speeds. They are especially effective where a heavy right turn occurs, which can lead to accidents at major/minor junctions. Where the 85 percentile approach speed on any approach (see Ref 10 for new schemes and Ref 11 for existing roads) exceeds 50 kph the roundabout must be designed so that any vehicle path through the roundabout has a maximum radius of 100m. For instance, if 3 entry lanes are chosen in rural areas, the minimum size required for 3 way and 4 way orthogonal junctions with single 7.3m carriageways simply to restrict vehicle speeds becomes nearly 60m Inscribed Circle Diameter (ICD). Since there is evidence that speeds round curves are increasing over the years, providing adequate deflection is important (see also Ref 12).

6.5.3 Siting of Roundabouts: Steep downhill gradients should be avoided at roundabout approaches or flattened to a maximum of 2% before entry where possible. The frequent occurrence of roundabouts should be avoided on rural roads. Roundabouts should be avoided between traffic signalled junctions subject to linked control.

6.5.4 Pedestrians and Cyclists: Roundabouts often need associated pedestrian crossings, especially in urban areas, when pedestrian and vehicle flows are high (see Ref 13). High volumes of cyclists may need special subways (see Ref 14) or other measures, but these must not create excessive diversions otherwise usage may be reduced.

6.5.5 Major Road Type: Roundabouts are equally suitable for both urban and rural areas. Mini roundabouts must only be used at existing junctions where the 85 percentile approach speeds on all approaches is less than 50 kph and space limitations preclude a normal roundabout.

6.5.6 Long Vehicles: In urban sections (where deflection is not necessary to reduce speeds), the space needs of long vehicles when turning may prevail over capacity requirements. The smallest size of roundabout negotiable by most long vehicles has an ICD of about 28m with a central island of 4m diameter. In some urban areas there may be insufficient space even for a small roundabout and a mini roundabout is recommended in these cases.

6.5.7 Land Take and Environmental Considerations: Small roundabouts do not use large amounts of land, sometimes less than the alternative single lane dualling, and can thus be attractive in sensitive areas. However, it is standard practice to light roundabouts, and this can be a problem in rural areas.

7. MAJOR / MINOR JUNCTION DESIGN ASPECTS

7.1 General

For new rural T-junctions or accesses, simple major/minor layouts should only be used when the flow on the minor road or access is not expected to exceed about 300 vehicles (2-way total) AADT. At existing rural and at urban T-junctions and accesses, the cost of upgrading a simple junction to the next highest junction in the hierarchy (ie a ghost island junction) will vary from site to site and could be quite expensive. However, upgrading should always be considered where the minor road or access flow exceeds 500 vehicles (2-way total) AADT or where a right turning accident problem is evident (See Ref 5). New crossroads, or development of existing crossroads, are not recommended for safety reasons. Staggered junctions should be used instead. Right/Left staggers are preferred to left/right staggers where practical. For design involving flows greater than the low flows described above, use should be made of equations which are available for the prediction of possible minor road entry flows into a major/minor junction as a function of the flow/geometry at the junction. These equations are reproduced at Appendix 2 and are applicable to all types of major/minor T-junctions including staggered junctions. As recommended for roundabouts, the range of Reference Flows developed should be used to produce trial designs for assessment. However, unlike roundabouts, consideration of a lower RFC ratio is recommended as a general rule when considering 100 kph design speed single carriageways or high speed dual carriageways. This is because the formulae have not been derived from these latter types of road. As far roundabouts manual or computerised methods may be used, but it is not realistic to calculate queue lengths and delays manually though this is a normal part of the computer output.

7.2 Manual Calculations

Using the formulae at Appendix 2 the RFC ratios of the various turning movements should be examined. The Design Reference Flows should be multiplied by 1.125 as described in paragraph 4.5.3. The standard error of capacity prediction due to variation between sites mentioned in paragraph 5.2 is 13%. Queuing should not occur in the various turning movements in the chosen design year peak hour in 5 out of 6 cases (schemes) if a maximum RFC ratio of about 85% is used. Similarly, if a maximum RFC ratio of about 75% is used queuing will theoretically be avoided in 39 out of 40 cases (schemes). At sites having no particular space restriction, and also where the design speed may be 100kph, usually in suburban and rural areas, the latter ratio should be used as a design yardstick, but in urban areas the former may be appropriate.

7.3 Computer Computation

The computer program PICADY (Ref 3) should be used. The appraisal should, as in paragraph 7.2, normally be based on a RFC ratio of about 85% in urban cases, or 75% in rural areas. In computing this, a time segment length of not less than 5 minutes should be used to build up the flow pattern during the peak. The program prints out the RFC ratios (labelled Demand/Capacity in the output), queue lengths and delays for each turning movement, for each time segment. Right turning queues and delays in the minor road should be virtually avoided in practice by use of the 75% RFC ratio. The loss of Nett Present Value (NPV) on a typical road scheme containing major/minor junctions by using the 75% factor instead of 85% may be relatively small in marginal cases where, for instance,the option between a ghost island and the next upward step in the hierarchy (single land dualling) is being investigated. Nevertheless, this sort of comparison should be sent down in the overall framework of design (see section 8).

7.4 Other Factors

Other factors which need to be considered are as follows:-

7.4.1 Restraint in the Use of some Dimensions: For the purpose of manual calculation of computer computation of turning stream capacities the maximum values used for central reservation width and visibilities should be 10m and 250m respectively even if it is proposed to provide physically greater values when the junction is constructed.

7.4.2 Safety: Major/minor junctions have the advantage that major road through traffic is not impeded, except at simple junctions. However, the high speed difference between through and turning traffic results in a poorer safety record than for roundabouts. See section 4 of Ref 5, and Ref 12.

7.4.3 Siting of Junctions: Major/minor junctions at or near the crests of hills should be avoided where possible. Frequent junctions on single carriageways reduce major road traffic speeds and thus link user benefits. See section 7 and Appendix 5 of Ref 5.

7.4.4 Pedestrians and Cyclists: Major/minor junctions are easier to negotiate for pedestrians and cyclists than roundabouts, especially where minor road channelising islands are present.

7.4.5 Major Road Type: Major/minor junctions which allow right turns should never be used on urban or rural D3 all purpose roads. Full dualling locally should not be used at major/minor junctions on otherwise single carriageway roads. See paragraph 6.5.1 of Ref 5.

7.4.6 Long Vehicles: The recommended layouts in Ref 5 provide adequate turning clearance for long goods vehicles as defined therein, but the longest vehicles can still present a risk by overhanging central islands while turning. Special cases should be considered on their merits.

7.4.7 Land Take and Environmental Considerations: Some single lane dualling layouts require as much land as, and sometimes more land than, equivalent roundabouts. It is not mandatory to light major/minor junctions in rural areas.

8. DECISION BETWEEN OPTIONS

8.1 General

The overall design/evaluation/decision process may be summarised as shown in Fig C below:-

