Supplementary Information to the "Traffic Review for the Land Use Proposal on the Western Part of Kennedy Town" (the Traffic Review) (Appendix II of Attachment V of the MPC Paper No. 1/16 refers) provided in April to June 2016 upon the receipt of a request from a member of the public

1) **Supplement to paragraphs 2.3 and 3.1 (Assumptions and methodology)**

- Traffic surveys were conducted in 2015 to collect existing vehicular and pedestrian flow data. As mentioned in Para. 2.3 of the Traffic Review, the design year adopted for the assessment was 2027. An interim year (2024) was also selected to review the need of any interim traffic improvement measures upon completion of the public housing site. To work out the traffic forecast for the design year and the interim year, a set of growth factors (+3.81% for years from 2015 to 2016, 0% for years from 2016 to 2021, +1.88% for years from 2021 to 2026, +0.34% for years from 2026 to 2031) was used with reference to the historical traffic data of Annual Traffic Census (from 2010 to 2014) published by the Transport Department and the planning data for the latest enhanced 2011-based Territorial Population and Employment Data Matrices published by the Planning Department.
- The vehicular and pedestrian trips induced by the new developments of the western part of Kennedy Town were also taken into account in the traffic forecast. The vehicular generation and attraction were estimated with reference to the trip generation and attraction rates in Tables 1 & 2 of the Appendix in Chapter 3 of Volume 1 of Transport Planning Design Manual published by the Transport Department. To estimate the pedestrian generation and attraction, two assumptions, i.e. 90% of the total population of the public housing development would make 2 trips per day, and the morning, afternoon and evening peak hours would account for 12%, 5% and 12% of the daily trip total respectively, were made based on the results of the Travel Characteristic Survey 2011.

2) **Supplement to paragraph 2.3 (Methodology of junction capacity assessment)**

Methodology

- Priority Junction - J28 (2015, 2024 and 2027 without improvement) and J29
  - The calculation is based on a set of empirical formulae/equations, which are shown in Annex A. In general, the parameters of junction geometry which have been found to exert the major influence on capacity of non priority movements are: major road width, width of central median, lane width available to waiting traffic streams and visibility distances for non priority

traffic streams. These parameters are employed in the equations.

- In evaluation, the design flow should be compared with the calculated capacity to produce a design flow/capacity ratio (DFC) for each non priority movement.
- The methodology of traffic forecast was explained in 1) above.
- In general, the performance of the junction is considered satisfactory if DFC of the priority junction is less than 0.85 during peak periods.

- Signalised Junction – J4, J4A, J28 (2027 with improvement) and J32
  - The concept and a set of equations for signal calculations are described and shown in Annex B.
  - The methodology of traffic forecast was explained in 1) above.
  - In general, the performance of the junction is considered satisfactory if Reserve Capacity of the signalized junction is greater than 15% during peak periods.

<u>Supplement to paragraph 3.1 (Methodology of pedestrian facilities assessment)</u>

- The methodology of estimation of pedestrian flow in 2027 was explained in 1) above.
- The Level-of-Services (LOS) is obtained by dividing the two-way pedestrian flow (i.e. 2-way ped/min) by the effective width of the footpath concerned.
- According to the Highway Capacity Manual (HCM-version 2000), the LOS is primarily based on the density of people in a given space. There are six levels. In general, LOS C is desirable for most design at streets with dominant "living" pedestrian activities. Extracted details of the HCM LOS are tabulated below.

Description of Level-of-Service (LOS)
(Reference:HCM 2000)

