
THE COBA 2021 USER MANUAL

PART 5

SPEEDS ON LINKS

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1. COBA ROAD CLASSES

- 1.1 Savings in journey time are generally the main source of COBA benefits following a highway improvement. The assessment of time savings requires knowledge of likely traffic flows and knowledge of the behaviour of the network under varying traffic loadings. It is the flow dependence of traffic speeds, and their variability from one occasion to another, that limits the direct use of observations of speeds on existing roads. The COBA approach is therefore to use nationally-derived two-way relationships wherever possible to predict traffic speeds.
- 1.2 Different speed prediction relationships are selected in COBA by allocating a link to a particular road class. Table 1/1 lists the COBA road classes.

Road Class	Description
1	Rural single carriageway
2	Rural all-purpose dual 2-lane carriageway
3	Rural all-purpose dual 3 or more lane carriageway
4	Motorway, dual 2-lanes
5	Motorway, dual 3-lanes
6	Motorway, dual 4 or more lanes
7	Urban, non-central
8	Urban, central
9	Small town
10	Suburban single carriageway
11	Suburban dual carriageway
12-14	User defined all-vehicle relationships
15-20	User defined light/heavy vehicle relationships

Table 1/1: COBA Road Classes

- Classes 1 to 6 are used for all-purpose roads and motorways that are generally not subject to a local speed limit. Classes 7 and 8 are used for roads in large towns or conurbations subject to 30 mph (48 kph) speed limits only. Class 9 is used in small towns or villages for routes subject to a 30 mph (48 kph) or 40 mph (64 kph) speed limit. Classes 10 and 11 are used for major suburban routes in towns and cities that are generally subject to a 40 mph (64 kph) speed limit.
- 1.3 The method of predicting speeds in COBA differs between rural roads, where there is little interaction between different links and junctions on the network, and urban and suburban roads, where delays at junctions tend to be inter-dependent. On rural roads, relationships are used to predict the speed of traffic on each link according to link geometry and traffic flow; delays at junctions are separately assessed using junction delay models. In urban areas, the road network has to be considered as a system rather than as a set of links and junctions. Accordingly COBA uses area wide urban speed relationships based on average journey speeds observed in towns and conurbations in England. For small town roads COBA models an average speed for the route.
- 1.4 The basic form of the speed/flow relationships varies between road classes. For rural, suburban and small town roads the speed of vehicles reduces as flow increases until a critical flow level is reached at which the rate of speed reduction increases until a minimum speed cut-off is reached. The relationships for urban roads have a uniform negative speed/flow slope for all flow levels above the minimum speed constraint. The other major difference is that rural and suburban relationships provide separate estimates of the average journey speeds of light vehicles and heavy vehicles, the urban and small town relationships provide a single estimate of the average vehicle speed. Light vehicles are defined as cars and light goods vehicles (LGV); heavy

goods vehicles are defined as other goods vehicles (OGV1 and OGV2), buses and coaches (PSV). The average speed of traffic as output by COBA in the journey time tables is obtained by combining the light and heavy vehicle speeds in the relevant proportions.

- 1.5 The relationships can predict speeds above the legal speed limit for the particular road class considered, if this occurs the speed of the vehicle type considered is reduced to the legal speed limit before any economic calculations are made. All relationships are subject to a minimum speed cut-off which varies by Road Class.
- 1.6 When flows reach a particular level on a link COBA produces an overcapacity report. It is a signal to the user that flows are about the highest levels that could normally be expected on a link of this standard. The levels of the 'capacity flags' (Q_C) for each road class are detailed in the following chapters and summarised in Part 5 Chapter 8 together with advice to the user regarding what action is required.

2. RURAL SINGLE CARRIAGEWAYS (ROAD CLASS 1)

2.1 The rural single carriageway speed relationships were derived from a study carried out in 1991 (Speed/flow/geometry relationships for rural single carriageways, TRRL Contractors Report No.319). None of the sections studied was subject to a local speed limit, and none included a junction where the road being studied lost priority. Observed journey speeds varied from 30 to 95 kph, and the proportion of heavy vehicles varied from 0 to 50% (both figures give the range of 10 minute averages observed).

2.2 Table 2/1 defines the geometric parameters and variables used in the relationships and gives the ranges of typical values over which the relationships should apply. The relationships cannot necessarily be taken to apply outside the given ranges of the variables. The geometric variables should all be averaged over a reasonable length of link, generally not less than two kilometres.

2.3 The actual width of surfaced road is defined by two parameters. The first (CWID) being the width of carriageway between any continuous white lines which may or may not be delineating a hard strip. The second (SWID) is the total width of any continuous edge line and hard strip. From examination of the speed prediction formula given in para 2.10 it can be seen that the presence of a continuous edge line increases average light vehicle speeds by 1.6 kph and the provision of a one metre hard strip by a further 1.1 kph.

2.4 Figure 2/1 shows how measurements of bendiness, hilliness and visibility are taken. Hilliness is $(H_R + H_F)$ and net gradient NG is $(H_R - H_F)$. On two-way links in COBA, net gradient is always zero because the two directions of flow are not disaggregated. On one-way links, rises and falls are defined with respect to the direction of traffic flow; in general they will not cancel out. The absolute value of net gradient should not exceed the total hilliness.

2.5 The average sight distance VISI is the harmonic mean of individual observations, such that:

$$VISI = n / (1/X_1 + 1/X_2 + \dots + 1/X_n),$$

where n = number of observations, and Xi = sight distance at point i. The harmonic mean is used because this reduces the effect on VISI of a few measurements of very high visibility.

2.6 For proposed new roads, VISI should normally be calculated from engineering drawings. Measurements of sight distance should be taken in both directions at regular intervals (50 metres for sites of uneven visibility, 100 metres for sites with good visibility) measured from an eye height of 1.05 metres to an object height of 1.05 metres, both points being above the road surface at the centre line of the carriageway. Sight distance should be the true sight distance available at any location, including any sight distance available across verges, and outside of the highway boundary wherever sight distance is available across embankment slopes or adjoining land. On the approach to a junction where the link being considered loses priority it is unnecessary to make further measurements once the junction is within the sight distance.

2.7 For existing roads, an empirical relationship has been derived which provides estimates of VISI given bendiness and edge details:

$$\text{Log VISI} = 2.46 + (VWID + SWID)/25 - \text{BEND}/400.$$

This relationship should normally be used for all existing roads for which bendiness and verge width have been measured. Note that the relationship is non-directional (that is it will give the same value of VISI for both directions of travel). On long straight roads or where sight distance is available outside the highway boundary, the formula will often significantly underestimate VISI. VISI should then be set to 700 metres for roads with high visibility; otherwise estimates should be made from plans or site measurements.

SYMBOL	VARIABLE DESCRIPTION	TYPICAL VALUES		
		Min		Max
DES	Is road designed to TD9 (DMRB 6.1.1.) Standards?	Yes	or	No
BEND	Bendiness; total change of direction (deg/km)	0		150
HILLS	Hilliness; total rise and fall (m/km)	0		45
NG	Net gradient one-way links only (m/km)	-45		45
JUNC	Side roads intersecting, both directions (no/km)	0		5
CWID	Average carriageway width between white line edge markings, excluding any painted out portion (m)	6		11
SWID	Average width of hard strip on both sides, including width of white line (m)	0		1.0
VWID	Average verge width, both sides (m)	0		7
VISI	Average sight distance (m)	100		550
PHV	Percentage of heavy vehicles (OGV1 + OGV2 + PSV)	2		30
V _L , V _H	Speed of light and heavy vehicles (kph)	45		90
S _L , S _H	Speed/flow slope of light and heavy vehicles (kph reduction per 1000 increase in Q)	5		50
Q	Flow all vehicles (vehs/hour/dir)			
Q _B	Breakpoint: the value of Q at which the speed/flow slope of light vehicles changes (vehs/hour/dir)		0.8 Q _C	
Q _C	Capacity flag: defined as the maximum realistic value of Q (vehs/hour/dir)	900		1600

Table 2/1: Definition of Variables Used in Speed Prediction Formulae for Rural Single Carriageways

2.8 The capacity flag of a single carriageway is set at:

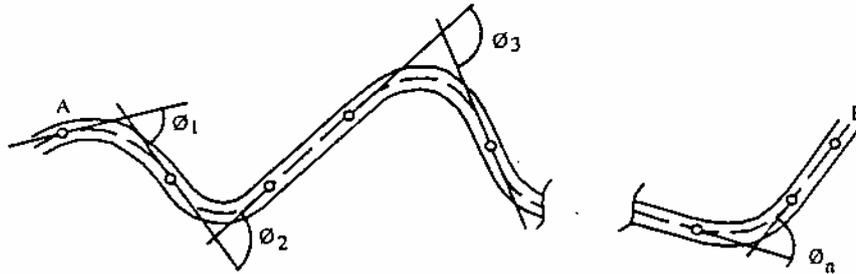
$$Q_C = \frac{2400(CWID - 3.65)}{CWID} \times \frac{(92 - PHV)}{80} \text{ vehicles/hour/dir.}$$

It therefore varies by flow group as the proportion of heavy vehicles change. When calculating the capacity flag CWID has a minimum value of 5.5 metres.