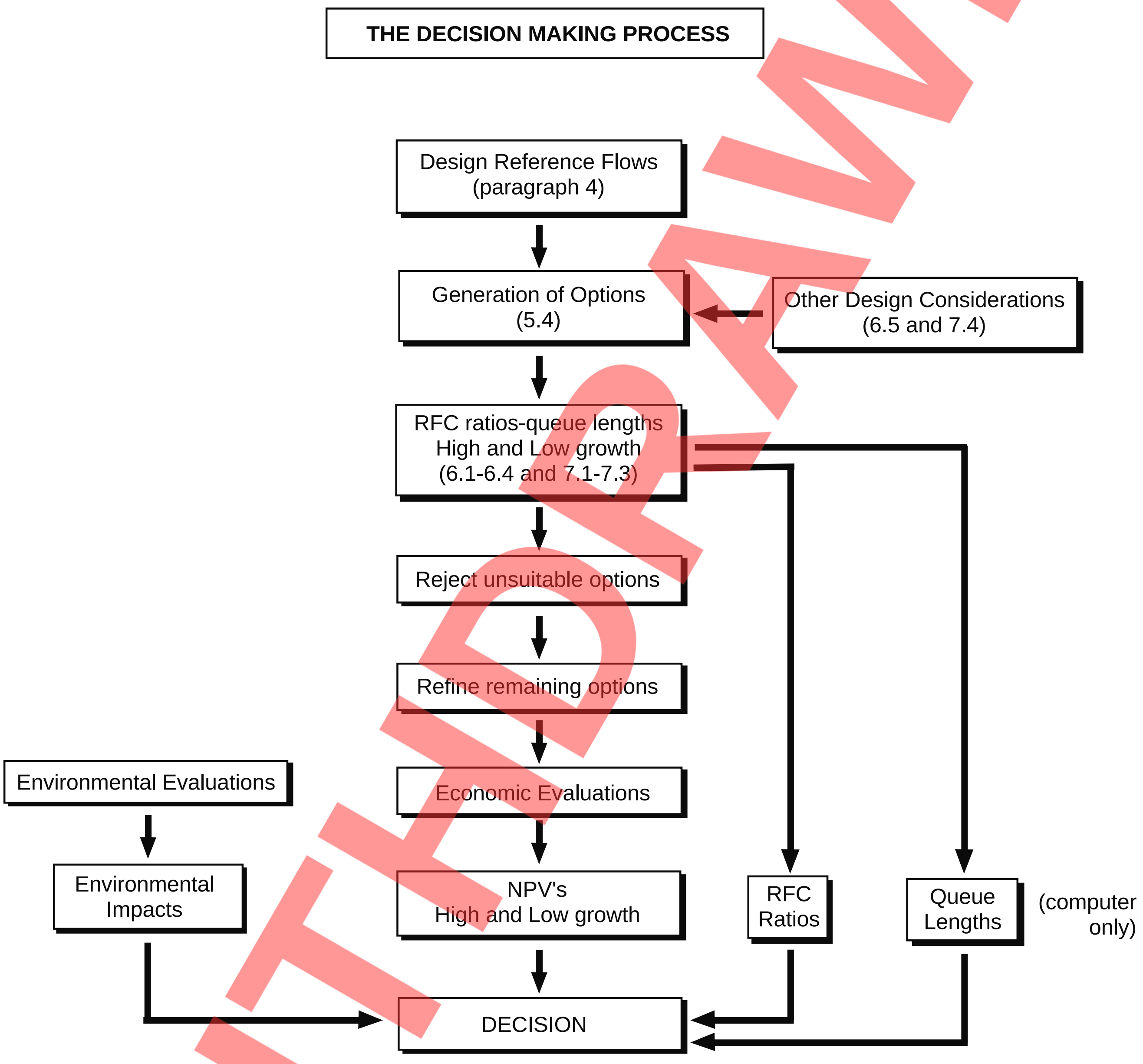


Fig C

8.2 Operational Evaluation

A decision may be taken to exclude certain options from further consideration when the RFC ratios becomes available. For instance, in the first example at Appendix 3 the RFC ratio of the 63m ICD roundabout at high growth is found to be a maximum of 97%. Bearing in mind the recommendations of sections 6 and 7, this option could have been rejected at this stage but, for the sake of illustration, it was retained and subjected to a full assessment.

8.3 Economic and Environmental Evaluation

8.3.1 The results of the economic and environmental evaluations should be presented in the form of an assessment framework. This will be a much simplified version of the frameworks shown in Refs 15 and 16, and it will be tailored to suit the circumstances of the particular junction under consideration.

8.3.2 The economic output is usually an important part of the framework. Additionally all relevant environmental impacts which are different between options should be included under the following main descriptions:-

a) Travellers
The cost of the delays to traffic over the scheme life for the trial designs should be evaluated at high and low growth using COBA 9 or MIDAS. In the course of this evaluation the compatibility between the Design Reference Flow turning movements used and the overall daily turning movements used in COBA 9, MIDAS or variants, should be taken into account. Small differences in delay costs should not be given too much weight. Changes in vehicle operating costs, accidents, driver comfort and convenience, view from the road and amenity will not differ to any appreciable extent between small changes in the same junction type and can be ignored.

b) Occupiers of Property
Sometimes there will be sufficient differences in land take, possibly involving demolition, to make an important issue in the choice between variants of the same type of junction. However, there are unlikely to be significant differences in disruption during construction, noise changes, visual effects, and severance.

c) Conservation and Enhancement Policies
It is unlikely that variants of the same junction type, will affect these issues unless, for instance, a listed building is close to the junction.

d) Financial Effects
The incremental NPVs for the trial designs at high and low growth should be sent out, together with construction costs.

8.4 Making a Decision

8.4.1 The various options can be assessed in pairs, the significant advantages of each being extracted from the framework and examined as shown in the examples in Appendix 3. The preferred option may be determined by a process of sequential elimination.

8.4.2 RFC ratios and usually queue lengths will be available at the final assessment stage but they will not necessarily predominate in the decision. If, however, there is a reason to believe that the economic evaluation tool is unable to model adequately the particular layout or traffic conditions, they may well provide overriding evidence.

8.4.3 Given the sensitivity of delays to the parameters in the junction capacity formulae and the uncertainty of turning movements, small differences in NPV are unlikely to be meaningful. In such cases, a lower capital cost, better safety performance or a higher capacity would carry more weight in the final decision.

9. REFERENCES

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4 SR582 The traffic capacity of major/minor priority junctions: TRRL: 1980.

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6 TA 30/82 Choice Between Options: DTp: 1982 (in preparation).

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8 lCOBA 9 Manual: DTp: 1981.

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10 TD 9/81 Highway Link Design: DTp: 1981.

11 TA 22/81 Vehicle Speed Measurement on all purpose roads: DTp: 1981.

12 "Accidents at four-arm roundabouts and dual carriageway junctions:- Some preliminary findings": Traffic Engineering and Control Vol.22, No.6, pages 339-344: June 1981.

13 TA 10/81 Design Considerations for Pelican and Zebra Crossings (and Addendum 16/1/81) DTp: 1981.

14 TD 3/79 Combined Pedestrian and Cycle Subways: DTp: 1979.

15 TD 8/80 Frameworks for Trunk Road Appraisal: DTp: 1980.

16 TA 7/80 The Preparation of Frameworks for Trunk Road Appraisal: DTp: 1980.

17 SR721 The capacity of some grade separated roundabout entries: TRRL: 1982.

10. ENQUIRIES

ROUNDABOUT CAPACITY FORMULA

1. The best predictive equation for the capacity of any roundabout entry (except those at grade separated interchanges, for which see paragraph 2 below) found by research to date is as follows:-

$Q_E = k\ (\ F - f_c Q_c\ )$ when $f_c Q_c$ is less than or equal to F

$= 0$ when $f_c Q_c$ is greater than F.

Where

Q_E	=	Entry flow in pcu/hour (1 HGV = 2 pcu)
Q_c	=	Circulating flow across the entry in pcu/hour
k	=	$1 - 0.00347\ (\ \phi - 30\ ) - 0.978\ \{(1/r) - 0.05\}$
F	=	$303x_2$
f_c	=	$0.210t_D\ (\ 1 + 0.2x_2\ )$
t_D	=	$1 + 0.5\ /\ (1+M)$
M	=	$\exp\ \{(D-60)/10\}$
x_2	=	$v + (e-v)\ /\ (1+2s)$
S	=	$1.6\ (e-v)\ /\ l^l$

and e, v, l^l, S, D, ϕ, and r are geometric parameters defined in paragraphs 3 and 4 below.

2. The above equation applies to all roundabouts except those at grade separated interchanges. For the latter there are differences of operation (see Ref 17). The "F" term in the above equation becomes "1.11F" and the "f_c" term becomes "$1.4f_c$". These differences are incorporated in the ARCADY program.