LOS	Flow Rate (ped/min/m)	Description
A	≤16	Pedestrians basically move in desired paths without altering their movements in response to other pedestrians. Walking speeds are freely selected, and conflicts between pedestrians are unlikely.
B	16 - 23	Sufficient space is provided for pedestrians to freely select their walking speeds, to bypass other pedestrians and to avoid crossing conflicts with others. At this level, pedestrians begin to be aware of other pedestrians and to respond to their presence in the selection of walking paths.
C	23 - 33	Sufficient space is available to select normal walking speeds and to bypass other pedestrians primarily in unidirectional stream. Where reverse direction or crossing movement exist, minor conflicts will occur, and speed and volume will be somewhat lower.
D	33 - 49	Freedom to select individual walking speeds and bypass other pedestrians is restricted. Where crossing or reverse-flow movements exist, the probability of conflicts is high and its avoidance requires changes of speeds and position. The LOS provides reasonable fluid flow; however considerable friction and interactions between pedestrians are likely to occur.
E	49 - 75	Virtually, all pedestrians would have their normal walking speeds restricted. At the lower range of this LOS, forward movement is possible only by shuffling. Space is insufficient to pass over slower pedestrians. Cross- and reverse-movement are possible only with extreme difficulties. Design volumes approach the limit of walking capacity with resulting stoppages and interruptions to flow.
F	>75	Walking speeds are severely restricted. Forward progress is made only by shuffling. There are frequent and unavoidable conflicts with other pedestrians. Cross- and reverse-movements are virtually impossible. Flow is sporadic and unstable. Space is more characteristics of queued pedestrians than of moving pedestrian streams.

- Ka Wai Man Road public housing development (Site 9) is the site with the largest population size of the Land Use Proposal on the western part of Kennedy Town. Pedestrians walking along the footpath of Ka Wai Man Road will be dispersed to the footpaths of adjacent roads, such as Victoria Road, which is similar to or even wider than that of Ka Wai Man Road. Due to the dispersion effect, the LOS of the footpath of Ka Wai Man Road is considered to be the most critical within the Land Use Proposal on the western part of Kennedy Town. As mentioned in Para. 3.1 of the Traffic Review, the walking environment of Ka Wai Man Road after the population intake of the public housing development would be acceptable.

<u>Supplement to paragraph 4.2 (Rationale for determining the provision of goods vehicle and private car parking spaces for public uses)</u>

- As mentioned in Para. 4.2 of the Traffic Review, about 50 goods vehicles and 70 private car parking spaces dedicated for public use will be provided at the underground car park at Site 3a, and 25 light goods vehicles parking spaces will be provided at the Ka Wai Man Road public housing development at Site 9.

Good vehicles

- It is observed that the demand of goods vehicles parking spaces on Hong Kong Island will be decreasing in the long run[1]. Taking into account the latest goods vehicle parking spaces currently available around Kennedy Town area and the potential reduction due to the termination of some short term tenancy parking sites in the future[2], we propose to provide about 50 parking spaces for goods vehicles at Site 3a and 25 parking spaces for light goods vehicles at Site 9.

Private cars

- Provision of parking spaces should be compatible with Government's overall transport policy. As a general principle, parking should be provided at a level which will not unduly attract potential passengers to use private vehicles in preference to public transport. The internal parking and loading/unloading facilities of new development sites should be self-contained and be provided within the site according to the requirements as stipulated in the Table 11 of Chapter 8 of Hong Kong Planning Standards and Guidelines (HKPSG). We constantly monitor the supply and demand situation of parking spaces of the territory. From time to time, we review the demand of parking spaces of various development types and update the parking standards in the HKPSG accordingly. Considering the above principle, and with due regard of the existing situation of illegal parking during mid-night at the Western District, about 70 parking spaces for private cars were suggested to be provided at Site 3a for public use in addition to the requirements stipulated in the HKPSG.

Supplement to section 6 (Public transport services for serving the new developments)

- The Transport Department will plan, at an appropriate time before the completion of the new developments, the use of public transport services including franchised bus and green minibuses (GMB) services for the new developments taking into account different factors including the changes of population and passenger demand, public views and result from service level

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1 Upon completion of the developments of the western part of Kennedy Town, it was estimated that the demand of parking spaces for goods vehicles in the Central and Western District would be less than 380.

2 Upon completion of the developments of the western part of Kennedy Town, the parking spaces available in the Central & Western District would be about 310.

investigation in order to meet the public need. This is to ensure the coordination between these public transport services and to promote the effective use of the resources of the public transport, as well as to minimize traffic congestion and roadside air pollution.