1) BENDINESS

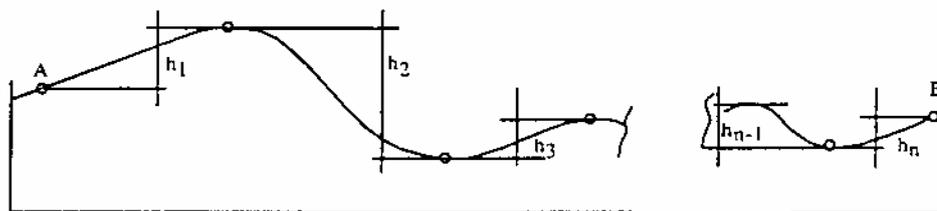
Plan

$$B. \text{ average bendiness of section AB} = \frac{\theta_1 + \theta_2 + \theta_3 + \dots + \theta_n \text{ (degrees)}}{\text{Distance AB (kms)}}$$



2) HILLINESS

Longitudinal profile



$$H_R \text{ Total Rises (} H_R \text{) of section AB} = \frac{h_1 + h_3 + \dots + h_n \text{ (metres)}}{\text{Distance AB (kms)}}$$

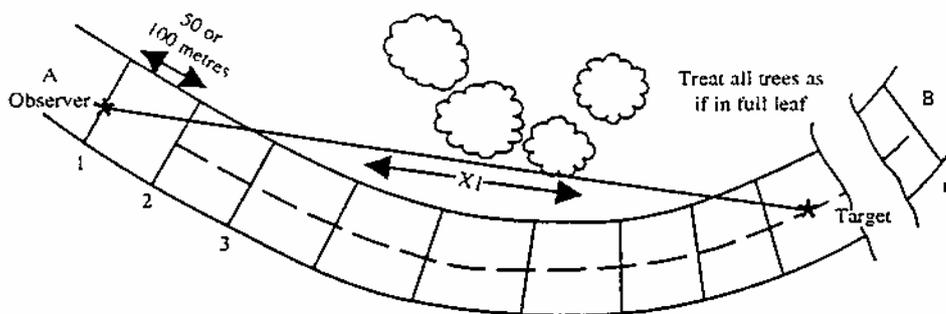
$$H \text{ average hilliness} = H_R + H_F$$

$$H_F \text{ Total Falls (} H_F \text{) of section AB} = \frac{h_2 + \dots + h_{n-1} \text{ (metres)}}{\text{Distance AB (kms)}}$$

$$N. \text{ net gradient} = H_R - H_F$$

3) VISIBILITY

Plan



Observer and Target with markers at 1.05m (standard driver height)

$$VISI \text{ average sight distance of section AB} = n / \left(\frac{1}{X1} + \frac{1}{X2} + \dots + \frac{1}{Xn} \right) \text{ (metres)}$$

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Figure 2/1: Measurement of Road Geometry on Rural Roads

2.9 The point of change of slope (Q_B) is given by the relationship:

$$Q_B = 0.8 Q_C.$$

2.10 For flow levels less than the breakpoint Q_B the speed prediction formulae for light vehicles in kph is:

$$V_L = 72.1 - 0.09 \times \text{BEND or } -0.015 \times \text{BEND for roads designed to TD9} \\ - 0.0007 \times \text{BEND} \times \text{HILLS} \\ - 0.11 \times \text{NG (one-way links only)} \\ - 1.9 \times \text{JUNC} \\ + 2.0 \times \text{CWID} \\ + \text{SWID} (1.6/\text{SWID} + 1.1) \\ + 0.3 \times \text{VWID} \\ + 0.005 \times \text{VISI} \\ - (0.015 + (0.00027 \times \text{PHV})) \times Q.$$

NOTE: for two-way links HILLS as entered by the COBA user is $H_R + H_F$; H_R is defined as the sum of the rises per unit distance and H_F is defined as the sum of the falls per unit distance (see Paragraph 2.4). For one-way links H_R is coded in the HILLS field and H_F in the DOWN field. H_F is used in the calculation of NG and in the calculation of vehicle operating costs.

2.11 For flow values greater than Q_B the speed prediction formula for light vehicles is:

$$V_L = V_B - 0.05 \times (Q - Q_B),$$

where V_B = speed at $Q = Q_B$.

2.12 For all flow levels the speed prediction formula for heavy vehicles in kph is:

$$V_H = 78.2 - 0.1 \times \text{BEND or ZERO for roads designed to TD9} \\ - 0.07 \times \text{HILLS} \\ - 0.13 \times \text{NG (one-way links only)} \\ - 1.1 \times \text{JUNC} \\ + 0.007 \times \text{VISI} \\ + 0.3 \times \text{VWID} \\ - 0.0052 \times Q,$$

subject to the constraint that if the calculated value of V_H is greater than V_L then V_H is set equal to V_L .

2.13 The minimum speed cut-off for rural single carriageways is 45 kph.

2.14 The speed prediction formulae may be used to estimate speeds in the two directions of travel separately (the difference arising entirely from the values of NG and, where sight distances are measured in each direction, VISI). In COBA this entails using pairs of one-way links (see below). However it is usually sufficient to treat both directions together, in which case net gradient is always zero.

Climbing Lanes on Single Carriageway Roads

- 2.15 For the detailed assessment of climbing lanes, the road should be treated by modelling the two directions of flow as separate one-way links. The traffic flow Q to be entered for each one-way link is half the two-way traffic flow. The width of the one-way links will be 6.6 metres (say) for the two lanes uphill and 3.4 metres for the single lane downhill. For the calculation of speeds on links COBA assumes that carriageway width CWID is twice the width of a one-way link. The effect of the climbing lane is to increase speeds uphill (due to increased carriageway width, reduced flow per lane, reduced access friction and possibly increased visibility) while speeds on the downhill link remain unaltered. For coding purposes the length of the climbing lane should be taken as the length of road over which the full width of the climbing lane is provided plus half the taper lengths.

3. RURAL ALL-PURPOSE DUAL CARRIAGEWAYS AND MOTORWAYS (ROAD CLASSES 2-6)

- 3.1 The speed relationships for rural all-purpose dual carriageways and motorways were derived from a study carried out in 1990 (Speed/flow/geometry relationships for rural dual carriageways and motorways, TRRL Contractors Report No 279), individual site lengths varied from 1.5 to 5 kilometres. None of the sections studied was subject to a local speed limit, and none included a junction where the road being studied lost priority. Observed journey speeds varied from 40 to 125 kph over flow ranges of 0 – to 2000 veh/hour/lane. (Both figures give the range of 10 minute averages observed).
- 3.2 Table 3/1 below defines the variables used in the relationships and gives the ranges of values over which the relationships should apply. The relationships cannot necessarily be taken to apply outside the given ranges of the variables. The geometric variables (BEND, HILL and H_R) should all be averaged over a reasonable length of road, generally not less than two kilometres.

SYMBOL	VARIABLE DESCRIPTION	TYPICAL VALUES	
		Min	Max
BEND	Bendiness; total change of direction (deg/km)	0	60
HILL	Sum of rises and falls per unit distance (m/km)	0	45
H _R	Sum of rises per unit distance one-way links only (m/km)	0	45
H _F	Sum of falls per unit distance one-way links only (m/km)	0	45
PHV	Percentage of heavy vehicles (OGV1 + OGV2 + PSV)	2	30
V _L , V _H	Speed of light and heavy vehicles (kph)	45	speed limit
S _L , S _H	Speed/flow slope of light and heavy vehicles (kph reduction per 1000 increase in Q)	0	55
Q	Flow, all vehicles, two-way or one-way (vehs/hour/lane)	0	2300
Q _B	Breakpoint: the value of Q at which the speed/flow slope of light vehicles changes (vehs/hour/lane)	1080	or 1200
V _B	Speed of vehicles at flow Q _B (kph)	80	105
Q _C	Capacity flag: defined as the maximum realistic value of Q (vehs/hour/lane)	1400	2250

Table 3/1: Definition of Variables Used in Speed Prediction Formulae for Rural All-Purpose Dual Carriageways and Motorways

- 3.3 Q_C, the maximum realistic value of Q, is taken as:

$2330 / (1 + 0.015 \times \text{PHV})$ for motorways, and

$2100 / (1 + 0.015 \times \text{PHV})$ for all-purpose dual carriageways.

It is this value of Q_C that triggers the overcapacity flag in COBA which is used to identify to users links which are likely to be overloaded. When flows reach this level the user must decide whether the flows are realistic and what course of action to take. Because COBA calculates the average lane flow of both directions of travel links with peak hour tidality may reach capacity earlier.

3.4 Q_B , the value of Q at which the speed/flow slope changes, is taken as 1200 and 1080 vehicles/hour/lane for motorways and all-purpose dual carriageways respectively.