3. The ranges of the geometric parameters in the data base were as follows (but see paragraph 6.5.1) for their recommended limits in new design):-

e	entry width	3.6 - 16.5m
v	approach half width	1.9 - 12.5m
l^l	average effective flare length	1 - ∞ (m)
S	sharpness of flare	0.0 - 2.9
D	inscribed circle diameter	13.5 - 171.6m
Φ	entry angle	0.0 - 77 (degrees)
r	entry radius	3.4 - ∞ (m)

4. The geometric parameters are defined as follows:-

a) The entry width, e, is measured from the point A along the normal to the nearside kerb, see Fig 1.

b) The approach half-width, v, is measured at a point in the approach upstream from any entry flare, from the median line (or offside edge of carriageway on dual carriageways) to the nearside kerb, along a normal, see Fig 1.

c) The average effective flare length, l', is found as shown in Fig 2. The line GF'D is the projection of the nearside kerb from the approach towards the "give way" line, parallel to the median HA and at a distance of v from it. BA is the line along which e' is measured (and is therefore normal to GBJ), and thus D is at a distance of (e-v) from B. The line CF' is parallel to BG (the nearside kerb) and at a distance of (e-v)/2 from it. Usually the line CF' is therefore curved and its length is measured along the curve to obtain l'.

d) The sharpness of flare, S, is defined by the relationship

$$S = 1.6\,(e-v) / l^{l}$$

and is a measure of the rate at which extra width is developed in the entry flare. Large values of S correspond to short severe flares and small values to long gradual flares.

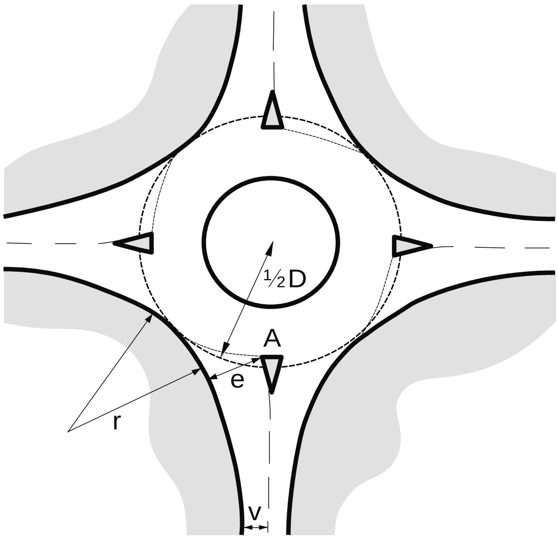
e) The inscribed circle diameter, D, is the diameter of the largest circle that can be inscribed within the junction outline, see Fig 1. In cases where the outline is asymmetric, the local value in the region of the entry is taken.
The extreme case arises for a double roundabout at a "scissors" crossroads. Fig 3 illustrates the determination of D in such cases.

f) The entry angle,ϕ, serves as a geometric proxy for the conflict angle between entering and circulating streams.
For roundabouts, having more than a distance of about 30m between the offsides of an entry and the next exit the construction is illustrated in Figs 4 and 5. Fig 4 refers to roundabouts where the circulator carriageway between an entry and the next exit is approximately straight. AD is parallel to the straight circulatory carriageway where A is as in Fig 1 and D is the point nearest to A on the median island (or marking) of the following entry. Fig 5 shows the equivalent construction for roundabouts with curved circulatory carriageways (or those where the line AD in Fig 4 is clearly not parallel to the circulatory carriageway) A'D' replaces AD as the line parallel to the circulatory carriageway. In both cases the line BC is a tangent to the line EF, which is midway between the nearside kerb line and the median line or the edge of any median island on the offside, where this line intersects the "give way" line. ϕ is measured as the acute angle between the lines BC and AD in Fig 4, and as the acute angle between BC and the tangent to A'D' at the point of intersection between BC and A'D' shown in Fig 5.
For all other roundabouts the construction is shown in Fig 6. The line BC is the same as in Figs 4 and 5. The line GH is the tangent to the line JK, which is in the following exit midway between the nearside kerb and the median line or the edge of any median island on the offside, where this line intersects the outer edge of the circulatory carriageway. BC and GH intersect at L. ϕ is then defined by:-

$$\phi = 90 - (\text{ angle GLB }) / 2$$

when the right hand side of the equation is positive. When the right hand side of the equation is zero or negative, $\phi = 0$.
Angle GLB is measured on the "outside" of the roundabout, that is, on the side facing away from the central island.

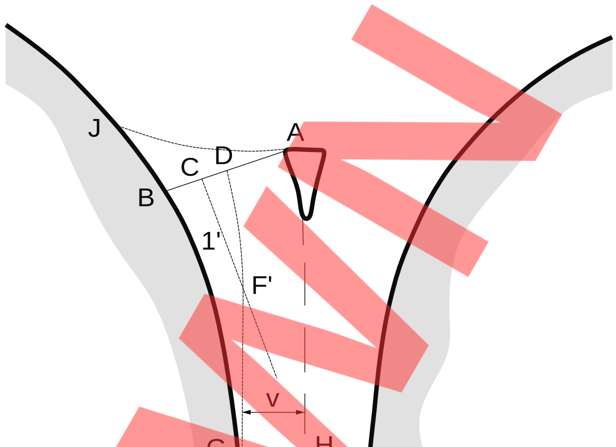
g) The entry radius, r, is measured as the minimum radius of curvature of the nearside kerb line at entry, see Fig 1. For some designs the arc of minimum radius may extend into the following exit, but this is not important provided that a half or more of the arc length is within the entry region.



A Point of maximum entry deflection at right hand end of give-way line

e Entry width
v Approach half width
r Entry Radius
D Inscribed circle Diameter

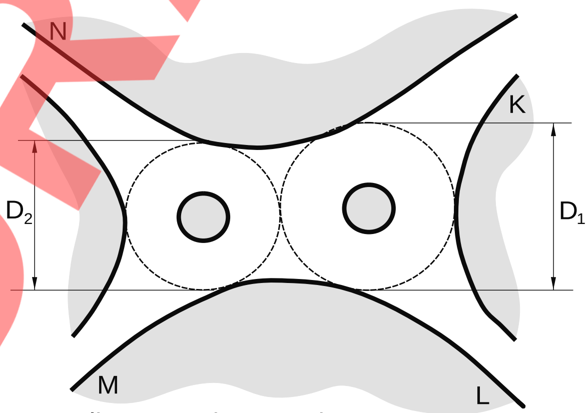
Geometric Design Features
Fig 1



1' Average effective flare length

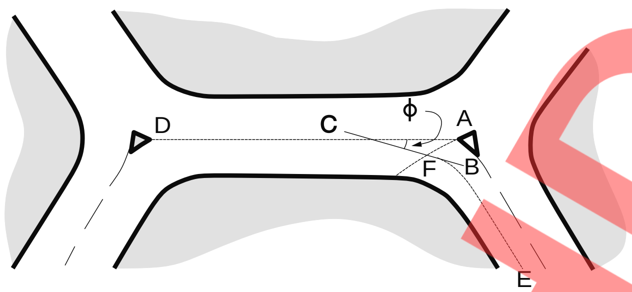
Average Effective Flare Length

Fig 2



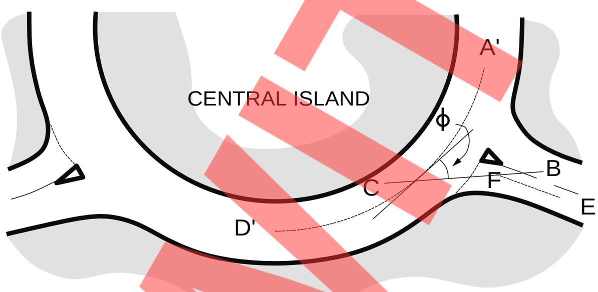
D_1 applies to entries K and L
D_2 applies to entries M and N

Fig 3



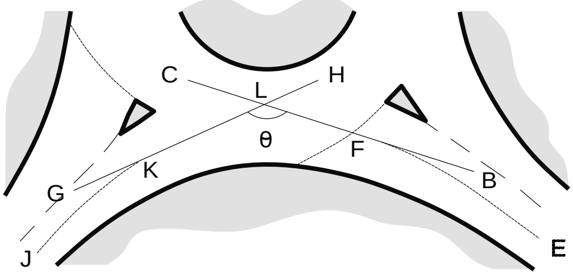
ϕ Entry Angle

Fig 4



ϕ Entry Angle

Fig 5



Entry angle ϕ defined as (90^o - θ/2)