Calculation of Capacity at Priority Junctions

The predictive equations discussed in paras. 4.3.6 are :

$Q_{B-A} = D\ [627 + 14W_{CR} - Y\ (0.364q_{A-C} + 0.144q_{A-B} + 0.229q_{C-A} + 0.52q_{C-B})]$ .... (1)

$Q_{B-C} = E\ [745 - Y(0.364q_{A-C} + 0.144q_{A-B})]$ .... (2)

$Q_{C-B} = F\ [745 - 0.364Y(q_{A-C} + q_{A-B})]$ .... (3)

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

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

$D = [1 + 0.094(w_{B-A} - 3.65)]\ [1 + 0.0009(V_{rB-A} - 120)]\ [1 + 0.0006(V_{lB-A} - 150)]$

$E = [1 + 0.094(w_{B-C} - 3.65)]\ [1 + 0.0009(V_{rB-C} - 120)]$

$F = [1 + 0.094(w_{C-B} - 3.65)]\ [1 + 0.0009(V_{rC-B} - 120)]$

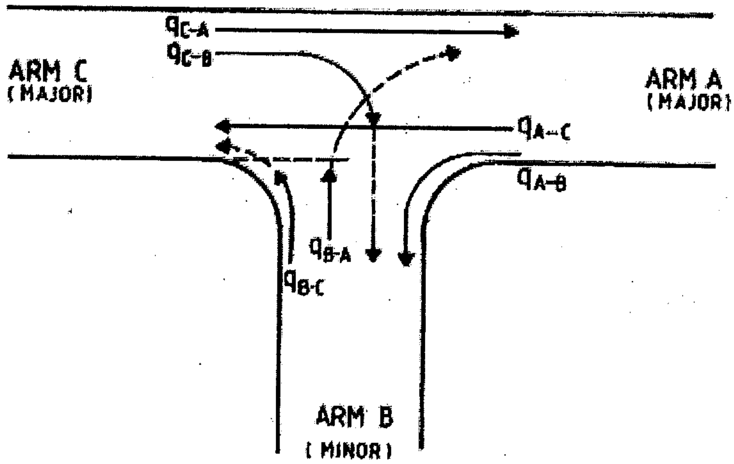
The symbols represent the following :-

Q_{B-A} =	the capacity of movement B-A	) See Drg. No. 4.3.A.1
Q_{A-B} =	the capacity of movement A-B and so on	) Capacities and flow are in pcu/hour (1 HGV = 2 pcu)
W =	major road width	) See Drg. No. 4.3.A.2
W_{CR} =	central reserve width (kerbed median only)	)
W_{B-A} =	lane width available to vehicle waiting in stream B-A, and so on	) See Drg. No. 4.3.A.3
V_{rB-A} =	visibility to the right for vehicles waiting in stream B-A	) Visibilities distances for minor road flow is measured
V_{lB-A} =	visibility to the left for vehicles waiting in stream B-A, and so on	) from a point 10m back from the give way line

All distances and widths are measured in metres and the ranges of parameters in the data base were as follows :-

w = 2.05 - 4.70

DIAGRAM 4.3.A.1: FLOWS AND NOTATION



The four parts of drawings show the main components of major road width. They are combined to give :

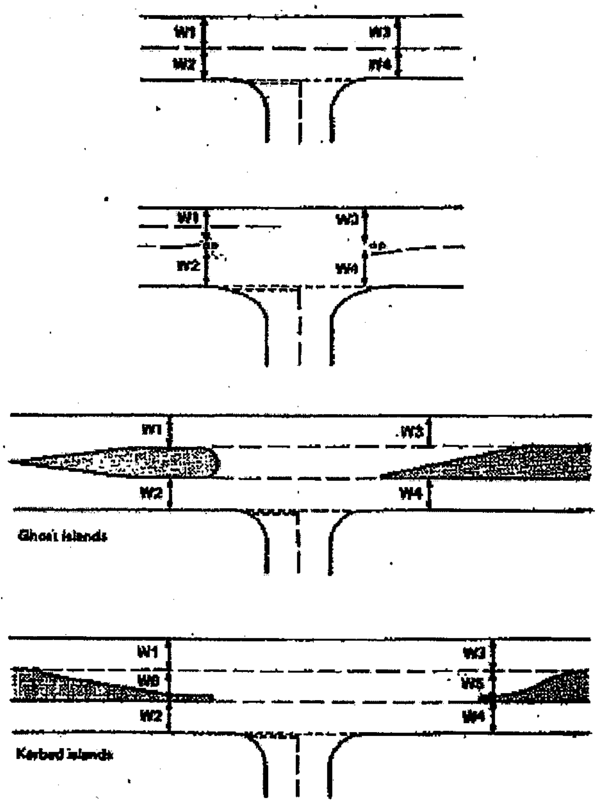
(1) the 'nearside' width: W_n
$W_n = ½ (W2 + W4)$