3.5 For flow levels less than the breakpoint (Q_B) the speed prediction formula for light vehicles, in kph is:

$$V_L = K_L \quad \begin{array}{l} - 0.1 \times \text{BEND} \\ - 0.14 \times \text{HILL (two-way links only)} \\ - 0.28 \times H_R \text{ (one-way links only)} \\ - S_L Q, \end{array}$$

where K_L is 108 for dual 2-lane all-purpose (COBA Class 2)
 115 for dual 3-lane all-purpose (COBA Class 3)
 111 for dual 2-lane motorways (COBA Class 4)
 118 for dual 3-lane motorways (COBA Class 5)
 118 for dual 4-lane motorways (COBA Class 6),

S_L the speed/flow slope for light vehicles,
 is 6 kph per 1000 vehicles.

NOTE: for two-way links HILLS as entered by the COBA user is $H_R + H_F$; H_F is defined as the sum of the falls per unit distance. For one-way links H_R is coded in the HILLS field and H_F in the DOWN field. H_F is not used in the speed prediction formulae, it is only used in the calculation of vehicle operating costs.

3.6 The speed prediction formula for heavy vehicles which is applied at all flow levels, in kph is:

$$V_H = K_H \quad \begin{array}{l} - 0.1 \times \text{BEND} \\ - 0.25 \times \text{HILLS (two-way links only)} \\ - 0.5 \times H_R \text{ (one-way links only),} \end{array}$$

where K_H is 86 for all-purpose (COBA Classes 2 and 3)
 93 for motorways (COBA Classes 4, 5 and 6),

subject to the constraint that if the calculated value of V_H is greater than V_L then V_H is set equal to V_L .

3.7 At flow levels greater than the breakpoint (Q_B) the speed prediction formula for light vehicles, in kph is:

$$V_L = V_B - 33(Q - Q_B)/1000.$$

3.8 Heavy vehicle speed/flow relationships do not change at the breakpoint.

3.9 The minimum speed cut-off for rural dual carriageways and motorways is 45 kph.

- 3.10 The number of lanes is determined from the road class except for D3AP (COBA Class 3) and D4M (Class 6) when the program will decide on the number of lanes from the road width.
- 3.11 H_R and H_F are distinguished in COBA on one-way links only, when they are defined with respect to the direction of traffic flow. On two-way links, H_R and H_F are both set to $H/2$.
- 3.12 The COBA dual carriageway speed/flow relationships are expressed in flow per lane. The majority of all-purpose dual carriageways and motorways are built with standard 3.65 metre width lanes. Consequently the research to develop the speed/flow relationships was undertaken on links with lanes of, or close to, the standard width and was not able to detect a significant width parameter for use in the speed prediction formulae. Also, unlike single carriageway links, the average speed of light vehicles on all-purpose dual carriageways is not influenced by the presence of a hard strip. If the average lane width of the proposed scheme is significantly less than the standard 3.65 metres then it may be necessary to code a special road class (see Chapter 7) the Overseeing Organisation may be contacted for advice.

Climbing Lanes on Dual Carriageway Roads

- 3.13 Climbing lanes on dual carriageway roads can be modelled in COBA using pairs of one-way links, that is, the two directions of travel are modelled as separate links. The traffic flow entered for each one-way link can be taken as half the two-way flow with carriageway widths being appropriate to each direction of travel. For example, if an extra lane in the form of a climbing lane is being added to a D2AP road, the uphill section should be modelled as D2AP (with width say 7.3m) in the Do-Minimum and as D3AP (width say 10.0m) in the Do-Something. The effect of the extra lane uphill will be to increase vehicle speeds due to the reduced flow per lane. The downhill section should be modelled as D2AP (width say 7.3m) for both the Do-Minimum and Do-Something. Unfortunately, the COBA default speed/flow relationships are not appropriate for the assessment of adding a climbing lane to an existing road because the speed/flow research was not developed for short lengths of uphill gradient.
- 3.14 Research carried out in 1990 (Economic assessment of climbing lanes on motorways and all-purpose dual carriageways, TRRL Contractors Report No CR143), produced individual lane speed/flow relationships specifically for roads with uphill gradients of 3% or more over a distance of at least 0.5km. These relationships, suitably adjusted so that they apply to the whole width of the road, can be used instead of the standard defaults contained in COBA. Link specific speed/flow relationships can be defined depending on the type of road being improved, the local percentage of heavy vehicles (PHV), gradient of the uphill section and length of climbing lane. A method for preparing a link specific speed/flow relationship for modelling climbing lanes on dual carriageway roads is given in Part 7 under the KEY 027 heading.
- 3.15 This method also allows the user to define different accident rates and costs for the Do-Minimum and Do-Something situations if appropriate. Where the climbing lane is to be added to an existing road, an analysis of the delay costs during construction (normally assessed using QUADRO) should be included. Also, a maintenance analysis (again using QUADRO) for the route both with and without the climbing lane needs to be carried out to estimate the change in user delay costs and capital costs for future maintenance. Current standards (TD 9/93 DMRB 6.1) indicate that a gradient of 3% over a distance of 0.5km is expected to be the minimum that would justify an additional lane.

4. URBAN ROADS (ROAD CLASSES 7 AND 8)

- 4.1 In built-up areas, generally subject to a local speed limit, the road network becomes more dense and intersections play a more significant role in determining speeds. The speed/flow relationships that have been developed for urban areas apply to the main road network in towns (population greater than 70,000) and cities where there is a 30 mph (48 kph) speed limit. The relationships are designed to represent average-speeds in towns on the roads that function as traffic links. A distinction is made between central and non-central areas, and they include an allowance for junctions. They are linear relationships of fixed negative slope with a minimum speed cut-off.
- 4.2 The relationships were derived from data obtained in the Urban Congestion Study 1976 (TRRL SR 438) and in similar surveys in 1971, 1967 and 1963. The surveys were carried out in eight towns (population range 77,000 to 560,000) and in the principal cities of five conurbations (population range 300,000 to 1,100,000). The surveys were not intended to provide relationships to estimate speeds on individual links in urban areas; in practice urban speeds depend more on junction geometry and turning movements than on link characteristics; and complex relationships would be required to provide reasonable estimates of speeds on each link. However, the surveys do provide sufficient information to estimate average speeds in towns, distinguishing central and non-central areas.
- 4.3 Central areas are defined as those including the main shops, offices and central railway stations, with a high density of land use and frequent multi-storey developments, as in the widely used classification 'central business district' or CBD. Conurbations will have several CBDs (for example, central areas of Croydon, Ealing and Hammersmith are CBDs in London) whilst most free-standing towns will normally have only one. Streets which have commercial or industrial development but are not of a high-density CBD nature should not be included in the central area. Non-central areas comprise the remainder of the urban area. With this classification, the central areas constituted between 4 per cent and 22 per cent (average 11 per cent) of the total street length in the networks of the 13 towns studied.
- 4.4 The definition and range of the variables used in the speed prediction formulae are given in Table 4/1.

SYMBOL	VARIABLE DESCRIPTION	TYPICAL VALUES	
		Min	Max
INT	Frequency of major intersections averaged over the main road network (no/km)	2	9
DEVEL	Percentage of road network with frontage development (%)	50	90
V	Average vehicle speed (kph)	15	48
V ₀	Speed at zero flow (kph)	28	48
Q	Total flow, all vehicles, per standard lane (vehs/hour/3.65m lane)	0	1200
Q _c	Capacity flag: defined as the maximum realistic value of Q (vehs/hour/3.65m lane)	800	

Table 4/1: Definition of Variables Used in Speed Prediction Formulae for Urban Roads

4.5 The average vehicle speed V kph at flow Q vehs/hour/3.65m lane is given by the relationship:

$$V = V_0 - 30 \times Q / 1000,$$

where V_0 , the speed at zero flow, is defined below for Central and Non-central areas. The maximum vehicle speed is limited to the legal speed limit and there is a minimum speed cut-off (Paragraph 4.11).

4.6 The maximum realistic flow (Q_C) at which the COBA capacity flag is triggered is 800 vehicles/hour/3.65m lane. For urban links this value is not affected by the proportion of goods vehicles.

4.7 Where the introduction of the scheme being evaluated is predicted to produce major flow changes in urban areas local journey time surveys should be undertaken to validate the COBA modelling of speeds through the area. Where local journey time information indicates that the standard relationships are inappropriate then the user is able to input a value of speed (V_j) and corresponding flow (Q_j) through which a line of fixed negative slope of -30 kph per 1000 vehicles is drawn.

4.8 Generally the use of area wide speed/flow relationships which include an allowance for junction delays will be satisfactory. However in some cases schemes are designed to relieve a particularly congested junction and there may be a good case for modelling the junction explicitly; the comparison of local journey time information with COBA modelled times will demonstrate whether this approach is necessary. Where a scheme is designed to relieve a set of junctions that are clearly interdependent, consideration should be given to using dynamic traffic modelling techniques as a supplement to the COBA evaluation, and as an aid to determining which junctions to model separately in the COBA network. Whenever individual junctions are modelled separately in the urban network, the case for so doing should be clearly set out and sent with the COBA analysis for validation.