Fig 6

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PREDICTION OF TURNING STREAM CAPACITIES

1. The best predictive equations for turning stream capacities (for major roads up to 85 kph approach speeds) found by research to date are, with reference to Fig 7:-

$$\frac{S}{q_{b-a}} = D\left(627 + 14W_{cr} - Y\left[0.364q_{a-c} + 0.114q_{a-b} + 0.229q_{c-a} + 0.520q_{c-b}\right]\right)$$

$$\frac{S}{q_{b-c}} = E\left(745 - Y\left[0.364q_{a-c} + 0.144q_{a-b}\right]\right)$$

$$\frac{S}{q_{c-b}} = F\left(745 - 0.364Y\left[q_{a-c} + q_{a-b}\right]\right)$$

$$where Y = (1 - 0.0345W)$$

In each of these equations the geometric parameters represented by D,E and F are stream-specific:-

$$D = \left[1 + 0.094\left(w_{b-a} - 3.65\right)\right]\left[1 + 0.0009\left(V_{rb-a} - 120\right)\right]\left[1 + 0.0006\left(V_{lb-a} - 150\right)\right]$$

$$E = \left[1 + 0.094\left(w_{b-c} - 3.65\right)\right]\left[1 + 0.0009\left(V_{rb-c} - 120\right)\right]$$

$$F = \left[1 + 0.094\left(w_{c-b} - 3.65\right)\right]\left[1 + 0.0009\left(V_{rc-b} - 120\right)\right]$$

Where wb-a denotes the lane width available to waiting vehicles in the stream B-A, and Vrb-a the corresponding visibilities, and so on. In all cases capacities and flows are in pcu/hour (1 HGV = 2.0 pcu) and distances in metres. If the right hand side of any equation is negative, the capacity is zero.

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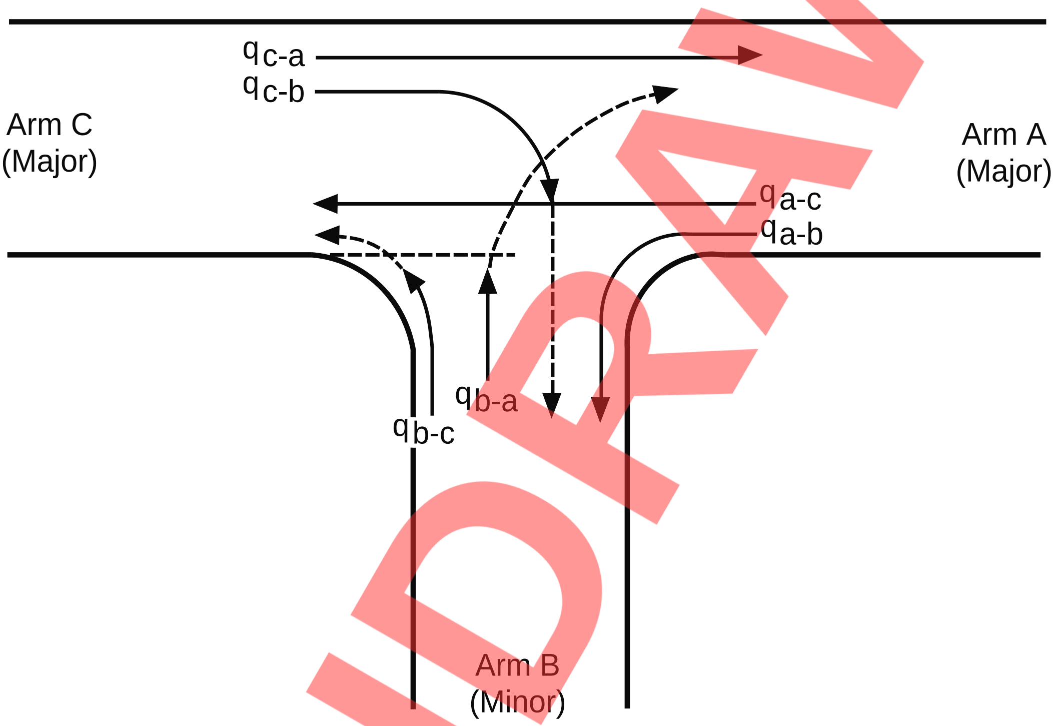

Notes
$q_{c\text{-}a}$ = the flow of vehicles for the stream c-a
$q_{b\text{-}a}$ = the flow of vehicles for the stream b-a
and so on
Superscript ^{s} (eg $q^s_{b\text{-}a}$) denotes the flow from a saturated stream, ie one in which there is stable queueing

2. Visibilities are measured in accordance with paragraph 8.2.2 of Ref 5. W, Wcr and w are measured as follows:-

Defintion of geometric parameters

(i) Lane width for non-priority streams, w (m)

Where there are clear lane markings the width is measured directly. The average of measurements taken at 5m intervals over a distance of 20m upstream from the give-way point is used. Any measurement excedding 5m is reduced to 5m before the average is taken. Where lane markings are unclear (or absent), Diagrams (a), (b), and (c) are used, and the lane width calculated according to:

$$w = (a + b + c + d + e) / 5 \text{ metres}$$

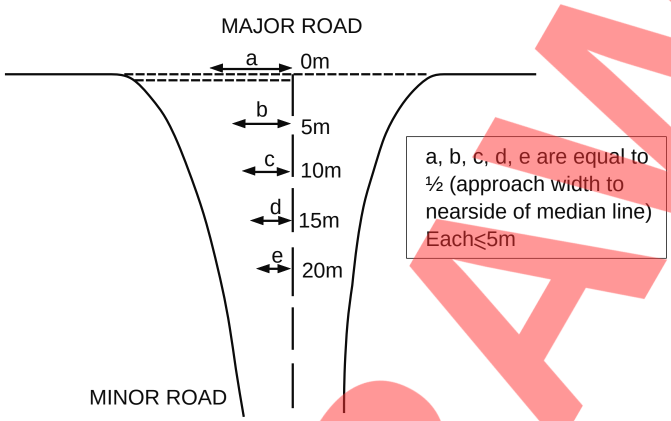

Diagram (a) Lane Width measurements for the right-turning minor road stream

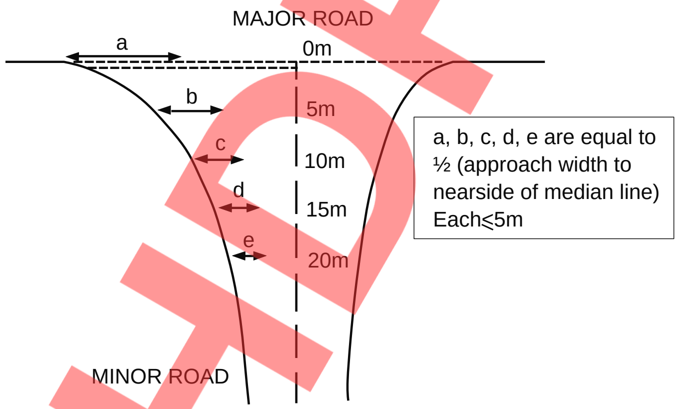

Diagram (b) Lane width measurements for the left-turning minor road stream

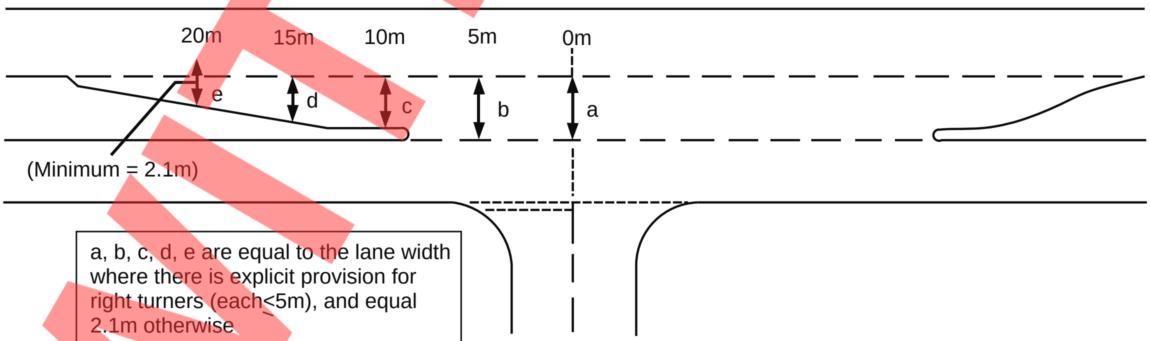

Diagram (c) Lane width measurements for the right-turning major road stream

(ii) Major road width, W (m) and its components

The four parts of diagram (d) show the main components of major road width. They are combined to give:

(1) the ‘nearside’ width : W_n

$$W_n = ½ (W2 + W4)$$

(2) the ‘farside’ width : W_f

$$W_f = ½ (W1 + W3)$$

(3) the total carriageway width : W

$$W = W_n + W_f$$

(4) (at dual carriageway sites with kerbed central reserve)
the width of central reserve: W_{CR}

$$W_{CR} = ½ (W5 + W6)$$

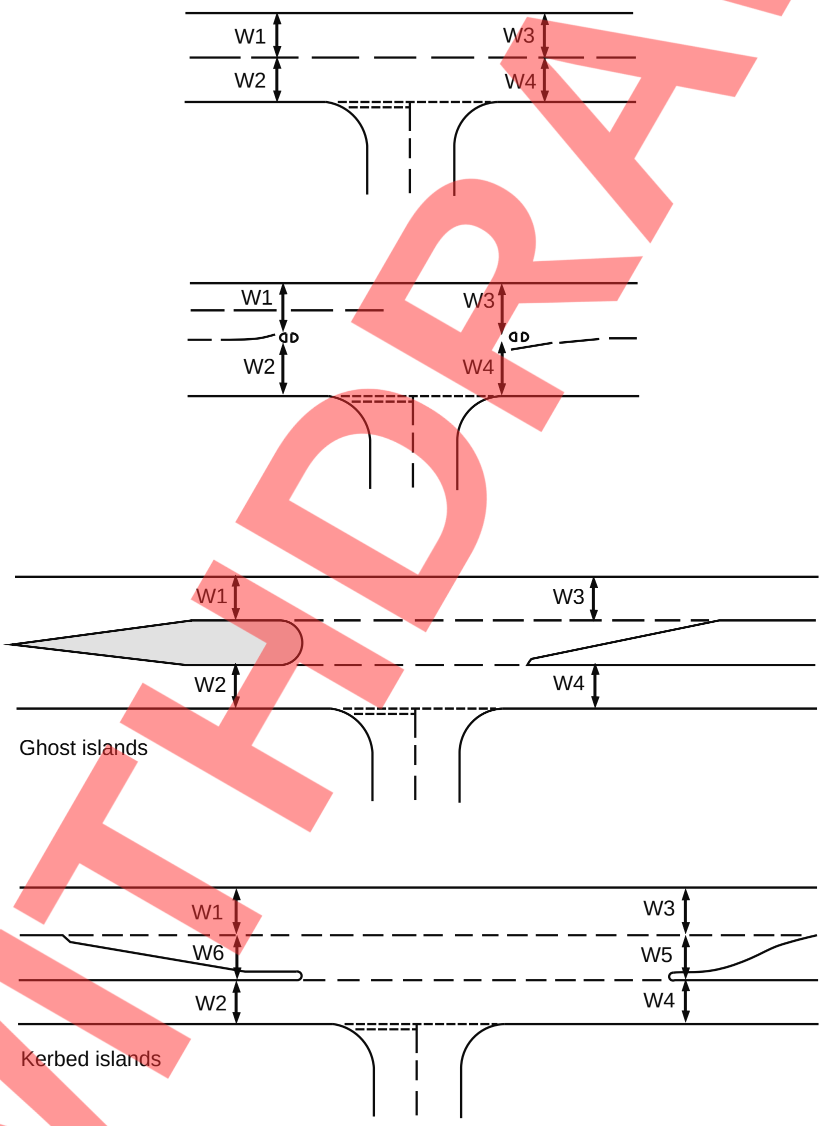

Diagram D Components of major road width

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3. The ranges of the geometric parameters in the data base were:- (all in metres)

w : 2.05 - 4.70

Vr: 17.0 - 250.0

Vl: 22.0 - 250.0

Wcr: 1.2 - 9.0 (dual carriageway sites only)

W: 6.4 - 20.0

The maximum values of visibility and Wcr used in calculation and computation should be 250 and 10m respectively, even if greater values are physically provided.

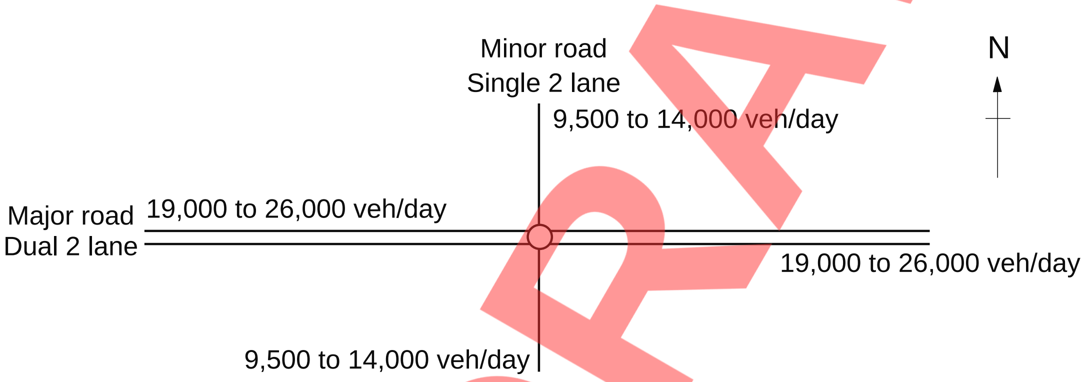
EXAMPLES

EXAMPLE 1

Roundabout Alternatives

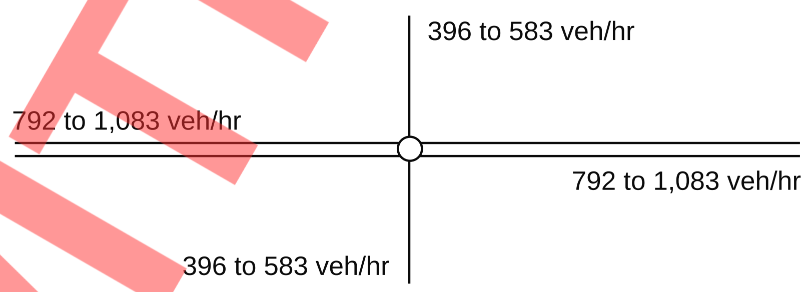
1 It has been decided to construct a roundabout at the junction between a DSAP road and a S2 road on the fringe of an urban area in character with very low seasonal variation.

2. The traffic information available from the traffic model, this being an urban scheme, is the expected normal high growth and low growth 2-way 24 hour Annual Average Weekday Traffic (AWWT) flows on each road for the year 1966 (a year about 12 years after the expected opening date):-



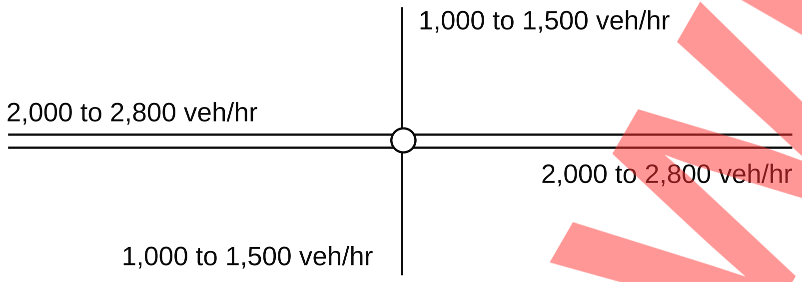
24 hour AAWT 2-way flows.

3. From the AAWT 2-way flows the Annual Average Weekday Hourly Traffic (AAWHT) 2-way flows on the approach roads in 1966 are calculated. AAWHT = AAWT/24, for example, 19000/24 = 792, 26000/24 = 1083, etc.