(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 4.3.A.2: MAJOR ROAD WIDTH W AND ITS COMPONENTS



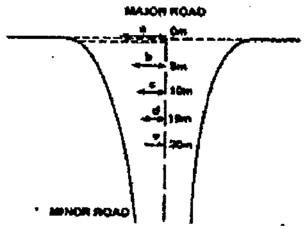
Lane width for non-priority streams, w (m)

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 exceeding 5m is reduced to 5m before the average is taken. Where lane markings are unclear (or absent), Diagram (a), (b), and (c) are used, and the lane width calculated according to :

DIAGRAM 4.3.A.3 : LANE WIDTHS FOR NON PRIORITY STREAMS

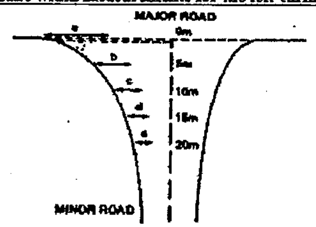
$W = (a + b + c + d + e) / 5$ metres

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



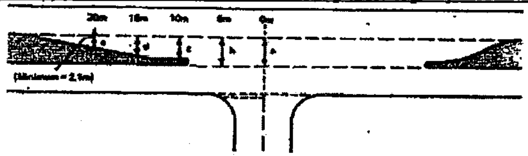
a, b, c, d, e are equal to ½ (approach width to nearside of median line)
Each $\leq$ 5m

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



a, b, c, d, e are equal to ½ (approach width to nearside of median line)
Each $\leq$ 5m

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



a, b, c, d, e are equal to the lane width where there is explicit provision for right-turners (each $\leq$ 5m), and equal 2.1m otherwise

2.4 Signal Calculations

2.4.1 Signal Calculation Concepts

2.4.1.1 The amount of traffic that can be pass through a signal-controlled intersection from a given approach i.e. the capacity, will depend on the green time available to the traffic and the maximum flow of vehicles pass the stopline during the green period.

2.4.1.2 The discharge of vehicles from a queue may be illustrated by Diagram 2.4.1.1. When the green period commences, vehicles take some time to start and to accelerate to normal running speed, but after a few seconds, the queue discharges at a more or less constant rate which is termed the 'saturation flow'.

2.4.1.3 The saturation flow may be defined as the maximum flow which can be obtained if there is a continuous queue of vehicles and they were given 100 percent green time.

2.4.1.4 For the convenience of signal calculation, the green and amber periods are replaced by an 'effective green' period (g), throughout which flow is assumed to take place at the saturation rate, and a 'lost' time (l) during which no flow takes place.

If
- k = combined green and amber period
- g = effective green period
- G = actual green time
- l = lost time for a single phase
- S = Saturation flow
- C = Cycle time