4.9 All links in a particular area of a particular town should have the same value for INT or DEVEL, and therefore the same speed/flow relationship per standard lane will apply. This emphasises the area -wide nature of the urban speed relationships. Changes in speeds in urban areas are predicted on a link -by- link basis, but the absolute level of speed is set by reference to the network- average speeds observed in central and non-central areas.

4.10 With large traffic models it is also important that a reasonable balance is struck between the network structure and zoning system, so that flows on individual links are representative of flows expected on the ground. In some cases this may not be possible, in which case the speed/flow effect described above for individual links may not be appropriate. Such cases should be referred to the Overseeing Organisation.

4.11 COBA imposes the constraint that urban speeds should not fall below the slowest average speeds observed in towns in practice. This puts a reasonable lower limit to the fall in speeds due to traffic growth. The minimum speeds in COBA are 15 kph for central areas and 25 kph for non-central areas.

Non - Central Areas - (Road Class 7)

4.12 Non-central areas are defined as all those areas not included in the central area definition. The speed V_0 in kph at zero flow is given by the relationship:

$$V_0 = 64.5 - DEVEL / 5 \text{ kph},$$

where DEVEL is defined as the percentage of the non-central road network of the town that has frontage development, counting business and residential development as 100% and open space as 0%. DEVEL is normally in the range 50-90% with average values about 80%. There is a minimum speed cut-off of 25 kph.

Central Areas - (Road Class 8)

- 4.13 Central areas are defined as those including the main shops, offices and railway stations, with a high density of land use as in the widely used classification 'central business district' (CBD). Streets which have commercial or industrial development but are not of a high density central business district nature should not be included in the central area. The speed V_0 in kph at zero flow is given by the relationship:

$$V_0 = 39.5 - 5 \times \text{INT} / 4 \text{ kph,}$$

where INT is a measure of the frequency of major intersections averaged over the main road network. INT is calculated by dividing the total number of lengths of road between major intersections in the central area by the total length of main road in the central area. Major intersections will generally be roundabouts or traffic signals, but they may also be uncontrolled junctions where a significant traffic movement loses priority. This network based value is not directly comparable with a route based value used for suburban links. INT should generally be in the range 2 to 9 per km, with an average of about 4.5. A value less than 2 is not appropriate for Central areas; if this occurs part or all of the area classified as 'Central' should be re-classified as 'Non-central'. There is a minimum speed cut-off of 15 kph.

5. SMALL TOWN ROADS (ROAD CLASS 9)

5.1 The main urban relationships do not apply to towns with a population of less than 70000, for villages and for rural roads with short stretches of development. The small town speed/flow relationships have been developed for these locations from the results of a study of speeds in small towns (Journey Speeds through Small Towns, 1982 - Halcrow Fox and Associates). Like the suburban speed/flow relationships (COBA Classes 10 and 11) they do not apply to individual links, they model traffic speeds over the whole of a route that is subject to a speed limit of 30 or 40 mph. Unlike the suburban relationships, however, they do not distinguish between light and heavy vehicles, and they specifically exclude junction delays; junctions where the route loses priority must be modelled separately. The user allocates links to Routes and COBA will calculate a Route specific speed/flow relationship which is applied to all the links on that route. Table 5/1 defines the variables used in the relationships and ranges of typical values.

SYMBOL	VARIABLE DESCRIPTION	TYPICAL VALUES	
		Min	Max
DEVEL	Percentage of route with frontage (%)	35	90
P30	Percentage of route subject to a 30 mph speed limit (%)	0	100
V	Average vehicle speed (kph)	25	64
V _B	Average vehicle speed at Q _B	38	57
Q	Total flow, all vehicles, per standard lane (vehs/hour/3.65m lane)	0	1200
Q _B	Breakpoint: the value of Q at which the speed/flow slope changes (vehs/hour/3.65m lane)	700	
Q _C	Capacity flag: defined as the maximum realistic value of Q (vehs/hr/3.65m lane)	1200	

Table 5/1: Definition of Variables Used in Speed Prediction Formulae for Small Town Roads

5.2 The breakpoint flow Q_B is taken as 700 veh/hour/3.65 metre lane. The maximum realistic flow at which the COBA capacity flag (Q_C) is triggered is 1200 veh/hour/3.65 metre lane. As with the main urban formulae, there is no correction for traffic composition.

5.3 The average speed in kph of all vehicles for flows below the breakpoint (Q_B) is given by:

$$V = 70 - \text{DEVEL}/8 - \text{P30}/8 - 12Q/1000,$$

where DEVEL is the percentage of the length of route that has frontage development, counting business and residential development as 100% and open space as 0%: the value will normally lie in the range 35% to 90%.

5.4 For flows greater than Q_B the average speed in kph of vehicles is given by:

$$V = V_B - 45(Q - Q_B)/1000,$$

until the minimum speed cut-off of 30 kph is reached.

- 5.5 These relationships should not be used for routes with $P30 < 10\%$ (that is for routes with an almost continuous 40 mph limit), $DEVEL < 65$ (that is with less than 65% development), and access friction less than 3; access friction being defined as the total number, both sides, of laybys, side roads and accesses per km (excluding house and field entrances) divided by the carriageway width in metres. In such cases, the route should be split into links, as appropriate, and the standard rural relationships should be used instead.

6. SUBURBAN ROADS (ROAD CLASSES 10 AND 11)

- 6.1 The suburban speed relationships apply to the major suburban routes in towns and cities where the speed limit is generally 40 mph (64 kph). The relationships were derived from a study carried out in 1969/70 of the major suburban radial routes of London, Manchester, Liverpool and Birmingham (Speed Flow Relationships on Suburban Main Roads, 1972 - Freeman Fox and Associates). Sixty-nine sections of single and dual carriageway were selected for study, individual section lengths varying from 1.6 to 8.3 km. The predominant speed limit was 40 mph (64 kph), although some sections had no local speed limit and some had a 30 mph (48 kph) speed limit. Observed journey speeds varied from 15 to 80 kph.
- 6.2 The suburban relationships provide estimates of the average journey speed of light and heavy vehicles separately, including delays at junctions. Table 6/1 below defines the variables used in the relationships and gives the ranges of values over which the relationships apply. The relationships cannot necessarily be taken to apply outside the given ranges of the variables. The geometric variables INT and AXS should be averaged over a reasonable length of link, generally not less than two kilometres. Congested junctions should be modelled separately and not included in the calculation of the value of INT.

SYMBOL	VARIABLE DESCRIPTION	TYPICAL VALUES	
		min	max
INT	Frequency of major intersections (no/km)		0 2
AXS	Number of minor intersections and private drives (no/km)		5 75
PHV	Percentage of heavy vehicles (%)		2 20
V _L , V _H	Speed of Light and Heavy Vehicles (kph)		25 64
S _L , S _H	Speed/flow slope of light and heavy vehicles (kph) reduction per 100 increase in Q		0 45
V ₀	Speed at Zero flow (kph)		48 64
Q	Total flow, all vehicles, per standard lane (vehs/hour/3.65m lane)		0 1500
Q _B	Breakpoint: the value of Q at which the speed/flow slope changes (vehs/hour/3.65m lane)		1050
Q _C	Capacity: defined as the maximum realistic value of Q (vehs/hour/3.65m lane)		1350 1700

Table 6/1: Definition of Variables Used in Speed Prediction Formulae for Suburban Roads

- 6.3 The basic form of the relationships is that speed reduces as flow increases. The initial speed being dependent upon the road standard, the number of major intersections and the number of minor intersections and private drives. The rate of decrease in speed is dependent upon the number of major intersections until a critical flow is reached at which point the rate of speed decrease changes to a fixed value until a minimum speed cut-off is reached.

6.4 There are important differences between the definition of the variable INT for suburban compared to urban roads. In suburban roads INT is specific to each section of route and major intersections are either roundabouts or traffic signals. Junctions between consecutive links should not be double counted, and classified junctions, whose delays are separately assessed, should be excluded from INT. The number of minor intersections and private drives, AXS, should be the total for both sides of the road (even for dual carriageways).

6.5 The two-way maximum realistic flow (Q_C), the flow at which the COBA capacity flag is triggered, is the same for both single and dual carriageways and is calculated by the relationship:

$$Q_C = 1500 (92 - PHV)/80 \text{ veh/hour/3.65m lane.}$$

6.6 For consistency between flow groups a standard value of 12% heavy vehicles (a typical value for main roads) is used to calculate the point of change of slope (Q_B) of light vehicles by the relationship:

$$Q_B = 0.7 \times Q_C = 1050 \text{ vehs/hour/3.65m lane.}$$

6.7 The speed for vehicles (V_0) at zero flow ($Q = 0$) in kph is given by:

$$V_0 = C - 5 \times INT - 3 \times AXS/20,$$

where, for single carriageways - ROAD CLASS 10

$C = 70$ for light vehicles, and

$C = 64$ for heavy vehicles,

for dual carriageways - ROAD CLASS 11

$C = 80$ for light vehicles, and

$C = 74$ for heavy vehicles.