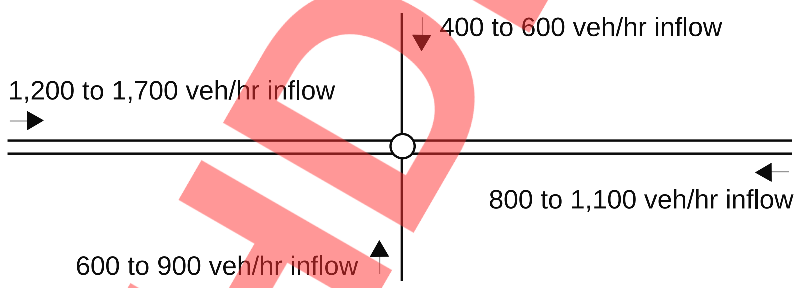
AAWHT 2-way flows.

4. As there is very low seasonal variation it has been decided in this particular case to use the estimate of the 30th highest hour in 1966 to obtain the 2-way flows on approach roads in the design peak hour. Thus AAWHT is factored by 2.547 (see TAM appendix D14, Table 5A for Main Urban Roads). For example, 792 x 2.547 = 2017, say 2000, etc.



Design peak hour 2-way flows.

5. To obtain the directional flows (i.e., the range of entry flows into the junction) from the design peak hour 2-way flows on the approach roads it has been decided in this case to assume a 60/40 split with the entry flows from the west and south dominant. For example, 2000 x 0.6 = 1200; 2800 x 0.6 = 1680, say 1700; 2000 x 0.4 = 800; etc



Directional flows.

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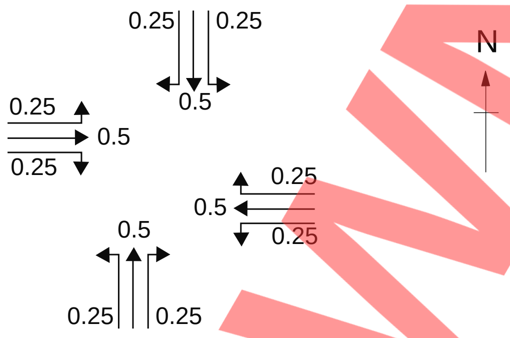
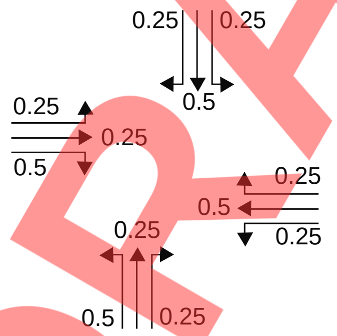
6 Turning movements to and from the south are expected to dominate, and the following three patterns are expected to reflect the range of possibilities in the design peak hour.

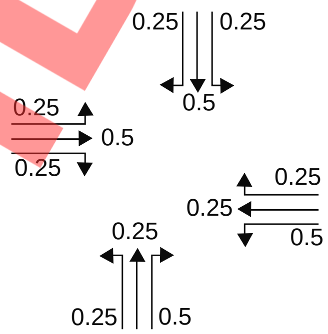
TM1



TM2



TM3



Directional flows when adjusted using turning proportions are termed "Reference Flows".

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7 The main design parameters for capacity are entry width, e, and flare length, l^1. An initial examination of the possibilities indicates the following ranges in this case:

ARM	e (m)	l^1 (m)
SOUTH	3.65 - 12	About 10 - 50
WEST	7.3 - 15	About 10 - 50
NORTH	3.65 - 12	About 10 - 50
EAST	7.3 - 15	About 10 - 50

A preliminary screening indicates the most plausible alternatives as a 63m ICD and a 70m ICD roundabout with the following parameters (see Appendix 1 for definitions):- (Note the need for vehicle path deflection since the approach roads are derestricted).

63mm ICD

ARM	v (m)	e (m)	l^1 (m)	R (m)	D (m)	ϕ (degrees)
SOUTH	3.65	7.30	25.0	20.0	63.0	30.0
WEST	7.30	10.50	25.0	20.0	63.0	30.0
NORTH	3.65	7.30	25.0	20.0	63.0	30.0
EAST	7.30	10.50	25.0	20.0	63.0	30.0

70m ICD

ARM	v (m)	e (m)	l^1 (m)	R (m)	D (m)	ϕ (degrees)
SOUTH	3.65	10.50	25.0	20.0	70.0	30.0
WEST	7.30	13.0	25.0	20.0	70.0	30.0
NORTH	3.65	10.5	25.0	20.0	70.0	30.0
EAST	7.30	13.0	25.0	20.0	70.0	30.0

8 The trial layouts are assessed for peak hour performance over the range of Reference Flows using the ARCADY program. The results shown overleaf indicate maximum RFC ratios, queue lengths and delays that can be expected. At high growth RFC ratios for the 63m ICD roundabout reach 97-98%, while the corresponding figures for the 70m ICD roundabout are 85%.

Traffic being appraised	Range of Reference flows (v.p.h.)		Checks on R.F.C. ratios, queue lengths and delays for trial designs: 63m ICD 3 lane entry D2 2 lane entry S2	Checks on R.F.C. ratios, queue lengths and delays for trial designs: 70m ICD 4 lane entry D2 3 lane entry S2
LOW GROWTH	1,200 → 400 ↓ 800 ← 600 ↑	TM 1	61%.2.4 → 41%.1.5 ↓ 40%.1.3 ← 51%.1.5 ↑	54%.1.3 → 32%.0.4 ↓ 35%.1.2 ← 41%.1.4 ↑
		TM 2	58%.1.3 → 41%.1.5 ↓ 45%.1.3 ← 51%.1.5 ↑	51%.1.3 → 32%.0.4 ↓ 39%.1.3 ← 41%.1.4 ↑
		TM 3	61%.2.4 → 45%.1.6 ↓ 40%.1.3 ← 47%.1.4 ↑	54%.1.3 → 34%.1.4 ↓ 35%.1.2 ← 38%.1.3 ↑
HIGH GROWTH	1,700 → 600 ↓ 1,100 ← 900 ↑	TM 1	87%.15.27 → 81%.4.19 ↓ 62%.2.14 ← 89%.7.22 ↑	85%.5.9 → 60%.1.8 ↓ 55%.1.3 ← 70%.2.8 ↑
		TM 2	89%.7.12 → 82%.4.20 ↓ 76%.3.8 ← 89%.6.22 ↑	78%.3.6 → 60%.1.8 ↓ 65%.2.5 ← 70%.2.8 ↑
		TM 3	97%.15.27 → 90%.10.57 ↓ 52%.2.4 ← 77%.3.11 ↑	85%.5.9 → 69%.2.11 ↓ 55%.1.3 ← 62%.2.6 ↑
			KEY: 74%.3.21 means maximum R.F.C. ratio 74% maximum queue length 3 vehicles, maximum delay per vehicle 21 seconds.	

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9 The cost of traffic delays over the scheme life is evaluated for the two options at high growth and low growth using COBA 9. The turning movements are necessarily modified to achieve balanced link flows (see section 7.8 of Ref8) on a daily basis while continuing to reflect the bias to and from the south. The 63m ICD roundabout is estimated to cost £63m ICD roundabout is estimated to cost £68,000 and the 70m ICD to cost £92,000, at 1979 prices . (These costs may appear low. They are extra over the basic link costs through the junction). The COBA 9 results are as follows. (All costs are discounted costs in thousands of pounds).