Then capacity Q from an approach may be expressed as

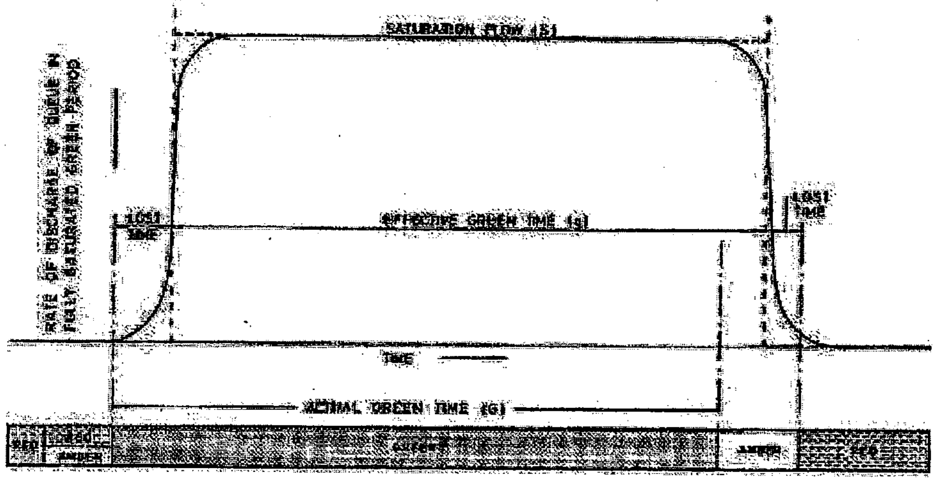
$$Q = \frac{gS}{C}$$

Where $g = k - l$

2.4.1.5 With amber period of 3 sec, l may be taken as 2 sec

Thus
$$k = G + 3$$
$$g = G + 3 - 2$$
$$g = G + 1$$

DIAGRAM 2.4.1.1: VARIATION WITH TIME OF DISCHARGE RATE OF QUEUE IN A FULLY SATURATED GREEN PERIOD



FOR AN AMBER PERIOD OF 3 SECONDS,
LOST TIME (l) MAY BE TAKEN AS 2 SECONDS.
THUS, g=G+3-2=G+1

2.4.2 Estimation of Saturation Flow

2.4.2.1 Exclusive Straight-ahead Lanes

TRRL RR 67 (Ref. 20) suggested that :-

(i) The saturation flow per lane (S) expressed in terms of passenger car units per hour (pcu/h), with no turning traffic may be given by
$S = 1940+100(W-3.25)$ for nearside lane or single lane entries
$S = 2080+100(W-3.25)$ for non-nearside lanes
Where W is the lane width at entry in metres.

(ii) The total saturation flow may be taken as the sum of the individual lane values.

(iii) Signal approaches are sometimes locally widened to provide an additional lane near the junction.

When the additional lane at the stopline is available for a distance back from the stopline at least sufficient to contain one full cycle capacity of traffic, the above methods of estimation of saturation flow apply. If the signal approach is only widened very close to the stopline, then the above formulae may be a considerable overestimate capacity. A more realistic approach capacity should be deduced from entry lanes before widening plus an allowance for extra vehicles stored every cycle in the additional lane provided near the intersection. In these circumstances, on sit estimation of saturation flow will be necessary.

Reference may be made to Table 2.2.3.2 in Section 2.2.3.5 for general guidance on marking of lanes at approaches.

2.4.2.2 Effect of Gradients

(i) For each 1 per cent of uphill gradient the saturation flow (S) should be decreased by 42 pcu/h. The gradient should be taken on the average slope between the stopline and a point on the approach 60m before it.

(ii) Downhill gradients have no effect on the saturation flow.

2.4.2.3 Effect of Composition

The effect of different types of vehicle on the saturation flow at traffic signals is given by the following p.c.u. equivalents :-

(Also in T.P.D.M. Vol. 2 Section 2.3)

Private Car, Taxi, Light Goods Vehicle	1.0
Motor Cycle and Motor Scooter	0.4
Medium or Heavy Goods Vehicles	1.75
Through Bus (see 2.4.2.6 (iii)) or coach	2.0
Pedal Cycle	0.2
Tram	3.5 – 5.0
Public Light Bus	1.5

2.4.2.4 Exclusive Turning Lane

$S_R = S/(1+1.5/r)$ for unopposing turning traffic

$S_R = (S-230)/(1+1.5/r)$ for opposed turning traffic

where r = radius of curvature of vehicle paths (m)

2.4.2.5 Lanes with Mixed Traffic

$S_M = S/(1+1.5f/r)$ for unopposing turning traffic

$S_M = (S-230)/(1+1.5f/r)$ for opposed turning traffic where f = proportion of turning vehicles in a lane.