6.8 The rate of decrease in speed (S) with increasing flow is the same for light and heavy vehicles and for single and dual carriageways. For values of flow (Q) less than the breakpoint (Q_B) for light vehicles and for all flow ranges for heavy vehicles:

$$S_L = S_H = 12 + 50 \times INT/3 \text{ kph per 1000 vehicles.}$$

For values of flow (Q) greater than the breakpoint (Q_b) the speed/flow slope for light vehicles increases to:

$$S_L = 45 \text{ kph per 1000 vehicles.}$$

6.9 The speed/flow slope for heavy vehicles does not increase when flow levels exceed the breakpoint. Therefore the calculated speed of heavy vehicles can exceed the speed of light vehicles, when this occurs the speed of heavy vehicles (V_H) is set to the speed of light vehicles (V_L).

6.10 The minimum speed cut-off is 25 kph for single carriageways and 35 kph for dual carriageways and the maximum speed is limited by the legal speed limit.

7. USER DEFINED RELATIONSHIPS (ROAD CLASSES 12 - 20)

7.1 In addition to the defined speed/flow relationship for Road Classes 1-11 the facility exists for the user to define special relationships. They are usually only used in special circumstances where the normal range of speed/flow relationships do not apply. They fit into two types:

i) **Special Road Classes 12-14,**

here the user can define relationships by the use of six speeds equated to specific flow levels. The relationships apply to both light and heavy vehicles and are independent of link geometric parameters

ii) **Road Classes 15-20,**

here the user can define basic constants to generate relationships similar to the form of those for the rural road classes 1-6, light and heavy vehicles are modelled separately. The effect of geometric parameters is the same as that for road classes 1-6 and cannot be changed. If these special road classes are used then, unlike classes 1-6, the speed/flow slope after the breakpoint will be fixed at the value defined by the user and will be independent of the proportion of heavy vehicles.

8. TREATMENT OF OVERCAPACITY ON LINKS

- 8.1 When the traffic flow on a link in a particular flow group exceeds the calculated theoretical ‘capacity’ for a lane, this is reported in PHASE 9 of the output (see Part 7 Paragraph 5.9). Details of the calculation used to determine the theoretical capacity for each by road type is given in Part 5. The overcapacity report is a signal to the user that the COBA evaluation is dealing with flow levels above those normally experienced on links of similar standard. Consequently the modelled situation may not be realistic and the benefits calculated by the program for the period of ‘overcapacity’ will become less meaningful the higher the degree of ‘overcapacity’. COBA has no control over the traffic flows that are input to the program. It is therefore possible for links in COBA to have traffic flows allocated, either in the base year or at some time in the future, that are not operationally feasible. Ultimately the user is asking COBA to assess a situation which breaks the bounds of common sense - that is, when future year traffic levels exceed physically practicable flows per lane. Depending on the Road Class overcapacity may be reached either before or after the calculated speeds fall below the minimum speed cut-off.
- 8.2 In exceptional circumstances it may be necessary to lower the minimum speed cut-off to obtain realistic speeds on the network. This is possible within COBA and can be implemented on a particular road class or an individual link. Any change contemplated should be agreed with the Overseeing Organisation in advance of any submission. The concept of minimum speeds on links is similar to that of maximum delays at junctions. The justification for using cut-offs of this nature is that in practice, one expects traffic behaviour to change to avoid bottlenecks.
- 8.3 Generally traffic flows input to COBA should not greatly exceed the calculated ‘capacities’, however when overcapacity is reported the user should consider three possible courses of action:
- i) if overcapacity is limited to a few links or junctions only and to the peak flow group only, the minimum speed cut-offs in COBA may be used as a proxy for the delay-avoiding behaviour that would be expected to occur. This amounts to assuming that the economic cost of alternative action is equal to the cost of travelling on the overloaded section of a route at the COBA minimum speed (or, on junctions, with the COBA maximum delay). Provided that benefits from the peak flow group are not a crucial factor in the economic case for the scheme, this assumption may be acceptable, but see ii) below;
 - ii) otherwise the user should consider whether simple changes to the assignment or ‘Do-Minimum’ improvements to the network may be devised to bring link or junction flows into better balance with link or junction capacities. If the ‘Do-Minimum’ involves a significant proportion of the ‘Do-Something’ expenditure (that is more than 20%), the ‘Do-Minimum’ expenditure should itself be economically justified against a ‘Do-Nothing’ (see Part 1 Chapter 2);
 - iii) exceptionally, in some large conurbations, predicted levels of congestion may be sufficiently severe to warrant modification to the COBA assumption that observed minimum speeds will be maintained in the future. In such cases the COBA cut-offs may not represent the full economic cost of trip suppression or alteration of time, route, destination or mode of travel. However, simply lowering the cut-off speeds (or increasing maximum delays) will ultimately overestimate the economic effect of congestion, because traffic will tend to take action to avoid or mitigate the effect of congestion. In these cases the advice of the Overseeing Organisation should be sought to ensure that a consistent approach to the problem is adopted.

9. REPRESENTATIVE DIAGRAMS OF SPEED FLOW RELATIONSHIPS

- 9.1 This chapter gives examples of the speed/flow relationships built into COBA for different types of road. In all cases the examples plotted should be seen as giving the form of a family of curves, differentiated according to the user defined geometric characteristics of each road.
- 9.2 The parameters used to define the example relationships for rural, urban, small town and suburban roads are given in Tables 9/1 to 9/4.

RURAL ROADS			
SINGLE CARRIAGEWAYS: CLASS 1			
	Typical S2	Designed to TD9/81	
CWID (m)	7.3	7.3	10
HILLS (m/km)	15	15	15
BEND (deg/km)	75	75	75
SWID (m)	0	1	1
VWID (m)	1	4	4
JUNC (no/km)	2	0.6	0.6
VISI (m)	300	400	400
MAXS	96 kph		
ALL-PURPOSE DUAL CARRIAGEWAYS: CLASSES 2 AND 3			
HILLS	15 m/km		
BEND	30 deg/km		
MAXS	113 kph		
MOTORWAYS: CLASSES 4,5 AND 6			
HILLS	15 m/km		
BEND	20 deg/ km		
MAXS	113 kph		
ALL CLASSES			
PHV	15%		
Minimum speed	45 kph		
(see Figures 9/1 and 9/2)			

Table 9/1: Values of Variables Used in Representative Speed/Flow Relationships - Rural Roads

URBAN ROADS	
NON-CENTRAL AREAS: CLASS 7	
DEVEL = 50% 'good' road conditions = 80% 'typical' road conditions = 90% 'poor' road conditions	
Minimum speed 25 kph	
CENTRAL AREAS: CLASS 8	
INT = 2 'good' road conditions = 4.5 'typical' road conditions = 9 'poor' road conditions	
Minimum speed 15 kph	
BOTH CLASSES	
MAXS 48 kph	
(see Figure 9/3)	

Table 9/2: Values of Variables used in Representative Speed/Flow Relationships - Urban Roads

SMALL TOWN ROADS				
CLASS 9				
Degree of devel	Light	Typical	Heavy	
DEVEL	35%	60%	90%	
P30	20%	50%	100%	
MAXS = 64 kph				
Minimum speed = 30 kph				
(see Figure 9/4)				

Table 9/3: Values of Variables Used in Representative Speed/Flow Relationships - Small Town Roads

SUBURBAN ROADS			
SINGLE CARRIAGEWAYS: CLASS 10			
DUAL CARRIAGEWAYS: CLASS 11			
Conditions	Good	Typical	Poor
INT (no/km)	0.4	0.8	1.2
AXS (no/km)	15	30	40
PHV	12%		
MAXS = 64kph			
Minimum speed = 25 kph single carriageways = 35 kph dual carriageways			
(see Figure 9/5)			

Table 9/4: Values of Variables Used in Representative Speed/Flow Relationships - Suburban Roads

- 9.3 Figure 9/1 shows the average speeds at different levels of traffic flow **per lane** for rural roads. The relationships cannot be applied to predict different speeds for different lanes of the same carriageway. The relationships exclude delays caused by merging on grade separated links, these must be modelled separately using the Merge Junction facility (see Part 6, paragraph 3.12). Because average vehicle speeds are calculated from light and heavy vehicle speeds weighted in the correct proportions there is a 'double knee' in the relationships. Figure 9/2 shows average speeds at different levels of total two way traffic flow, the advantage of this presentation is that, given a predicted traffic flow, the difference in predicted traffic speeds on different standards of road can be read directly from the diagram (without calculating the different levels of flow per lane). The speed differences will of course depend upon the precise geometric characteristics of the roads and the proportion of heavy vehicles.
- 9.4 The relationships for urban, small town and suburban roads are plotted on Figures 9/3 to 9/5. It must be remembered that the suburban and small town relationships were developed to give average speeds on routes and the urban relationships for areas of a town, consequently individual link speeds are unlikely to be modelled accurately.