First Scheme Year 1985

Traffic Figures 1996

	Construction Costs	Delay Costs Low	Delay Costs High
63 ICD	48	876	1485
70 ICD	66	851	1388
	18	25	97

Therefore Incremental NPV is going from 63m ICD to 70m ICD is:-

Low growth +7

High growth +79

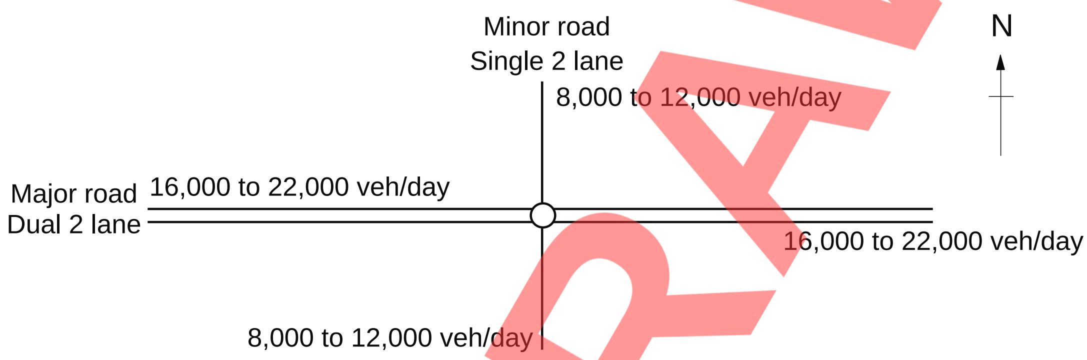
10 Thus it is likely in this case that the 70m ICD roundabout would be chosen as it shows a much more acceptable maximum RFC ratio (ie 85%) than the smaller design at high growth and a good incremental NPV at high growth, for very modest increase in size. However, if high growth was not likely to occur, of if the scheme involving the larger roundabout involved (say) high statutory undertakers' costs, the 63m ICD roundabout would become more attractive.

EXAMPLE 2

Roundabout Alternatives

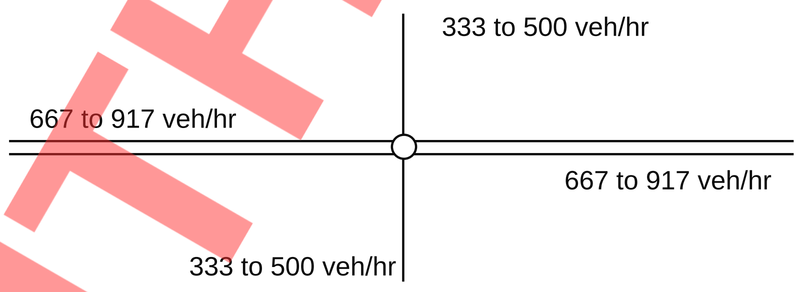
1 It has been decided to construct a roundabout at the junction between a D2AP road and a S2 road on a recreational inter-urban route. Traffic flows vary greatly throughout the year with exceptionally high flows at weekends during the summer months. The surrounding network is also very congested during these summer weekends, but it is considered unrealistic to cater specifically for these exceptional occurrences at this site.

2 The traffic information available from the traffic model is the expected normal high growth and low growth 2-way 24 hour Annual Average Daily Traffic (AADT) flows on each road for the year 2001 (about 13 years after the expected opening date):-



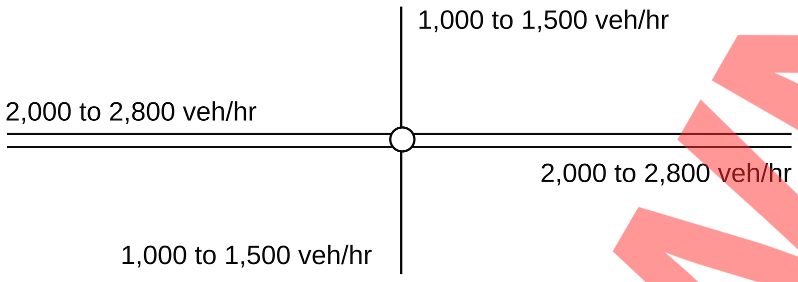
24 hour AADT 2-way flows.

3 From the AADT 2-way flows the Annual Average Hourly Traffic (AAHT) 2-way flows on the approach roads in 2001 are calculated. AAHT = AADT/24, for example, 16000/24 = 667, 22000/24 = 917, etc.



AAHT 2-way flows.

4 As there is a very high seasonal variation, and it is considered unrealistic to cater specifically for exceptional periods, it has been decided in this particular case to use the estimate of the 200th highest hour in 2001 to obtain the 2-way flows on approach roads in the design peak hour. Thus AAHT is factored by 3.024 (See TAM Appendix D14, Table 5A for Recreational Inter-Urban roads). For example, 667 x 3.024 = 2017, say 2000; 917 x 3.024 = 2773, say 2800; etc.



Design peak hour 2-way flows.

5 To avoid repetition the calculation of Reference flows, and trial layouts are assumed to be identical to those in Example 1.

6 The cost of traffic delays over the scheme life is evaluated for the two options at high growth and low growth using COBA 9. The turning movements are modified to achieve balanced link flows on a daily basis (see section 7.8 of Ref 8), while continuing to reflect the bias to and from the south. The 63m ICD roundabout is estimated to cost £68,000 and the 70m ICD roundabout to cost £92,000, at 1979 prices. (These costs may appear low. They are extra over the basic link costs through the junction). The COBA 9 results are as follows:- (all discounted costs in thousands of pounds).

	First Scheme Year	1989	
	Traffic Figures	2001	
	Construction Costs	Delay Costs	
		Low	High
63 ICD	37	617	1253
70 ICD	50	598	1027
	13	19	226

Therefore Incremental NVP is going from 63m ICD to 70m ICD is:-

Low Growth +6

High Growth +213

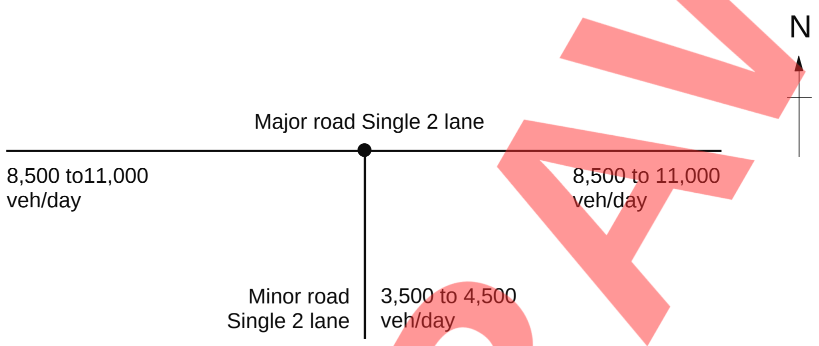
7 Thus it is extremely probable that in this case that the 70m ICD roundabout would be chosen, as it has extremely good incremental NPV above low growth for very modest increase in size and an acceptable maximum RFC ratio of 85% at high growth. Also the slight extra land take is less of a problem in rural areas.

EXAMPLE 3

Major/Minor Junction Alternatives

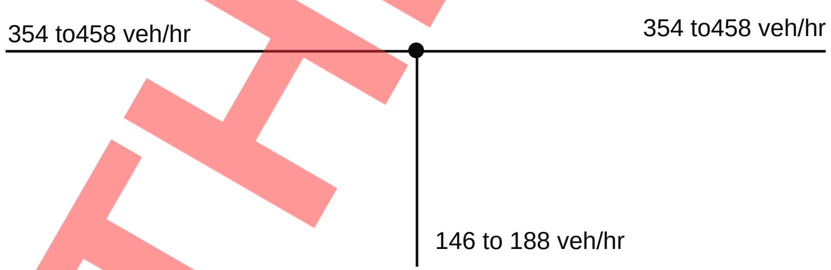
1. It has been decided to construct a major/minor junction at the T-junction between two S2 roads. The major road is an inter-urban road (design speed 100 kph) which is expected to have a typically inter-urban seasonal variation pattern.

2. The traffic information available from the traffic model is the expected normal high growth and low growth 2-way 24 hour AADT flows on the each road for the year 2001 (about 15 years after the expected opening date):-



24 hour AADT 2-way flows

3. From the AADT 2-way flows the AAHT 2-way flows on the approach roads in 2001 are calculated. AAHT = AADT/24. For example, 8500/24 = 354; 11000/24 = 48;etc.