2.4.2.6 Effect of Waiting Vehicles & Bus Stops

(i) Reduction in saturation flow caused by parked vehicles in the vicinity of the approach is equivalent to a loss of carriageway width at the stopline as follows :-

loss in c/w width = 1.68 - $\frac{0.9(z-7.62)}{G}$ (metres)

where z ($\geq$7.62) is the clear distance of the nearest parked vehicle from the stopline in metres.
G is green time in seconds.
If the whole expression becomes negative the effective loss should be taken to be zero. The effective loss may be increased by 50 per cent for a parked medium or heavy goods vehicle.

(ii) No stopping restriction before and beyond a junction may be considered where junction capacity is tight or where the parking/stopping activities are adversely affecting the effective operation of the junction signals.

(iii) The actual effects of a stopping bus are complex and will vary considerably depending on bus stop location, bus dwell time, parking activity, lane configuration and traffic volume. Until further research is accomplished, it is recommended that a p.c.u. value of 5.0 may be taken for stopping buses that operate and stop within 200 m of the signals.

Designers should also refer to Section 2.4 of T.P.D.M. Vol. 9 for detailed guidelines on siting of bus tops.

2.4.3 Measurement of Saturation Flows

2.4.3.1 It is not always practicable to conduct direct measurements of saturation flows e.g. because of resources constraints or when designing new intersections. Direct measurements are however definitely worthwhile for critical intersections working near saturation as the direct measurements will provide more accurate flows for detailed signal plan preparation or junction capacity analysis.

2.4.3.2 Formal measurements of saturation flow shall be conducted in accordance with U.K. Road Note No. 34.

2.4.3.3 Less formal measurements may be made for approaches where there are existing signals or where the junction is police controlled. It is usually satisfactory to carry out such counts commencing, say, 5 seconds after the traffic has started so as to eliminate starting delays and continuing the count until traffic begins to "tail off". A reasonably consistent value of saturation flow could usually be obtained after several counts.

2.4.4 Total Lost Time per Cycle (L)

2.4.4.1 With 2 sec red/amber (starting amber)

3 sec amber (leaving amber)
$I = 5 + R$ (see diag. 2.3.2.1)
where R = all red period
I = intergreen perod
If lost time for a single phase in the green and amber period $(Z) = 2$ sec
loss time per phase change $= (I - 3) + z$

Total lost time per cycle
$L = (I - 1)$
or $= (R + 4)$

2.4.4.2 For a full pedestrian stage during which all traffic is stopped, the pedestrian green and flashing green periods should be considered as additional all red periods (i.e. additional lost times) in signal calculations.

2.4.5 Flow Factors

2.4.5.1 Flow factor 'y' for each phase is given by

$$y = \frac{\text{design flow for an approach (q)}}{\text{saturation flow for an approach (S)}}$$

2.4.5.2 Where more than one approach is operating during a phase the maximum 'y' should be used. Where an early cut-off or late start is to be used in connection with a right turn the 'y' values for the right turn and for the approach with shortened time should be added together to represent one phase unless the straight on traffic on the same approach as the right turn has a higher value, when this latter figure should be taken.

2.4.5.3 The summation of these higher y values for the phases is the 'Y' value which is in fact a measure of the congestion and which will be used for calculating the optimum signal setting.

$Y = \Sigma y$

2.4.6 Cycle Times (C_o, C_m and C_p)

2.4.6.1 For an isolated signal installation, where the mean traffic level is constant and where vehicle arrivals are at random, the U.K. Transport and Road Research Laboratory (TRRL) has shown that the optimum cycle time for minimum delay is given by :-

$$C_o = \frac{1.5L + 5}{1 - Y} \text{ secs}$$

2.4.6.2 Also and the cycle time which is just sufficient to pass the traffic is given by :-

$$C_m = \frac{L}{1 - Y}$$

2.4.6.3 This is the minimum possible cycle time which may be associated with excessively long delays. In designing linked signals a cycle time should be chosen which provides a margin over this minimum possible cycle time for the key intersection. In practice it will be generally appropriate to choose a practical cycle time, such that the installation is then loaded to 90 per cent of its capacity

$$C_p = \frac{0.9L}{0.9-Y}$$

2.4.6.4 In locations where pedestrian crossing volumes are high it will be desirable to use as short a cycle time as practicable to minimize delays imposed on pedestrians. It will be of good practice to limit cycle times to below 90 seconds and where this is not intended due to capacity reasons, careful checking of site conditions should be made to ensure that pedestrians are not being endangered. In designing for new junctions the maximum operating cycle time should be limited to 90 seconds.