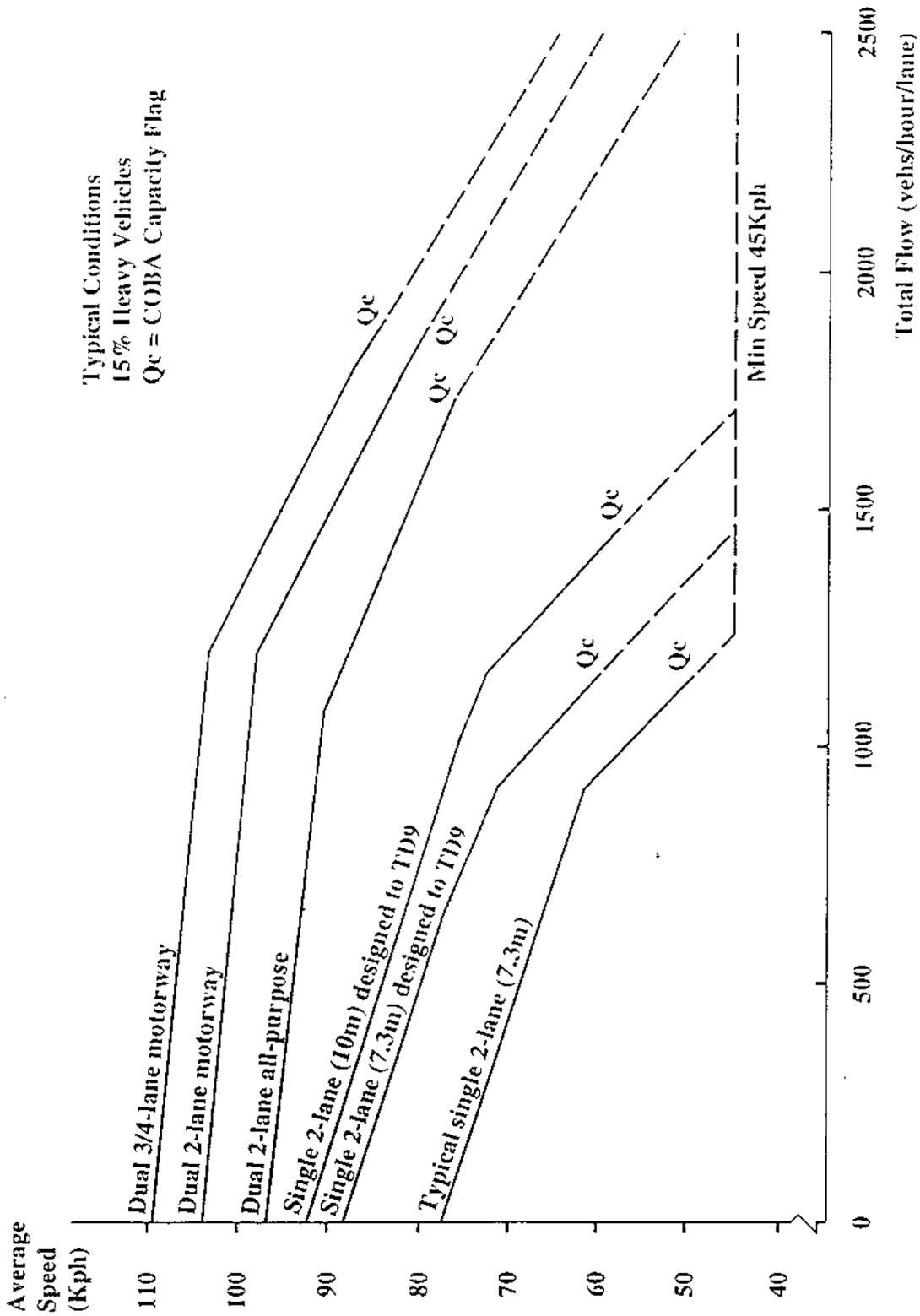


Figure 9/1: Typical Rural Speed/Flow Relationships - Vehicles/Hour/Lane

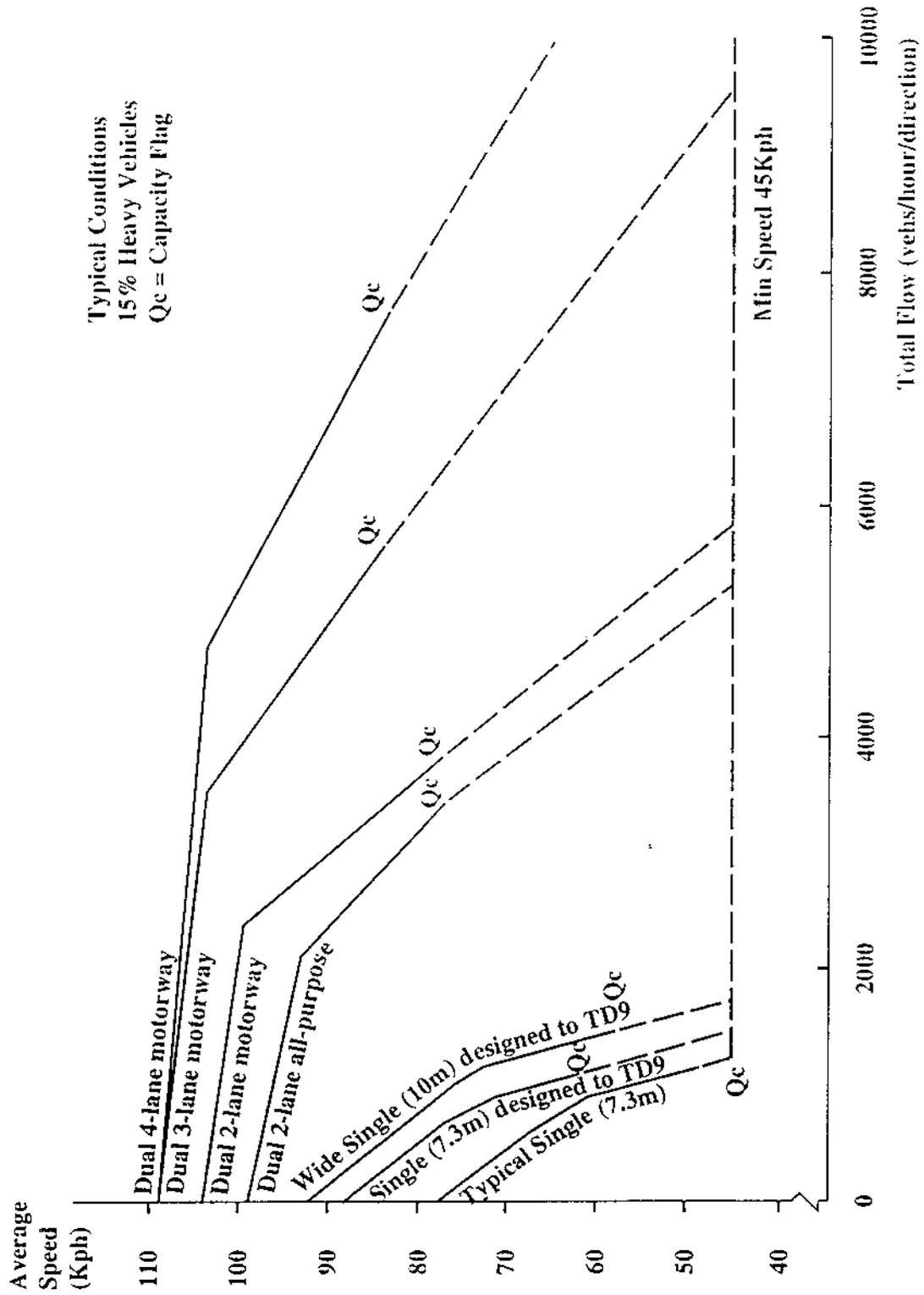


Figure 9/2: Typical Rural Speed/Flow Relationships - Vehicles/Hour/Direction

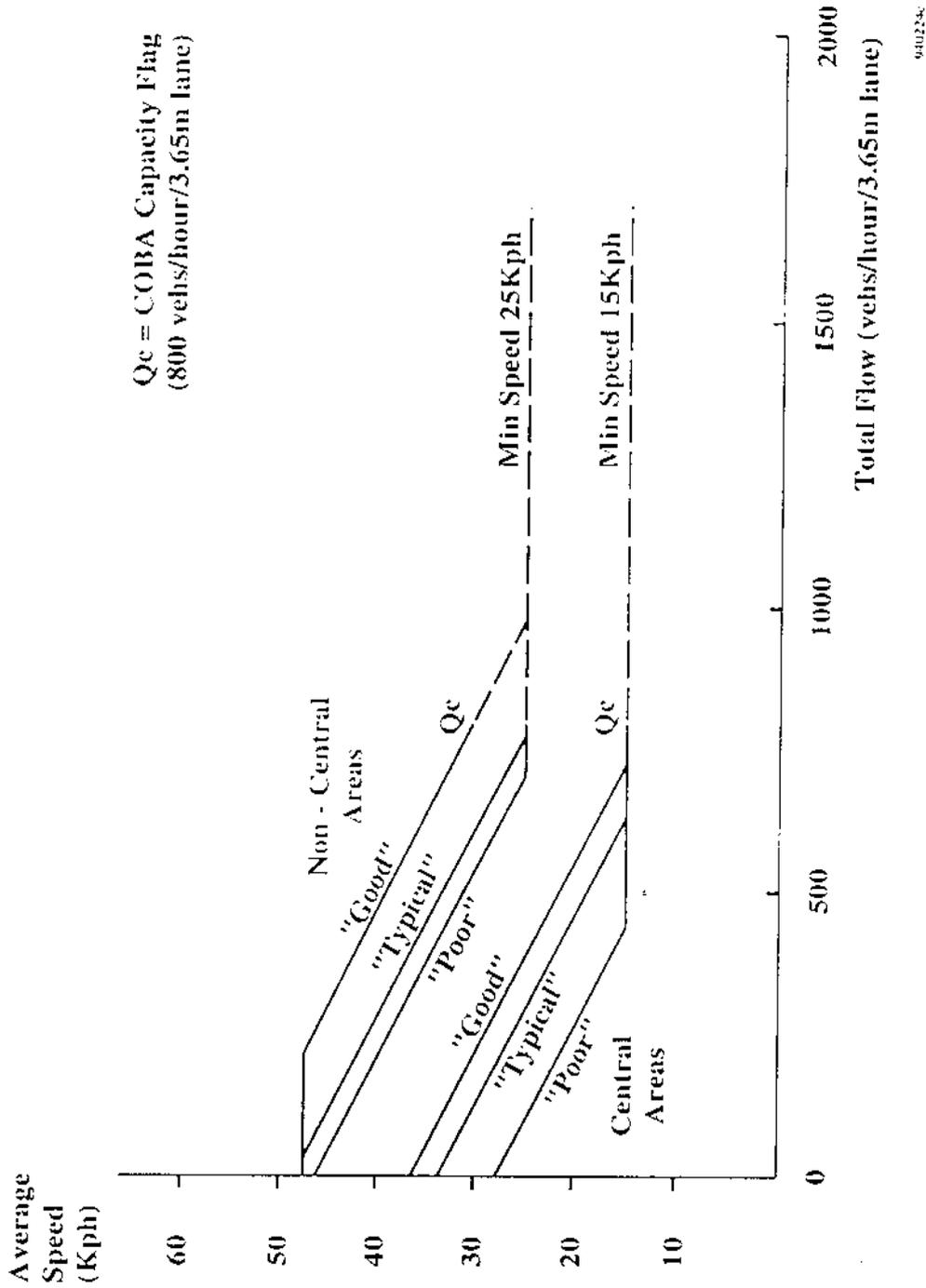


Figure 9/3: Typical Urban Speed/Flow Relationships

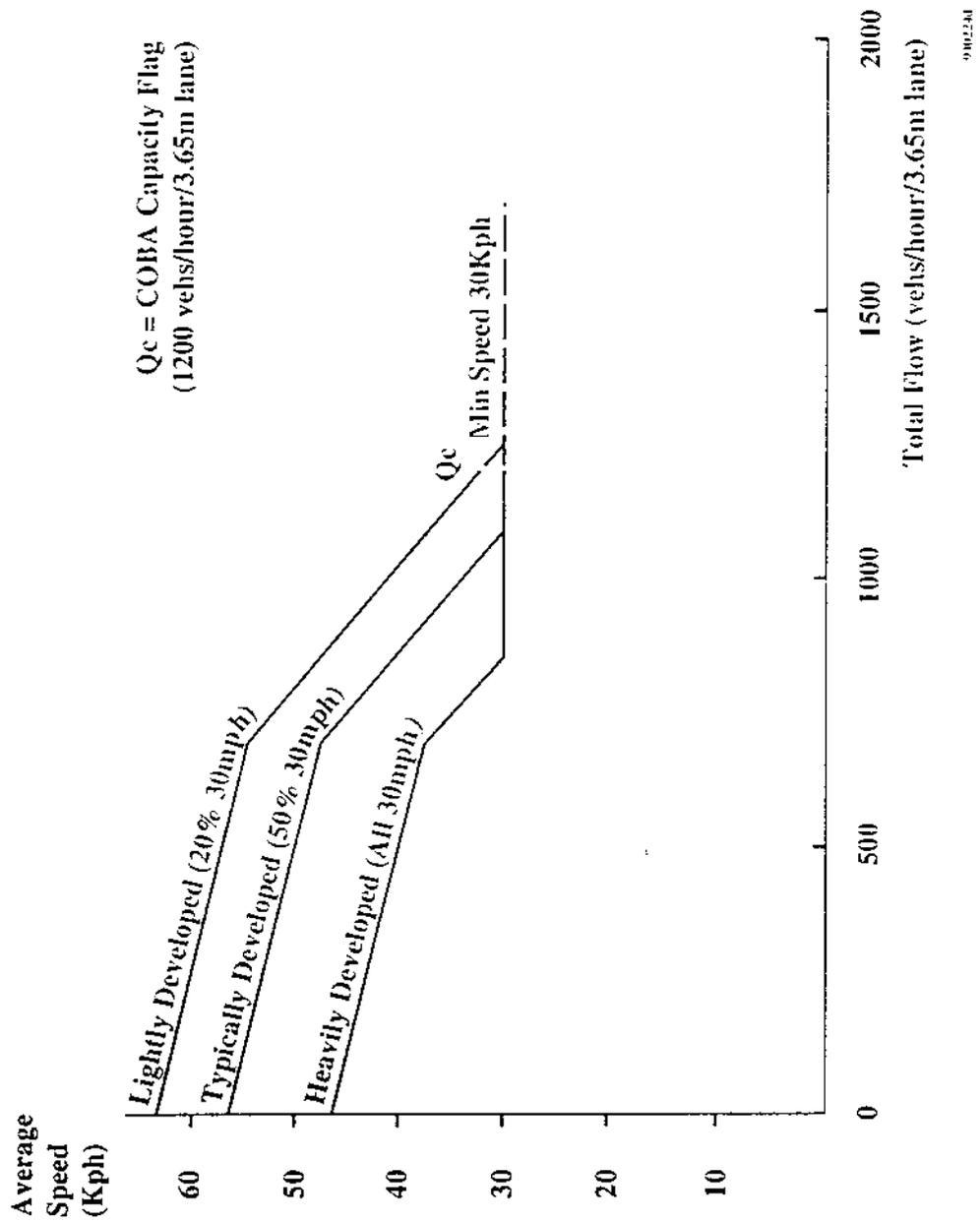


Figure 9/4: Typical Small Town Speed/Flow Relationships

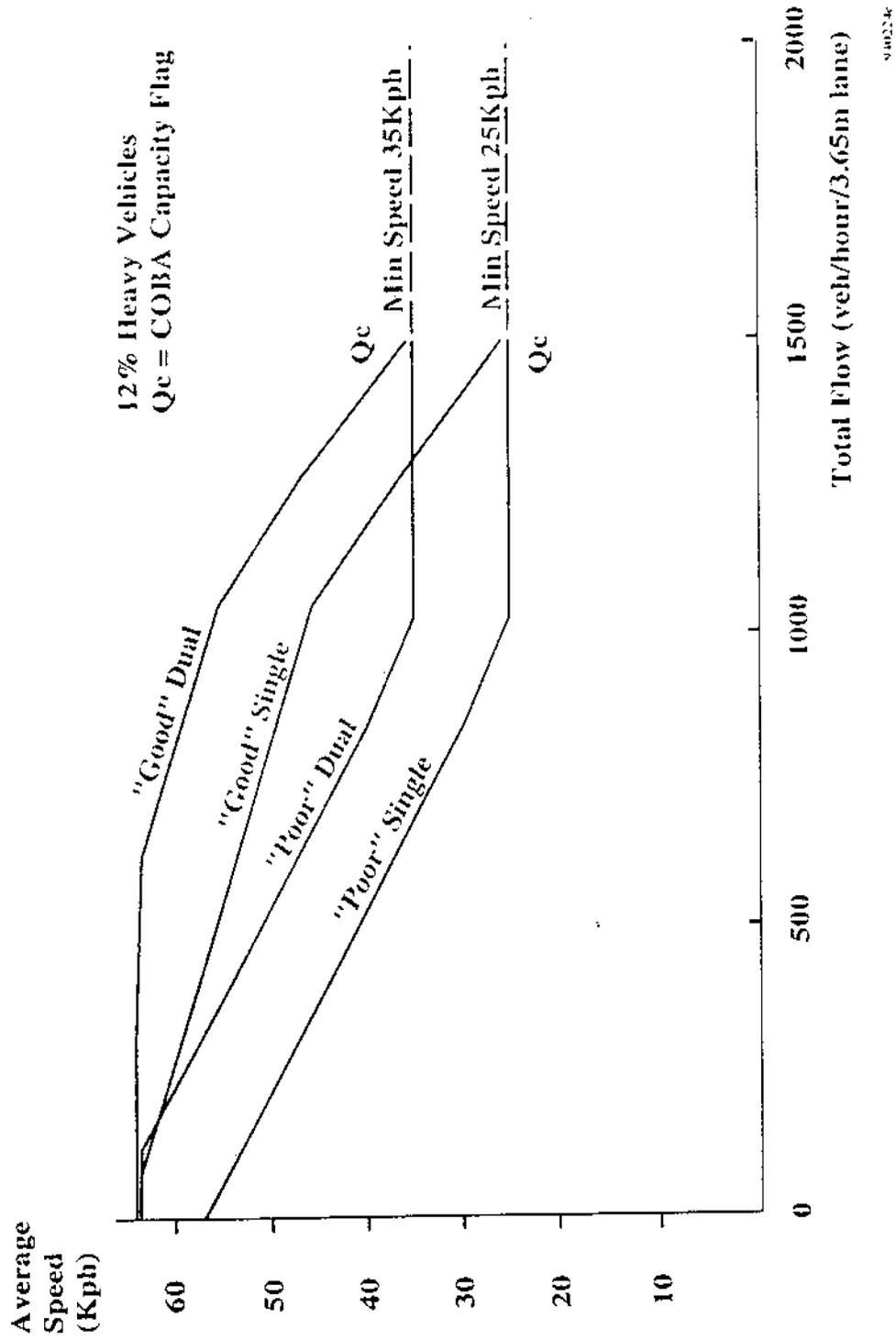


Figure 9/5: Typical Suburban Speed/Flow Relationships

10. LOCAL JOURNEY TIME MEASUREMENTS

- 10.1 It is always good practice to check observed journey times with those on the assignment networks and those computed by COBA. COBA journey times for each flow group are printed out for a specified year in Phase 8 of the COBA output.
- 10.2 Deleted.
- 10.3 Journey time measurements carried out for COBA should be used to estimate average speeds only. The journey time measurements should be geared towards estimating observed journey time on the whole of the bypassed section of route, individual estimates of speed on each link are not required. In general the longer the section of route which is being bypassed, the lower should be the variability of observed journey times (measured by the coefficient of variation). Fewer observations should be needed to estimate the journey time over a longer section of route to a given level of accuracy.
- 10.4 The most important consideration for local journey time measurements is that the journey times should be representative of conditions throughout the year. A large number of measurements carried out on one day will usually be worth less than fewer measurements spread over several days. Generally measurements should be taken to cover both peak and off peak hours of the day.
- 10.5 Deleted.
- 10.6 The journey time measurements should be compared with the COBA modelled times and flows in each flow group (modelled speeds in the journey can be obtained by use of the 'Journey Time Year' facility in COBA - see Part 7 Chapter 4). If junctions are modelled then the COBA modelled delays must be added to the link times before comparison with survey results (see paragraph 10.9). If observed and modelled journey times are not in reasonable agreement then the definition of local speed/flow relationships should be considered.
- 10.7 On routes where traffic flows show marked seasonality, journey time measurements should be carried out during the neutral months (April, May, June, September and October). This will give the best estimate of average traffic speeds over the whole year. Summer peak period journey time measurements may be useful to identify particular junction problems; in this case the measurement should be used to justify modelling key junctions in the COBA network, rather than altering the urban speed/flow relationships. Where more than one junction is modelled the effects of possible interaction between the junctions should be considered. Since these formulae include the whole of the delay attributable to the junction, but do not include running time in the approaches, the measurement technique for excluding junction delay from the overall links journey time should be as follows:
- i) timing points should be selected in the junction approaches beyond the normal limit of queues;
 - ii) timing and the balancing of overtaking and overtaken vehicles, should cease at those points in approaches; and,
 - iii) the time required to cover the remainder of each approach at an appropriate speed should be added to the time recorded. The speed used should, if possible, be the calculated link speed.
- 10.8 Deleted.

Modelled and Observed Journey Time Comparison

- 10.9 The acceptance of the modelled journey times is a matter of judgement. In reaching a conclusion the user should consider the following questions, bearing in mind that in general the traditional morning and evening peak hours will generally be represented by flow groups 4/5 and the inter-peak will generally be contained within flow groups 2 and 3. Flow Group 1 represents the quiet nighttime hours and it is unlikely that journey time surveys for these hours will be justified:
- were the flow levels at the time of observation similar to the COBA modelled flows?
 - What is the reliability of the observed journey times, especially junction delays?
 - What is the difference, if any, between observed peak and off-peak journey times and is this difference reflected in the modelled times?
 - Is there a distinct tidality effect and in this being modelled by COBA?
 - Is the journey significant in terms of the assessment of the scheme? (Routes with significant flow changes require a better validation than those with minor flow changes).
- 10.10 Tables 10/1 and 10/2 give suggested layouts for recording the observed and the COBA modelled journey times to assist the user in making a valid comparison.

From Node A to B	Time (Seconds)			Flow (vph)		
	AM Peak	Off-Peak	PM Peak	AM Peak	Off-Peak	PM Peak
Link W						
Link X including Junction Y						
Link Z						
Total				n/a		
Ditto for Node B to Node A						

*Note: Dates and times of observations should always be recorded

Table 10/1: Observed Journey Times

From Node A to B	Time (Seconds)				Flow (vph)			
	FG2	FG3	FG4	FG5	FG2	FG3	FG4	FG5
Link W								
Link X								
Junction Y								
Link Z								
Total					n/a			
Ditto for Node B to Node A								

Table 10/2: COBA Modelled Journey Times

11. ACCURACY OF LOCAL JOURNEY TIME MEASUREMENTS

- 11.1 The level of accuracy required for COBA should in theory be determined by detailed sensitivity testing. For example, if the journey time on an existing route is known to a certain accuracy at a given confidence level, this implies that the Net Present Value of the scheme should fall within a certain range at the same level of confidence (ignoring uncertainty arising from forecasting and specification errors which are less tractable to statistical analysis). If the range of NPV's is too large to be useful, more journey time measurements would be needed. In practice for most schemes the principal factor will be the difference between the journey time on the main existing route and the journey time predicted by COBA for the new route. This **difference** in times should be large compared with the **variation** in observed times. As a general rule it should be realistic to aim for an accuracy of $\pm 10\%$ in the estimate of journey time on the existing route, at the 95% confidence level. (On individual links the level of accuracy need not be so great).