AAHT 2-way flows

4. In view of the free flowing nature of the contiguous network, it has been decided in this particular case to use the estimate of the 50th highest hour in 2001 to obtain the 2-way flows on approach roads in the design peak hour. Thus AAHT is factored by 2.891 (See TAM Appendix D14, Table 5A for Inter-urban roads). For example, 354 x 2.891 = 1023, say 1000; 458 x 2.891 = 1324, say 1300; etc.

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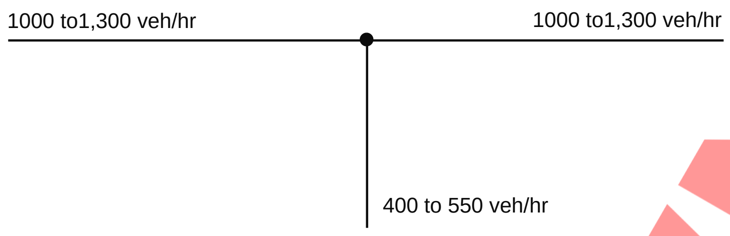
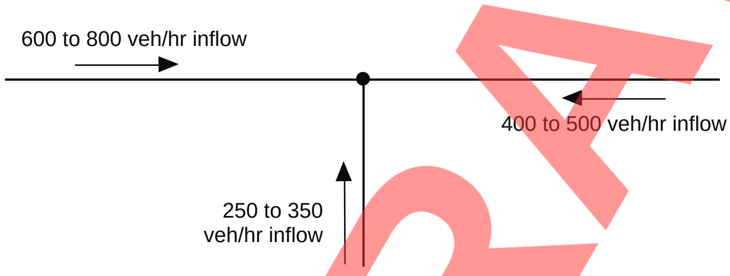


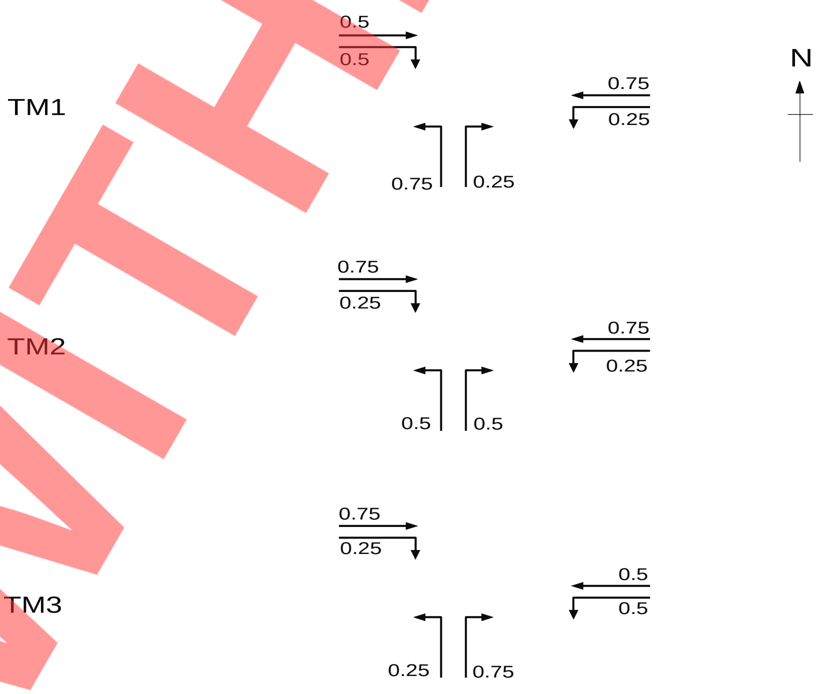
Design Peak Hour 2-way flows

5. To obtain the directional flows (i.e. the range of the entry flows into the junction) from the design peak hour 2-way flows on the approach roads it has been decided in this case to assume a 60/40 split with the entry flows from the west and south dominant, For example, 1000 x 0.6 = 600; 1300 x 0.6 = 780, say 800; 1000 x 0.4 = 400; etc.



Directional flows

6. The dominant turning movements are not known, so the following three patterns will be assessed as they should reflect the range of possibilities in the design peak hour.



Directional flows when adjusted using turning proportions are termed "Reference Flows".

7. From the Reference Flows it appears that there are two feasible alternative layouts:- a 3.5m wide ghost island and single lane dualling, both with two entry lanes on the minor road. The geometric parameters are as follows:- (all dimensions in metres, see Appendix 2)

3.5m Ghost Island

W = 6.00

Wcr = 0.00

Wcr = 3.50

Vrc - b = 250.0

Vr = 225.0, minor road

Vl = 225.0, minor road

wb - c = 4.25

wc - a = 4.25

Single Lane Dualling

W = 8.00

Wcr = 10.00

Wc - b = 4.50

Vr - b = 250.0

Vr = 225.0, minor road

Vl = 225.0, minor road

wb - c = wb - a = 4.25

8. The trial layouts are assessed for their peak hour performance over the range of Reference Flows using the PICADY program. The results are shown following and indicate maximum RFC ratios, queue lengths and delays that can be expected. The maximum RFC ratio is 89% on the right turn from the minor road for the 3.5m ghost island at high growth for TM3.

Traffic being appraised	Range of Reference flows (v.p.h.)		Checks on R.F.C. ratios, queue lengths and delays for trial designs	
			3.5m Ghost island 2 lane minor road entry	Single Lane Dualling 2 lane minor road entry
LOW GROWTH	600 400 250	TM 1	53%.1.12 32%.0.8 20%.0.12	47%.1.9 31%.0.8 13%.0.7
		TM 2	26%.0.8 22%.0.7 35%.1.13	24%.0.7 21%.0.7 24%.0.8
		TM 3	26%.0.8 11%.0.6 50%.1.16	24%.0.7 11%.0.6 35%.1.9
HIGH GROWTH	800 500 350	TM 1	74%.3.21 50%.1.11 42%.1.24	67%.2.15 46%.1.10 23%.0.10
		TM 2	37%.1.9 35%.1.10 65%.2.30	33%.0.8 32%.0.9 40%.1.12
		TM 3	37%.1.9 18%.0.8 89%.5.61	33%.0.8 15%.0.7 57%.1.15
			KEY:- 74%.3.21 means maximum R.F.C. ratio 74%, maximum queue length 3 vehicles, maximum delay per vehicle 21 seconds.	

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9. The cost of traffic delays over the scheme life evaluated for the two options at high growth and low growth using COBA 9. The turning movements are modified to achieve balanced link flows on a daily basis (see section 7.8 of Ref 8), but there are still three distinct cases of equal left and right turns from the minor road, a predominant left turn from the minor road (75/25 split) and a predominant right turn from the minor road (25/75 split). The 3.5m ghost island is estimated to cost £39,000, at 1979 prices. The COBA 9 results are as follows:- (all discounted costs in thousands of pounds).

First Scheme Year	1987
Traffic Figures	2001

Equal minor road turning movements

	Construction Costs	Delay Costs Low	Delay Costs High
3.5m Ghost Island	6	90	136
Single Line Dualling	24	86	128
	18	4	8

Incremental NPV in going from ghost island to single lane dualling is:-

Low growth	-14
High growth	-10

Predominant left turn from minor road

	Construction Costs	Delay Costs Low	Delay Costs High
GI	6	81	121
SLD	24	79	118
	18	2	3

Incremental NPV:-

Low Growth -16

High Growth -13

Predominant right turn from minor road

	Construction Costs	Delay Costs Low	Delay Costs High
GI	6	102	163
SLD	24	95	144
	18	7	19

Incremental NPV:-

Low growth -11

High Growth +1

10. Having examined the results it can be seen that the only case of the suggested maximum RFC ratio of 75% being exceeded is the right turn out of the major road for the ghost island at high growth when there is a predominant right turn from the minor (89%, 5, 61). However, since the major road is 100kph design speed, RFC values exceeding 75% should not be accepted (see paragraph 7.2). Additionally the differences in NPV between the ghost island option and the single lane dualling option are only small, therefore (other things being equal) the single lane dualling option should be chosen.

In cases where single lane dualling is being considered a check must be made on the character of the major road for at least 3km on either side of the proposed junction. If stretches of full dualling or local dualling occur (or are likely to occur) then it would be prudent to adopt the ghost island instead.

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