2.4.7 Green Times

2.4.7.1 Signal setting for the effective green periods(g) should be in proportion to the y values on each approach, with an allowance for lost time

$$\frac{g1}{g2} = \frac{y1}{y2} \text{ etc.}$$

where g = effective green period
y = flow facto

and $g = \dfrac{y(c - L)}{Y}$

G = actual green period

$G = g - 1$

c = cycle time

i.e. $g_1 = \dfrac{y_1(c - L)}{Y}$

L = total lost time

Y = summation of flow factors

$$g_2 = \frac{y_2(c - L)}{Y} \text{ etc.}$$

c- L = total effective green time

time

2.4.7.2 Care should be taken in performing the calculation if parallel pedestrian facilities are included in the junction method of control. If they are included, the minimum green times for the minor movements could well be dictated by parallel pedestrian crossing green times. This could well considerably distort the green split calculation.

2.4.8 Degree of Saturation

2.4.8.1 Degree of saturation (X) for individual approaches may also be expressed as :-

$$X = \frac{q}{Q}$$

where q = design flow
Q = capacity of approach
c = cycle time
S = saturation flow

2.4.8.2 The degree of saturation should be the same for all the predominant arms of an intersection when the signal timings are optimum and is given by

$$X_o = \frac{2Y}{1 + Y}$$

2.4.9 Junction capacity Analysis

2.4.9.1 The ultimate capacity of an intersection may be defined as the maximum flow which can pass through the intersection with the same relative flows on the various approaches and with the existing proportions of turning traffic.

2.4.9.2 Generally capacity will increase as the cycle time increases, since the ratio of lost time to useful time decreases (the effect becomes negligible when the cycle is very long). In practice, for maximum reserve capacity assessment, a maximum cycle time of 120 seconds should be adopted. However it should be noted that for new installations, the maximum operating cycle time should be limited to 90 seconds (See 2.4.6)

2.4.9.3 If the capacity were taken as the flow which could just be accommodated by such a cycle the delays would normally be excessively high. A practical capacity of 90 per center of this maximum possible flow, which produces generally acceptable delays, is recommended.

2.4.9.4 To calculate reserve capacity at 120 sec cycle time

The cycle time which is just long enough to pass all the traffic, is given by

$$C_m = \frac{L}{1 - Y}$$

Similarly the maximum Y which could be accommodated by a fixed cycle time c is given by

$$Y_{max} = 1 - \frac{L}{c}$$

For c_m = 120 sec

$$Y_{max} = 1 - \frac{L}{120}$$

For practical purposes, the ultimate Y value, Yult, is taken as 90 per cent of Ymax.
Yult = 0.9 - 0.0075 L
The percentage of ultimate reserve capacity is then given by

$$R.C.(ult) = \frac{(Yult - Y)}{Y} \times 100\%$$

2.4.9.5 To calculate Reserve Capacity at any cycle time c, R.C.(c)

(i) Sometimes it may be of interest to know the reserve capacity of a junction operating in the current cycle time i.e. R.C.(c) if c = current cycle time

$$Y_{max} = 1 - \frac{L}{c}$$

and assuming a practical Y value of 0.9 Ymax

$$R.C.(c) = \frac{0.9Y_{max} - Y}{Y} \times 100\%$$

(ii) Care should be taken in performing this calculation if parallel pedestrian facilities are included in the method of control. For minor movements particularly, the minimum green time for the approach can well be dictated by the parallel pedestrian crossing green times. This could well considerably distort the R.C. calculation.