- 11.2 If estimates of journey time are required to an accuracy of $\pm a$ at a confidence level of y (where a is expressed as a fraction), this implies that the true mean journey time should lie in the interval $(m - am)$ to $(m + am)$ in $y\%$ of cases, where m is the observed mean journey time. For example, if the observed mean journey time m is 10 minutes, the accuracy required $\pm 10\%$ ($a = 0.1$) and the confidence level $y = 95\%$, then the requirement is that in 95% of cases, the true mean journey time should be between 9 and 11 minutes (in other words, 95% confidence that the true mean journey time is 10 ± 1 minute).
- 11.3 A measure of the variability of observed journey times is the coefficient of variation (CV), defined as the ratio of the standard deviation to the mean of the observed journey times. (CV is assumed here to be expressed as a percentage). If n is the number of observations of journey time and X_1 to X_n the individual journey times, the estimate of true mean journey time (m) is

$$m = \frac{\sum X_i}{n}$$

and the estimate of the standard deviation (s) of true mean journey time is

$$s = \sqrt{\frac{\sum (X_i - m)^2}{n-1}}$$

and the coefficient of variation (CV) is then

$$CV = \frac{s}{m} \times 100$$

- 11.4 If it is assumed that observed journey times are normally distributed, the Student's t -distribution may be used to determine the level of accuracy achieved in the estimate of true mean journey time. The true mean journey time is required to lie in the interval $m \pm am$, and according to the t -distribution it is expected to lie in the interval

$$m \pm t \cdot \frac{s}{\sqrt{n}}$$

where the value of t depends on $(n-1)$, the number of degrees of freedom, and y , the confidence level.

Thus accuracy (a) is

$$a = t \cdot \frac{s}{m \cdot \sqrt{n}}$$

- 11.5 For example, if 20 measurements have been carried out and the observed mean journey time is 15 minutes with a standard deviation of 5 minutes, then at the 95% confidence level, the value of t (given in standard statistics text books) is 2.093 with 19 degrees of freedom. The accuracy a is thus

$$a = 2.093 \cdot \frac{5}{15 \cdot \sqrt{20}} = 0.16$$

or only 16%. More journey time measurements would generally be required to achieve a satisfactory level of accuracy.

- 11.6 Table 11/1 can be used to determine how many journey time measurements would be needed to achieve a given accuracy with 95% confidence in the result.

Coefficient of Variation of observed journey times (CV)	Number of Journey Time Measurements	
	Accuracy $\pm 10\%$	Accuracy $\pm 20\%$
5%	4	2
10%	7	2
15%	11	3
20%	18	5
30%	35	9
40%	64	16
50%	100	25

Table 11/1: Required Number of Journey Time Measurements

- 11.7 In the example given in 11.5, the coefficient of variation ($CV = s/m \times 100$) of the observed journey times is 33%. By interpolation the table shows that, for an accuracy of $\pm 10\%$ at the 95% confidence level about 45 journey time measurements would be required. Consideration should therefore be given to mounting a second journey time survey to cover another 25 timed measurements along the route under survey.
- 11.8 Deleted.